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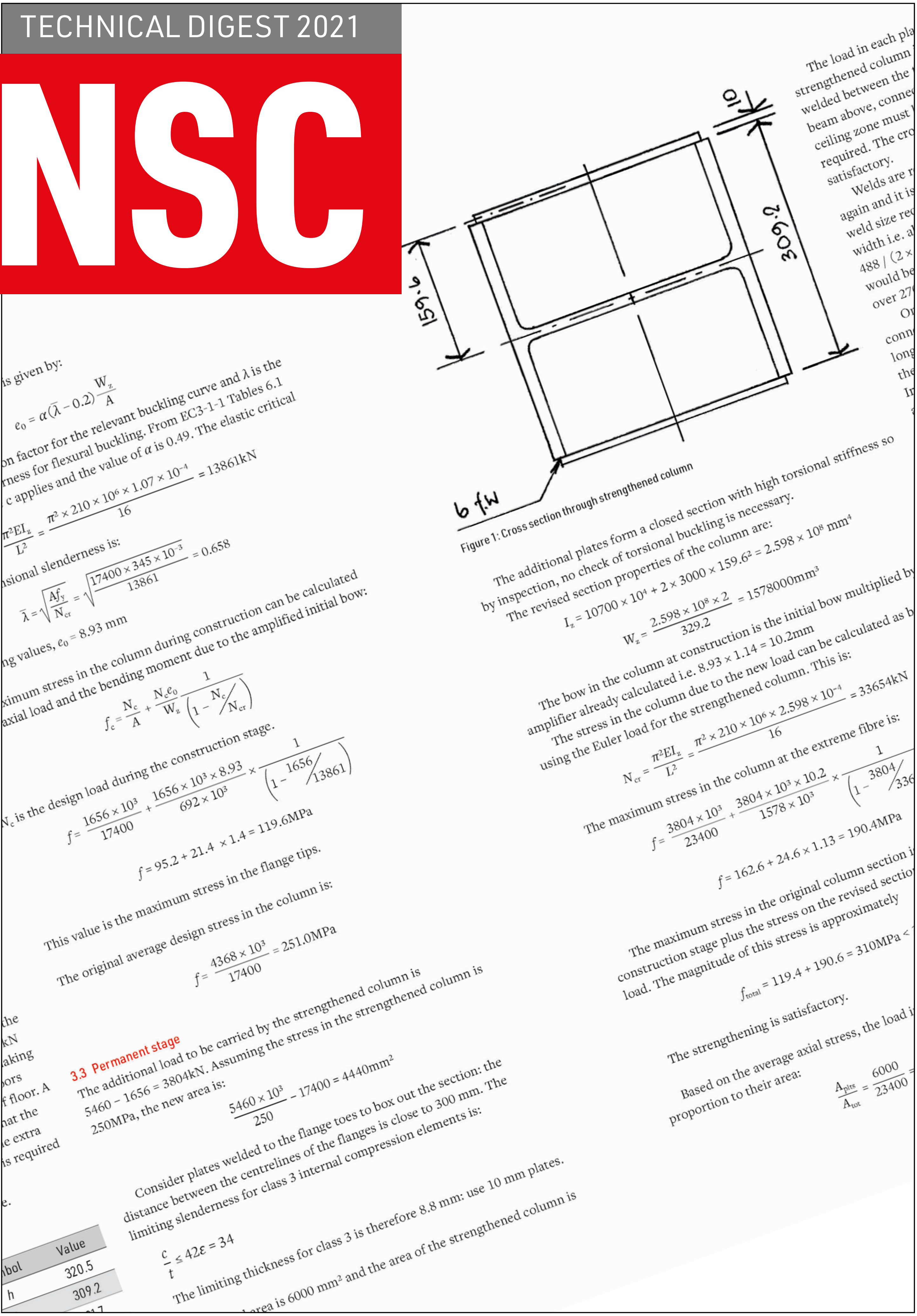


...is given by:

$$e_0 = \alpha(\bar{\lambda} - 0.2)\frac{W_z}{A}$$

...on factor for the relevant buckling curve and λ is the ...rness for flexural buckling. From EC3-1-1 Tables 6.1 ...c applies and the value of α is 0.49. The elastic critical

$$\frac{\pi^2 EI_z}{L^2} = \frac{\pi^2 \times 210 \times 10^6 \times 1.07 \times 10^{-4}}{16} = 13861\text{kN}$$

...sional slenderness is:

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} = \sqrt{\frac{17400 \times 345 \times 10^{-3}}{13861}} = 0.658$$

...ng values, $e_0 = 8.93$ mm

...ximum stress in the column during construction can be calculated ...axial load and the bending moment due to the amplified initial bow:

$$f_c = \frac{N_c}{A} + \frac{N_c e_0}{W_z}\frac{1}{\left(1 - N_c/N_{cr}\right)}$$

N_c is the design load during the construction stage.

$$f = \frac{1656 \times 10^3}{17400} + \frac{1656 \times 10^3 \times 8.93}{692 \times 10^3} \times \frac{1}{\left(1 - 1656/13861\right)}$$

$$f = 95.2 + 21.4 \times 1.4 = 119.6\text{MPa}$$

This value is the maximum stress in the flange tips.

The original average design stress in the column is:

$$f = \frac{4368 \times 10^3}{17400} = 251.0\text{MPa}$$

...the ...kN ...aking ...ors ...f floor. A ...hat the ...e extra ...is required ...e.

...bol	Value
	320.5
h	309.2

3.3 Permanent stage

The additional load to be carried by the strengthened column is 5460 − 1656 = 3804kN. Assuming the stress in the strengthened column is 250MPa, the new area is:

$$\frac{5460 \times 10^3}{250} - 17400 = 4440\text{mm}^2$$

Consider plates welded to the flange toes to box out the section: the distance between the centrelines of the flanges is close to 300 mm. The limiting slenderness for class 3 internal compression elements is:

$$\frac{c}{t} \le 42\varepsilon = 34$$

The limiting thickness for class 3 is therefore 8.8 mm: use 10 mm plates.

...area is 6000 mm² and the area of the strengthened column is

Figure 1: Cross section through strengthened column

The additional plates form a closed section with high torsional stiffness so by inspection, no check of torsional buckling is necessary.

The revised section properties of the column are:

$$I_z = 10700 \times 10^4 + 2 \times 3000 \times 159.6^2 = 2.598 \times 10^8\ \text{mm}^4$$

$$W_z = \frac{2.598 \times 10^8 \times 2}{329.2} = 1578000\text{mm}^3$$

The bow in the column at construction is the initial bow multiplied by amplifier already calculated i.e. 8.93 × 1.14 = 10.2mm

The stress in the column due to the new load can be calculated as b... using the Euler load for the strengthened column. This is:

$$N_{cr} = \frac{\pi^2 EI_z}{L^2} = \frac{\pi^2 \times 210 \times 10^6 \times 2.598 \times 10^{-4}}{16} = 33654\text{kN}$$

The maximum stress in the column at the extreme fibre is:

$$f = \frac{3804 \times 10^3}{23400} + \frac{3804 \times 10^3 \times 10.2}{1578 \times 10^3} \times \frac{1}{\left(1 - 3804/336...\right)}$$

$$f = 162.6 + 24.6 \times 1.13 = 190.4\text{MPa}$$

The maximum stress in the original column section i... construction stage plus the stress on the revised sectio... load. The magnitude of this stress is approximately

$$f_{total} = 119.4 + 190.6 = 310\text{MPa} < ...$$

The strengthening is satisfactory.

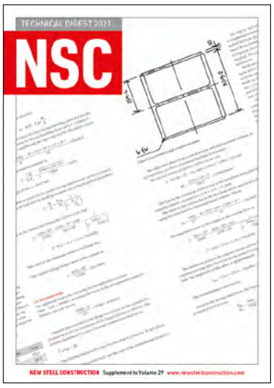
Based on the average axial stress, the load i... proportion to their area:

$$\frac{A_{plts}}{A_{tot}} = \frac{6000}{23400} = ...$$

The load in each pla... strengthened column ... welded between the ... beam above, conne... ceiling zone must ... required. The cro... satisfactory.

Welds are r... again and it is... weld size req... width i.e. a... 488 / (2 × ... would be... over 27...

O... conn... long... the... In...



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Essential reading for net-zero carbon designers

Nick Barrett - Editor

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

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

The Digest is part of the steel construction sector's long-established commitment to keep designers in steel up-to-date with the latest technical guidance to help them take advantage of the numerous benefits of steel as a sustainable construction material, which is more important than ever as the construction industry gets fully behind the drive to net-zero carbon.

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

NSC is a popular source of advice and news, and is where the highly regarded Advisory Desk Notes and longer Technical Articles are first published, and immediately made available on *newsteelconstruction.com*.

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

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

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

U-Frame action design according to Eurocodes

Ricardo Pimentel of the SCI discusses the consideration of U-Frame action to restrain members susceptible to flexural and lateral torsional buckling according to Eurocodes.

Introduction

Buckling phenomena frequently govern the design of steel members under compression or for elements partially compressed. To achieve a good compromise between steel tonnage and performance, discrete restraints along the compressed member (or along the compressed part of the member) can be used. However, for certain cases, introducing restraints as part of an orthodox bracing system is not feasible and designers must use other options to achieve a capable structural solution. The use of U-frame action offers this opportunity.

U-Frame action general principles

The classic U-Frame action example can be found in "half-through" railway bridges[1],[2] or pedestrian bridges. The key concept of the U-frame action is illustrated in Figure 1a. The two longitudinal girders are subjected to a sagging bending moment, which causes compression in the top flanges. At certain locations along the bridge span, a continuous U-shaped frame is formed from the horizontal deck beams and vertical elements in the main girders – usually full depth stiffeners welded to the web. There is a stiff connection between the end of the deck beam and the vertical elements, so that an appropriate bending stiffness between the vertical elements and the floor beams is achieved. The flexural stiffness of the U-frame provides discrete spring restraints to the compressed beam flanges. These elastic restraints increase the resistance of the girders to lateral torsional buckling. The same principle can be applied to a trussed solution (Figure 1b). If the vertical posts of the truss are connected to the adjacent floor beam, the top compressed chords will have spring restraints, which will increase the out-of-plane flexural buckling resistance of the chord. The concepts described may be extended to other forms of construction based on the same principles.

Although the concept of U-frame action is often related to "half-through" railway bridges or pedestrian bridges, the concept may be also used when designing conventional downstand composite bridge beams during the construction stage or to prevent lateral torsional buckling of the compressed bottom flanges near the bridge internal supports. The typical configuration shown in Figure 1a may not suffice and bracing elements (typically forming a "K" shaped bracing arrangement) or haunched cross beam solutions may be provided to increase the stiffness and effectiveness of the restraints, to provide an effective torsional bracing or simply to establish clear segments for the beam buckling verification.

Structures such as portal frames[3],[4], multi-storey buildings with continuous composite beams or arched bridges may also rely on U-frame action to provide restraint against buckling.

General advice and design principles

The stiffness of the U-frames is the key for the structural behaviour and design. Care must be taken not only while selecting the members sizes but also while undertaking the connections design and detailing.

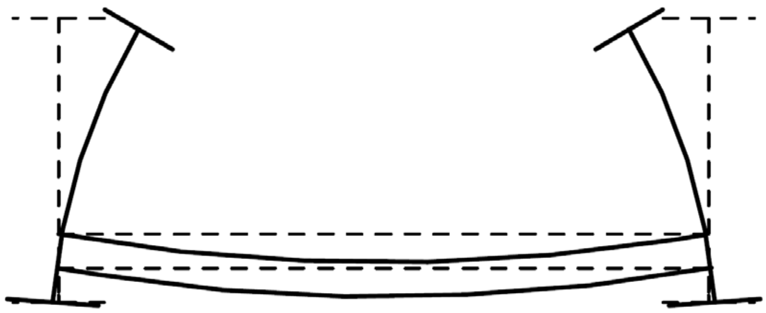
Semi-rigid connections will decrease the stiffness of the U-frame, which in turn will decrease the stiffness of the point restraints and therefore the buckling resistance of the restrained elements. The joint stiffness classification can be assessed based on EN 1993-1-8[5]. The stiffness of semi-rigid joints must be used while assessing the U-frame behaviour.

In addition to the stiffness of the elements forming the U-frame, the resistance of the elements must also be checked including second order effects. Guidance on the design is provided in references [6], [7], [8] and [9].

The objective when considering U-frame action is to address the elastic stability problem of the elements that are being restrained – typically the elastic critical buckling force for the compression element restrained by intermediate spring supports. Designers will then use typical buckling resistance verifications based on buckling curves according to Eurocode 3. Even for U-frame action in composite structures, the lateral torsional buckling resistance will still be based on the buckling curves from EN 1993-1-1[10] section 6.3.

U-Frame action in composite structures

Typical UK composite practice assumes simply supported beams, which offers a good compromise considering structural capacity, slab detailing and straightforward analysis and design. For certain cases – due to say high loading or atypical requirements for serviceability floor performance – a continuous solution may be utilised. The unrestrained steelwork of a continuous composite beam experiences compression over the supports and therefore is susceptible to local and member buckling. To avoid the unattractive practice of introducing restraints to the beam bottom flange, U-frame action may be considered, achieved by the combined behaviour of two adjacent parallel floor beams and the slab. The restraint to buckling is



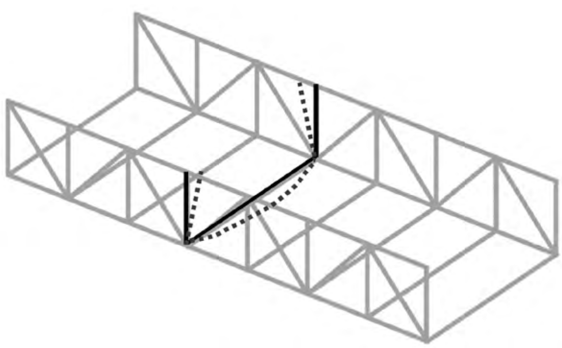


Figure 1: U-Frame action a) I-shaped girders – addressing lateral torsional buckling b) Trussed solutions – addressing flexural buckling

based on the stiffness k_s (Figure 2), which accounts for the stiffness of the beam web and the slab. The design is covered by Eurocode 4[11] and further guidance on the buckling resistance of continuous composite beams can be found in reference [12].

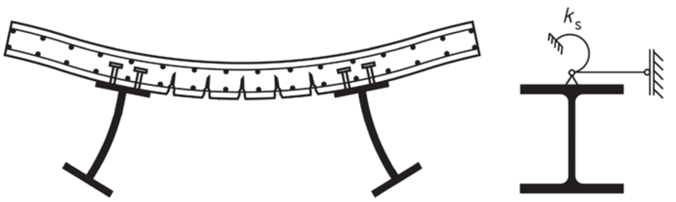


Figure 2: U-Frame action in a continuous composite beam

U-frame stiffness and design

For the typical U-frame configurations such as the ones presented in Figure 1, the stiffness C_d can be calculated from EN 1993-2[6] Table D.2 as follows:

$$C_d = \frac{E\,I_v}{\frac{h_v^{\,3}}{3} + \frac{h^2 b_q I_v}{2I_q}}$$

Eq. (2)

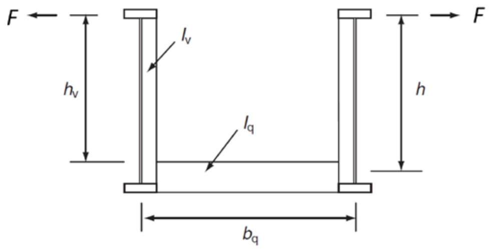


Figure 3: Generic model for U-frame stiffness calculation

where:

- I_v is the second moment of area of the vertical stiffeners;
- I_q is the second moment of area of the cross/horizontal member;
- h is the distance between the centroid of the compressed flange and the centroid of the cross member;
- h_v is the distance between the centroid of the compressed flange and the top of the cross member;
- b_q is the spacing of the main girders.

I_v may be calculated assuming the contribution of a web width equal to t_s + 30 ε t_w [13], where t_s is the thickness of the vertical stiffener, t_w is the thickness of the web, $\varepsilon = \sqrt{(235 / f_{y,w})}$ and $f_{y,w}$ is the yield strength of the web. If flange plates are welded to the vertical stiffeners, the inertia I_v can be calculated based on the obtained equivalent “I” section. Higher stiffnesses can be achieved by specifying two double “T” shaped stiffeners on each side of the main girder web, which can each be cut from a standard “I” or “H” section. The stiffness of the connections between cross beams and verticals may be accounted for in equation 2 by adding the term h^2EI_v/S_j in the denominator, where S_j is the stiffness of the connection[2].

The assessment of U-frame stiffness may seem straightforward for the orthodox configuration shown in Figure 3. However, for certain cases the designer may wish to prepare a simple FE model from which the stiffness can be calculated. The stiffness can be calculated by applying a pair of forces “F” (Figure 3) in the U-frame, measuring the deflection at the tip of the flange and dividing the load by the measured deflection. The stiffness (C_d) may then be used to assess the elastic critical force of a top chord of a truss or for a compressed flange of a beam susceptible to lateral torsional buckling.

The elastic critical buckling force can be calculated based on an analytical approach or determined from FE models. The analytical approach, based on a beam supported by an elastic foundation, can be undertaken as follows, based on EN 1993-2 section 6.3.4.2 (6):

$$N_{crit} = mN_E \qquad \text{Eq. (3)}$$

where:

$N_E = \dfrac{\pi^2 EI}{L^2}$, where L is the distance between rigid braces;

$m = \dfrac{2}{\pi^2}\sqrt{\gamma} \geq 1$ where $\gamma = cL^4/EI$ and $c = C_d/l$, in which l is the distance between U-frames.

From equation (3), the buckling length of the compressed member can be obtained from:

$$l_{crit} = \sqrt{\frac{\pi^2 EI}{N_{crit}}}$$

Equation (3) assumes that the end frames are rigid, which will not be the case for most practical cases. The influence of the flexibility of the supports may be considered by replacing the variable “m” in Equation (3) by[14], [7]:

$m_e = \dfrac{\sqrt{\gamma}}{\left(\dfrac{\pi}{\sqrt{2}} + \dfrac{0.69}{X + 0.5}\right)^2}$, where $X = \dfrac{C_e}{\sqrt{2}}\left(\dfrac{l^3}{C_d^{\,3}EI}\right)^{0.25}$ and C_e is the stiffness of the end frame according to equation (2).

The stiffness of the end supports will only have an influence in the design of critical segments close to the supports. The influence of the stiffness of the supports may be neglected for segments located at least 2.5 × l_{crit} from the supports[7].

Design for flexural buckling according to Eurocode 3

Consider a 20 m span pedestrian bridge with two 1575 mm deep longitudinal warren trusses with additional vertical members as shown in Figure 4. The trusses are 3 m apart and the verticals spaced at approximately 1.67 m. Five U-frames are provided in the structural solution: 2 end frames and 3 interior equally spaced frames (5 m between U-frames). The axial loads in the chords are estimated as 550 kN at midspan. The design will be based on EN 1993-1-1 section 6.3.2.4 (1) or EN 1993-2 6.3.4.2.

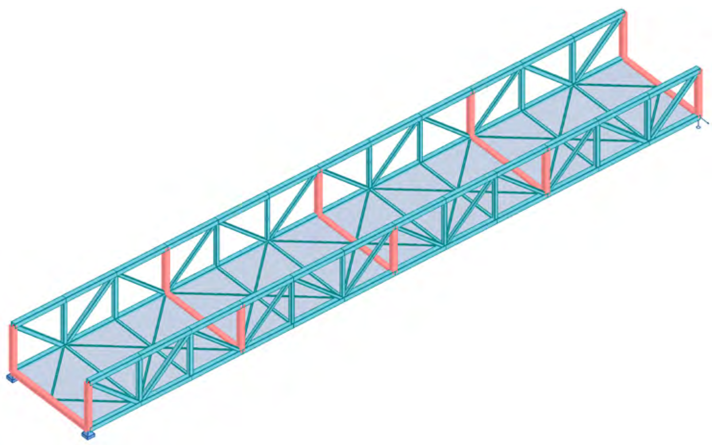


Figure 4: Trussed pedestrian bridge with 20 m span

A hot finished SHS 150 × 150 × 6.3 was selected for the preliminary design, which gives a buckling resistance of 673 kN for a 5 m buckling length and γ_{M1} = 1.10. The cross beams at U-frame locations have the same cross section (SHS 150 × 150 × 6.3: I = 1220 cm⁴).

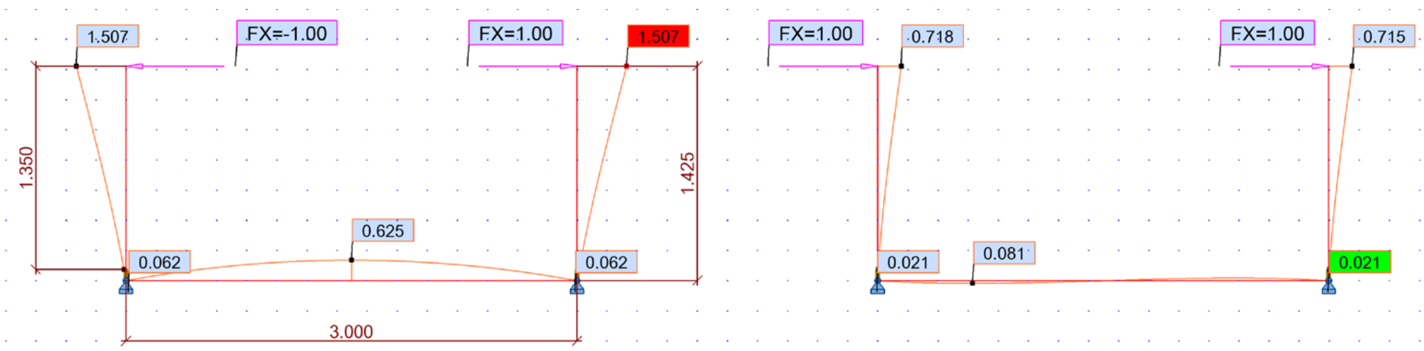


Figure 5: U-Frame stiffness based on a simple FE model[17]

The stiffness of the U-frames can be calculated as follows, using equation (2):

$$C_d = \frac{210 \times 10^6 \times 1220 \times 10^{-8}}{\frac{1.35^3}{3} + \frac{1.425^2 \times 3 \times 1220 \times 10^{-8}}{2 \times 1220 \times 10^{-8}}} = 662.69 \text{ kN/m}$$

A simple FE model with bar elements was completed to compare the calculated U-frame stiffness. Rigid links were used to model the distance h_v. Forces with opposite directions represent the critical mode (producing higher deflection). From the analysis results, the deflection is 1.507 mm under opposing forces of 1 kN. The spring stiffness can be obtained as follows:

$C_{d,FEM} = \frac{1}{1.507 \cdot 10^{-3}} = 663.57$ kN/m which is very close to the value of 662.69 kN/m previously calculated (See Fig. 5).

Having calculated the U-frame stiffness, the elastic critical force can be determined:

$$c = \frac{C_d}{l} = \frac{662.69}{5} = 132.54 \text{ kN/m}^2$$

$$\gamma = 132.54 \times 20^4/(210 \times 10^6 \times 1220 \times 10^{-8}) = 8277.16$$

$$m = \frac{2}{\pi^2} \times \sqrt{8277.16} = 18.44$$

$$N_E = \frac{\pi^2 \times 210 \times 10^6 \times 1220 \cdot 10^{-8}}{20^2} = 63.21 \text{ kN}$$

$$N_{crit} = 18.44 \times 63.21 = 1165.44 \text{ kN}$$

The benchmark buckling resistance of the chord assuming a buckling length between U-frames is:

$$N_{E,5m} = \frac{\pi^2 \times 210 \times 10^6 \times 1220 \times 10^{-8}}{5^2} = 1011.44 \text{ kN}$$

It can be concluded that the U-frames are of benefit in restraining the chord if the end restraints were rigid, as the critical load (1165.44 kN) is higher in comparison with the value obtained (1011.44 kN) considering the distance between U-frames for the chord buckling length.

As the end restraints are also U-frames (and therefore flexible), the values calculated previously are not accurate. The flexibility of the end supports needs to be considered as follows:

$$X = \frac{662.69}{\sqrt{2}} \left(\frac{5^3}{662.69^3 \times 210 \times 10^6 \times 1220 \times 10^{-8}} \right)^{0.25} = 1.69 \text{ m}$$

$$m_e = \frac{\sqrt{8277.16}}{\left(\frac{\pi}{\sqrt{2}} + \frac{0.69}{1.69 + 0.5} \right)^2} = 14.13$$

$$N_{crit} = 14.13 \times 63.21 = 893.50 \text{ kN}$$

The chord effective length can be back-calculated as follows:

$$l_{crit} = \sqrt{\frac{\pi^2 \times 210 \times 10^6 \times 1220 \times 10^{-8}}{893.50}} = 5.32 \text{ m}$$

The buckling length is shown to be a little longer than the system length.

As an alternative approach, a FE model was also completed to evaluate the elastic critical buckling load of the chord. The spring supports of 663.57 kN/m were modelled (including end frames). The buckling shape obtained for the cord is represented in Figure 6. Each bar was subdivided in 10 segments for accuracy[15],[16]. For simplicity, the axial load was considered constant along the chord, which is conservative. The analysis reports an elastic critical buckling resistance of N_{crit} = 985.42 kN.

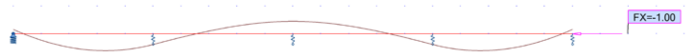


Figure 6: Chord elastic critical buckling load - flexible supports[17]
10 FE per par. Critical multiplier: α_{crit} = 985.42, N_{crit} = 985.42 kN

The effective length for the chord based on the FE analysis is therefore:

$$l_{crit} = \sqrt{\frac{\pi^2 \times 210 \times 10^6 \times 1220 \times 10^{-8}}{985.42}} = 5.07 \text{ m}$$

A FE model was also completed to evaluate the impact of the axial load gradient in the compressed chord. The results are shown in Figure 7. The model with constant axial force shows a lower resistance, as the less stable end segments had a higher axial force. The analysis reports an elastic critical buckling resistance of N_{crit} = 1193.99 kN.

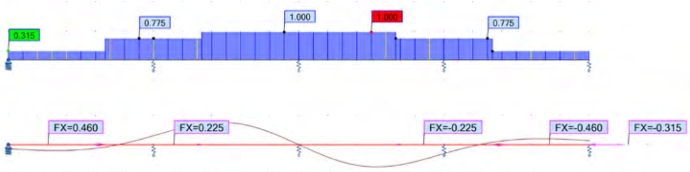


Figure 7: Chord elastic critical buckling load - flexible supports with axial load gradient[17]
10 FE per par. Critical multiplier: α_{crit} = 1193.99, N_{crit} = 1193.99 kN

The new effective length for the chord based on the FE analysis is therefore:

$$l_{crit} = \sqrt{\frac{\pi^2 \times 210 \times 10^6 \times 1220 \times 10^{-8}}{1193.99}} = 4.60 \text{ m}$$

It can be concluded that both analytical and numerical approaches show a good agreement. The chord member could then be checked against

EN 1993-1-1/EN 1993-2 rules for member stability using the back-calculated effective buckling length.

More sophisticated analyses may be undertaken where the elastic critical buckling load is obtained directly from 3D FE models. The designers must ensure that the internal bar releases and support conditions are correctly modelled to obtain a realistic behaviour of the structure. Such models may be completed using bar elements or with shell elements. The elastic critical buckling loads may differ from the more simplistic models due to the three-dimensional structural behaviour. Shell element models can be also used to more accurately obtain the U-frame stiffness as shown in Figure 5, which may also include the stiffness of the connections. The simple model represented in Figure 5 may also account for joint stiffness by including spring internal bar releases at the joints.

Design for lateral torsional buckling according to Eurocode 3

To illustrate the process applied to an "I" beam, consider a half-through bridge with a 42 m span. The main girders are spaced apart by 9 m. At a preliminary design stage, the top flanges were defined as 1000 × 120 mm plates, the bottom flanges 1500 × 70 mm and the webs 2810 × 20 mm. All plates were S460 (yield strength of 390 MPa for the top flange and 440 MPa for the web). The cross girders are spaced at 3.5 m centres along the bridge span and have a total depth of 700 mm, a centroid at half depth and a major axis second moment of area of 4.50 × 10⁹ mm⁴. The vertical "I"-shaped posts of the U-frame have a second moment of area of 1.20 × 10⁹ mm⁴, which comprises a gusset of 400 × 15 mm perpendicular to the web, an end plate of 425 × 40 mm and a web contribution of $\sqrt{(235 / 440)} \times 30 \times 20 + 15 =$ 453.50 mm.

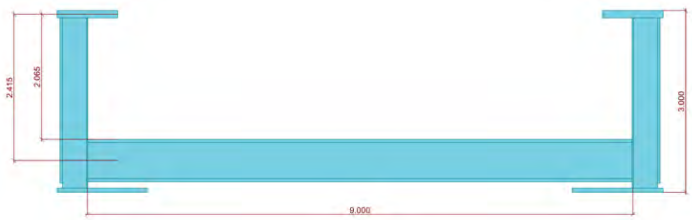


Figure 8: Half-through bridge with plated girder worked example geometry

The challenge of the worked example is to check the main beam for lateral torsional buckling accounting for the contribution of the U-frame action provided by the cross beams and vertical stiffeners. The design will be undertaken based on EN 1993-1-1 section 6.3.2.4(1) / EN 1993-2 6.3.4.2. The process will be analogous to the one described in the previous worked example, but for this case an equivalent compression strut needs to be defined for the process. EN 1993-2 clause 6.3.4.2(7) states that the area of the equivalent strut may be assumed the area of the compressed flange plus 1/3 of the compressed web area, accounting for the effective web area due to local plate buckling. The main beam cross section is class 4 under sagging bending moment due to the slender web. The effective cross section area is represented in Figure 9. In this example, the effective area of the equivalent strut is therefore A_{eff} = 1000 × 120 + 20 × (373.85 + 560.77) / 3 = 126230.80 mm². The equivalent strut out of plane second moment of area is based on the flange plate (ignoring the small contribution of the web), which has a value of $I = 1.0 \times 10^{10}$ mm⁴.

According to Figure 3, the example comprises the following data:

I_v [mm⁴]	1.20 × 10⁹
I_q [mm⁴]	4.5 × 10⁹
h_v [mm]	2065
h [mm]	2415
b_q [mm]	9000
C_d [kN/m]	25367.74
C_e [kN/m]	25367.74
C [kN/m²]	7247.92
γ	10739.68
m	21.00
X [m]	0.60
m_e	12.77
N_e [kN]	11749.53
N_{crit} [kN]	150035.68
A_{eff} [mm²]	126230.80

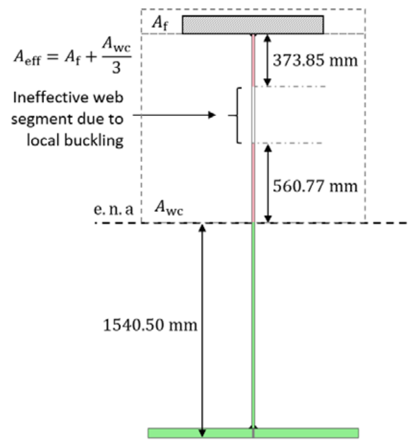


Figure 9 : Main beam effective area[18]

The normalized slenderness to calculate the lateral torsional buckling resistance of the girder can be calculated as follows – EN 1993-2 section 9.3.4.2(4):

$$\bar{\lambda}_{LT} = \sqrt{\frac{A_{eff} \times f_y}{N_{crit}}} = \sqrt{\frac{126230.80 \times 390 \times 10^{-3}}{150035.68}} = 0.57$$

As $\bar{\lambda}_{LT} > 0.20$ the beam is susceptible to lateral torsional buckling. With the value of $\bar{\lambda}_{LT}$, the buckling verification according to EN 1993-1-1 section 6.3 can be undertaken.

Similar FE models could be used to evaluate the elastic critical buckling resistance as described for the previous pedestrian bridge worked example.

The calculations presented above assume that the flange is subjected to a constant bending moment. The bending moment gradient for the segment under analysis may be allowed for according to EN 1993-2 section 6.3.4.2 (7) Note.

For the cases where the concrete deck is effectively connected to the beam webs, the stiffness I_q may be considered as the second moment of area of the deck (see reference [12]) and $I_v = t_w^3/12(1 - \nu^2)$, where ν is Poisson's ratio[19].

Conclusions

1. U-frame action is present in many forms of construction, from single- and multi-storey buildings to bridges, related with steel or composite structures;
2. U-frame action may be used to provide elastic (spring) restraints to elements which at first sight may look like an unrestrained component, such as a compressed chord susceptible to flexural buckling or a flange of a "I" girder susceptible to lateral torsional buckling;
3. The stiffness of the connections between U-frame elements is important if the stiffness of the U-frame is to be correctly calculated;
4. U-frame action is used with elastic stability calculations, so that N_{crit} and then $\bar{\lambda}$ may be calculated for elements susceptible to flexural buckling.
5. For members subject to lateral-torsional buckling, a non-dimensional slenderness $\bar{\lambda}_{LT}$ may be calculated.
6. U-frame action can be accounted for by undertaking an analytical procedure based on available guidance or based on a more general method using FE analysis; in the examples shown, a good agreement between the two possible design routines was achieved.

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New guidance on fire resistance of galvanized steel sections

By Dr Francisco Meza, Principal Engineer, SCI

Galvanized steel carpark at Sky Headquarters (London). Photo: Philip Durrant

Introduction

Galvanizing to EN ISO 1461[1] is commonly used to provide protection against corrosion for a wide variety of steel components, ranging in size from nuts and bolts to large structural sections. The process involves dipping steel components into molten zinc (which is usually around 450°C) for a few minutes. Unlike a paint coating, the metallurgical bond that is formed through galvanizing becomes part of the steel itself and is not merely a chemical or mechanical bond. As a result, galvanized steel not only provides corrosion protection but also has a high resistance to mechanical damage during handling, storage, transport and erection.

The zinc protective layer also provides a reduced surface emissivity of the steel component, which influences the rate at which the temperature of a steel section increases when exposed to a source of heat. Laboratory and full-scale testing[2,3], have demonstrated that below approximately 500°C, the galvanized coating remains stable, and its surface emissivity is around half of that for non-galvanized steel. A galvanized steel section will therefore heat up at a slower rate than an equivalent non-galvanized section which means an increased duration of fire resistance or increased load bearing resistance for a given fire exposure period.

Temperature increase of a steel member under fire conditions

Heat transfer to a steel member is predominantly by two mechanisms — radiation and convection. EN 1993-1-2[4], clause 4.2.5.1 gives a simple heat transfer model, which is used to determine the increase in temperature of a steel member $\Delta\theta_{a,t}$ over a small time interval Δt of no larger than 5 seconds. This simple heat transfer model is given by equation (1).

$$\Delta\theta_{a,t} = k_{sh} \frac{A_m/V}{c_a \rho_a} \dot{h}_{net} \Delta t \quad [K] \qquad \text{Eq. (1)}$$

where:

$\dot{h}_{net}$ is the design value of net heat flux per unit area [W/m²]
c_a is the specific heat of steel [J/kgK]
ρ_a is the density of steel [kg/m³]
A_m/V is the section factor of the member, per unit length [m^{-1}]
k_{sh} is a correction factor, commonly attributed to the shadow effect of flanges

Equation (1) has to be solved following an iterative procedure because the specific heat c_a and the net heat flux $\dot{h}_{net}$ are both temperature dependent. The

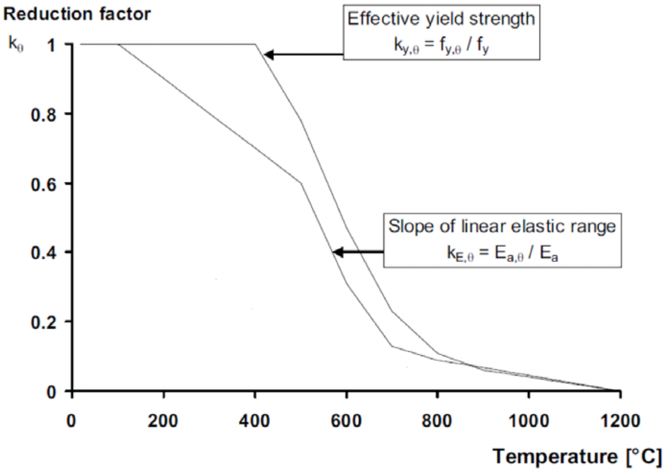

Figure 1: Strength and stiffness reduction factors of steel at elevated temperatures

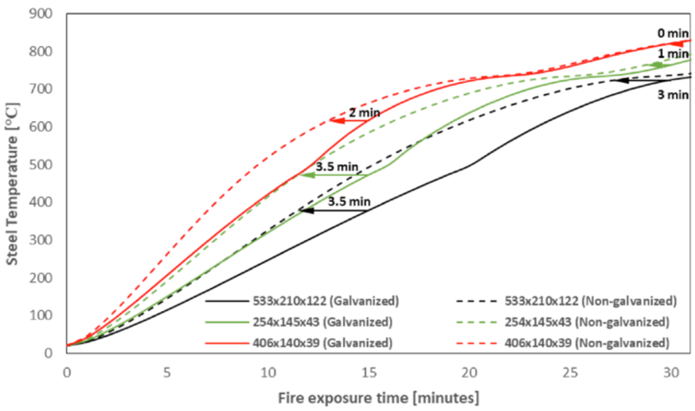

Figure 2: Temperature rise of galvanized and non-galvanized steel sections subject to the standard nominal fire curve

temperature reached by a steel member at a given time in a fire can then be determined by summing the small increments in temperature $\Delta\theta_{a,t}$ over the total time of fire exposure.

The net heat flux $\dot{h}_{net}$ to the surface of a steel member is given in EN 1991-1-2, clause 3.1 as the sum of the heat transfers by convection $\dot{h}_{net,c}$ and by radiation $\dot{h}_{net,r}$, expressed as:

$\dot{h}_{net} = \dot{h}_{net,c} + \dot{h}_{net,r}$ [W/m²] Eq. (2)

The convective heat flux is calculated as:

$\dot{h}_{net,c} = \alpha_c (\theta_g + \theta_a)$ [W/m²] Eq. (3)

where:

- α_c is the coefficient of heat transfer by convection, taken as $\alpha_c = 25$ [W/m²K] when the standard temperature-time curve is used
- θ_g is the gas temperature in the vicinity of the fire exposed member [°C]
- θ_a is the surface temperature of the member [°C]

The radiant heat flux is calculated as:

$\dot{h}_{net,r} = \phi\, \varepsilon_m \varepsilon_f\, \sigma[(\theta_r + 273)^4 - (\theta_a + 273)^4]$ [W/m²] Eq. (4)

where:

- ϕ is the configuration factor, conservatively taken as 1.0
- ε_m is the surface emissivity of the member
- σ is the Stephan Boltzmann constant, $\sigma = 5.67 \times 10^{-8}$ [W/m²K⁴]
- ε_f is the emissivity of the fire, which is generally taken as 1.0
- θ_r is the effective radiation temperature of the fire environment, which for fully fire engulfed members may be taken as $\theta_r = \theta_g$ [°C]
- θ_r is the surface temperature of the member [°C]

The density and specific heat of galvanized steel is the same as that of non-galvanized steel, and they can be determined in accordance with EN 1993-1-2, clauses 3.2.2 and 3.4.1.2, respectively. The surface emissivity of non-galvanized steel is given in EN 1993-1-2, clause 2.2 as $\varepsilon_m = 0.70$ for all temperatures. An emissivity value for galvanized steel has now been derived from studies by a number of European researchers, and an amendment to EN 1993-1-2 will be included in the next revision of the standard (due to be published in about 2023) in which the surface emissivity for galvanized steel will be given as:

$\varepsilon_m = 0.35$ for $\theta_a \leq 500°C$

$\varepsilon_m = 0.70$ for $\theta_a > 500°C$

Therefore, when calculating the increase in temperature of a galvanized steel member, all the required parameters (with the exception of the surface emissivity) are the same as those used to determine the increase in temperature of a geometrically equivalent (i.e. same section factor A_m/V and correction factor k_{sh}) non-galvanized steel member. The slower temperature increase in a galvanized steel member is therefore only due to the lower radiant heat flux introduced, as shown by equation (4).

Fire resistance

The design resistance of a steel member in fire is determined in a similar manner as the design resistance at room temperature, with an allowance made for the reduction in the relevant mechanical properties of the steel at elevated temperatures. When the resistance is not governed by member instabilities, such as the resistance of tension members, or the bending moment resistance of laterally restrained beams, the only material parameter affecting the resistance is the yield strength, and its reduction with temperature is accounted for through the reduction factor $k_{y,\theta} = f_{y,\theta}/f_y$, where $f_{y,\theta}$ is the yield strength at elevated temperature, and f_y is the yield strength at room temperature. When the resistance is governed by member instabilities, such as columns susceptible to flexural buckling, the resistance is also affected by the reduction in stiffness of the steel with temperature, which is accounted for through the reduction factor $k_{E,\theta} = E_{a,\theta}/E_a$, where $E_{a,\theta}$ is the slope of the linear elastic range at elevated temperature, and E_a is the modulus of elasticity at room temperature.

EN 1993-1-2, Table 3.1 gives values for $k_{y,\theta}$ and $k_{E,\theta}$ at discrete temperatures ranging from 20°C to 1200°C. These are shown in Figure 1, and are applicable to both galvanized and non-galvanized steel. Therefore, for a given fire exposure, the slower temperature increase in galvanized steel can be expected to lead to structural members with a higher fire resistance than an equivalent non-galvanized steel member. Or put in other words, when subject to the same

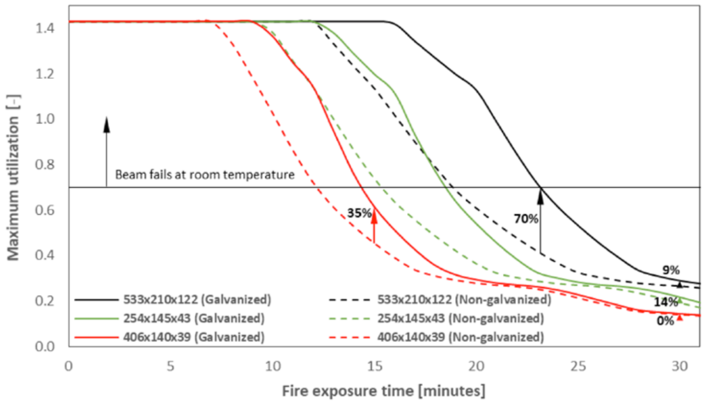

Figure 3: Fire resistance of galvanized and non-galvanized steel beams exposed to fire on three sides as a function of time

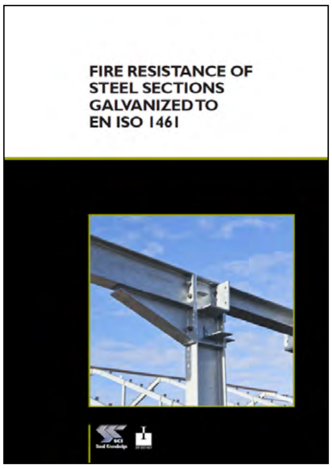

Figure 4: New SCI publication (P429) for the fire design of galvanized steel members

loading conditions, a galvanized steel member can be expected to achieve a longer fire exposure than an equivalent non-galvanized steel member.

Benefit of using galvanized steel in fire

The benefit of utilizing galvanized steel members for fire resistance is apparent in structures that require short fire resistance periods, that is, 15 or 30 minutes of fire exposure, where the temperature reached by the galvanized steel members is around 500°C. Examples of structures that require such fire resistance periods include car parks and single-storey residential/office buildings[5]. There may also be benefit in using galvanized steel for other types of structures, such as single storey industrial buildings or some multi-storey office buildings, where the use of sprinklers may enable a reduction of the minimum fire period to 30 minutes.

Another important factor that affects the rate at which the temperature in a steel member increases is the section factor. In EN 1993-1-2, the section factor is defined as the surface area of the member exposed to a fire per unit length, A_m, divided by the volume per unit length, V. Therefore, a beam exposed to a fire on four sides has a higher section factor than an equivalent one exposed on three sides. This factor has the same effect irrespective of whether the section is galvanized or non-galvanized, as it only depends on the geometric proportions of the cross-section.

Figure 2 compares the rise in steel temperature of galvanized and non-galvanized steel beams for three different Universal Beam sections (533 × 210 × 122, 254 × 146 × 43, 406 × 140 × 39) exposed to fire from three sides with section factors $k_{sh}\ [A_m/V]_m$ of 75 m^{-1}, 109 m^{-1} and 170 m^{-1}, respectively. The figure shows that by using galvanized steel, the maximum fire exposure can be increased by up to 23 %. If the gains in fire exposure time using galvanized steel are translated into increased resistance, the advantages are more pronounced. Figure 3 shows that for the steel beams discussed, the resistance (or utilization) at 15 minutes fire exposure can be increased by up to 35 % as a result of galvanizing.

SCI publication for the design of galvanized steel members in fire

As shown here, the process of designing a steel member in fire is made complicated primarily due to the need to know the temperature of the member at the time of interest. This is in essence an iterative process which requires solving equation (1) hundreds of times. SCI has recently published a design guide which greatly simplifies the design of galvanized steel members in fire, avoiding any need for iteration[6] (Figure 4). The publication includes design tables to calculate fire resistances and maximum fire exposure periods for galvanized steel beams, composite beams, columns, and plates in tension, according to the Eurocodes[4,7], and the UK and Irish National Annexes. Design tables in accordance with BS 5950[8] are also provided. The design tables clearly show where the use of galvanized steel leads to an increase in fire resistance or fire exposure compared to non-galvanized steel. Worked examples are also provided to illustrate the use of the tables.

The publication is available as a free download from the SCI bookshop and Steelbiz (*https://portal.steel-sci.com/shop.html*) and Galvanizers Association website (*https:// www.galvanizing.org.uk*)

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Verification of beams subject to a hogging bending moment

David Brown of the SCI considers the solutions to this complex situation

What's the problem?

In buildings, beams are generally designed as simply supported. Even composite beams are assumed to be simply supported - when we readily appreciate they are not. Just occasionally, designers are faced with a member where the bending moment reverses at some point within the member length. In continuous floor beams there will usually be restraints to one flange only, so there will be a length where the other flange is unrestrained. The bending moment diagram for a continuous beam is of the form shown in Figure 1.

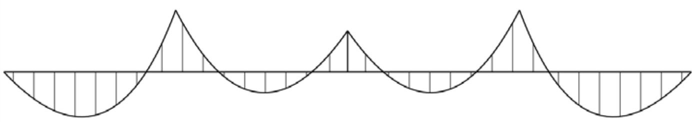

Figure 1: Continuous beam bending moment diagram

In this case, the top flange of the beam is usually restrained, possibly at intervals, but more commonly a continuous restraint to the top flange. In the hogging portions of the bending moment diagram, the bottom flange is in compression and unrestrained. The challenge is to verify this length.

"It's easy, the point of contraflexure is a restraint"

This is a common assumption, which the SCI is occasionally asked to endorse when applied to the floor beam considered in Figure 1. The idea comes from assumptions made in the design of portal frames, with the suggestion that this can be applied to floor beams. After all, the beam does not know if it is in a portal frame or in a floor.

The practice of assuming the point of contraflexure to be a "virtual lateral restraint" to the bottom flange has been enshrined in portal frame design for many years. The advice is found in clause 5.5.5 of BS 5950 and in SCI publications P252 (BS 5950) and P399 (Eurocode design). There are certain requirements to be met, which are clearly related to the idea of an "inverted U-frame", which has been covered in other *New Steel Construction* articles. The purlins must be sufficiently stiff – manifest as the rule that they must be at least 25% of the rafter depth. The connection to the flange must be sufficiently rigid – manifest as the rule that the connection from purlin to rafter must have at least two bolts. These rules had their origins in the 1970's, when purlins were hot rolled and rafters had tapered flanges. When discussing this question, Professor Horne commented "...even the small torsional restraint obtained with a continuous rail and two bolts in the cleat, but without a web stiffener, is sufficient to prevent the spread of torsional failure from a length of member with the outstand flange in compression to part of the member with the outstand flange in tension".

P399 subtly notes that the assumption of a virtual lateral restraint to the bottom flange is UK practice. Other designers may be suspicious of this bold assumption.

The suggestion is that this assumption may be applied to a floor beam which has a similar arrangement – restraint to the top flange and a bottom flange which changes from tension to compression.

Although many designers might have taken this route, verifying the member between the point of contraflexure and the support, published guidance prohibits this. In the *Designers' Guide to EN 1994-1-1*[1], section 6.4.1, we read "It should not be assumed that a point of contraflexure is equivalent to a lateral restraint".

In some situations, it is common practice to avoid the uncertainty altogether and provide a restraint at the required location. This is the typical solution for bridges, and (for example) trusses.

What are the alternatives?

In short, the answer is to "do it properly". The task is straightforward if the member is a bare steel beam. The proper approach is complicated if the member is a composite beam, so this article proposes that a conservative approach is to pretend the composite beam is in fact steel alone. There is a "simplified verification" method for composite beams in the design standard which does not involve any calculations (it does in the UK National Annex variation!) but as will be seen, the scope means it is of very limited use. The two approaches are examined in the following sections.

Bare steel beam

The solution here is to model the complete span in *LTbeam* or *LTBeamN*, ensure that the bending moment diagram is correct, model the correct restraints and use the software to determine M_{cr}. The calculation of the lateral torsional buckling resistance $M_{b,Rd}$ then follows the normal route. The resistance is checked against the largest moment in the span, which for a fixed ended beam and UDL, will be at the support.

Figure 2 shows the dimensions, loading and resulting bending moment diagram for a continuous beam with fully fixed supports. The hogging moment at the support is $wL^2/12$. The point of contraflexure is 2114 mm from each support.

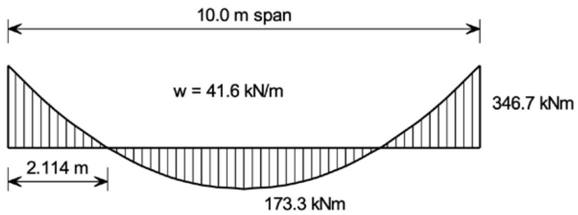

Figure 2: Beam and bending moment diagram

It is assumed that the beam is non-composite, but has a continuous lateral restraint to the top flange – presumably from whatever applies the UDL.

The beam may be modelled with fixed supports, or as simply supported but with a hogging moment applied at each end – it makes no difference to the value of M_{cr}. The selected beam is a 406 × 178 × 60, in S355. With a continuous lateral restraint to the top flange the value of M_{cr} = 1031 kNm. The buckled form is shown in Figure 3(a).

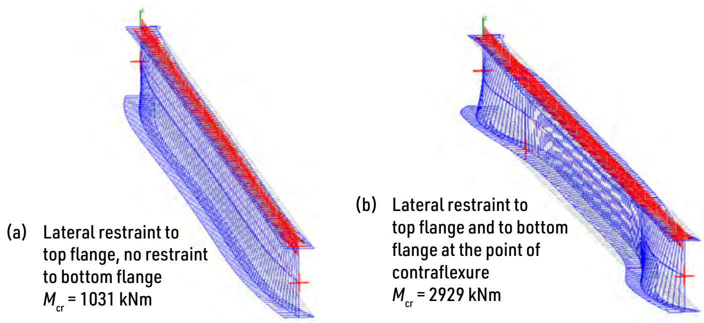

Figure 3: Buckled form of continuous beam

Completing the process:

$\bar{\lambda}_{LT} = 0.643$; $\phi_{LT} = 0.714$; $\chi_{LT} = 0.861$; $f = 0.819$

$\chi_{LT,Mod} = 1.0$

$M_{b,Rd} = 426$ kNm, > 347 kNm, OK.

If a restraint is introduced to the bottom flange at the point of contraflexure, the buckled form is shown in Figure 3(b), and $M_{cr} = 2929$ kNm – quite different to the real situation.

If the prohibited approach of simply checking from the point of contraflexure to the support had been followed (for interest, not a SCI recommendation!) $M_{cr} = 2426$ kNm, demonstrating that the elastic buckling moment is wildly different to that of the correctly modelled beam.

Composite beams

Verification of the hogging zone of a continuous composite beam is covered in clause 6.4.2 of BS EN 1994-1-1. The principles are straightforward and familiar – a pair of beams and the slab form an inverted U-frame, as shown in Figure 4 (taken from Figure 6.11 of the standard).

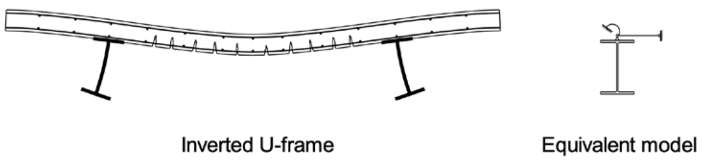

Figure 4: U-frame and model (from BS EN 1994-1-1)

The stiffness of the inverted U-frame depends on the stiffness of the slab, the stiffness of the beam web and (at least conceptually) the stiffness of the connection between beam and slab. Reference 1 notes that the flexibility of the shear connection between beam and slab can be neglected.

Once the stiffnesses have been calculated, M_{cr} for the composite member can be determined, which, as shown in Figure 4, is based on the member with a continuous lateral restraint to the top flange and a rotational spring stiffness at the same level.

Unfortunately, the process is not for the faint-hearted. Reference 1 notes that the calculation of the rotational spring stiffness is straightforward "apart from finding the cracked flexural stiffness of a composite slab". This calculation requires knowledge of the profiled slab dimensions, slab reinforcement and properties of the cracked composite section.

M_{cr} is then calculated, but this is for the composite section. The expression for a uniform steel beam cannot be used. The Eurocode does not give an expression, but this may be found in Reference 1. A value of the reduction factor χ_{LT} is calculated, but this is applied to the design resistance (in hogging) of the composite section. The resistance is compared to the hogging moment including the effects of shrinkage. In all, a complex set of calculations for designers who are not experienced in the detail of composite design – made even more complicated by the continuity which gives rise to hogging moments, the effects of cracking and shrinkage. Designers are commended to review example 6.7 in Reference 1 before undertaking their own verifications.

Simplified verification of composite beams

BS EN 1994-1-1 clause 6.4.3 offers the attractive prospect of a very much simplified approach "without direct calculation". In the core Eurocode, this is a simple test of the steel beam depth – below a tabulated maximum depth, there is no need to complete any calculation – the member is deemed to satisfy. The core Eurocode presents maximum depths for IPE sections. The UK National Annex demands the calculation of a "section parameter" in NA.2.8, which is a purely geometric parameter, but more involved than the section height limit in the core Eurocode.

This approach looks very appealing, but the associated conditions in clause 6.4.3(1) mean that in common practice, designers may be frustrated that they fall outside the scope. In addition to limitations on relative span lengths, the loading must be uniformly distributed – but critically, the design permanent load must exceed 40% of the design total load.

In the calculation which resulted in the design load of 41.6 kN/m used above, the characteristic loading was taken as $g_k = 3.0$ kN/m² and $q_k = 5.0$ kN/m², which is considered to be a reasonable pair of loads for a typical composite beam. The design loading is therefore:

Permanent: 1.35 × 3.0 = 4.05 kN/m²

Total: 1.35 × 3.0 + 1.5 × 5 = 11.55 kN/m²

The design permanent load is therefore only 35% of the design total load, so the use of the simplified approach is not permitted.

Conservative solution for composite beams

The approach proposed here may be very conservative, but it has the advantage of speed. If the bare steel beam is modelled with a lateral restraint (only) to the top flange, and found to be satisfactory, the composite member will also be satisfactory. Modelling as a bare steel beam neglects the contribution of the slab and the rotational spring stiffness.

As a comparison, consider example 6.7 in Reference 1. The verification in the hogging region concludes that the composite resistance of 767 kNm exceeds the ultimate moment with shrinkage included of 656 kNm. The steel beam is an IPE450 in S355, 12 m span and one half of a two-span continuous beam.

In *LTBeam* the loading was arranged to produce the correct hogging moment at the internal support. A continuous lateral restraint to the top flange was modelled. $M_{cr} = 1098$ kNm from this analysis.

Completing the process:

$\bar{\lambda}_{LT} = 0.741$; $\phi_{LT} = 0.789$; $\chi_{LT} = 0.800$; $f = 0.955$

$\chi_{LT,Mod} = 0.838$

$M_{b,Rd} = 505$ kNm, which is unsatisfactory and shows the method to be conservative.

This result could be improved if the rotational spring stiffness was included in the model – if the rather involved calculations were undertaken to determine the stiffness. Taking a significant short cut by adopting the value of 96.4 kNm/rad calculated in example 6.7, M_{cr} increases to 2234 kNm, and $M_{b,Rd} = 584$ kNm – still not satisfactory.

With only a lateral restraint to the top flange, an IPE500 delivers a resistance of 648 kNm, which is close enough to the 656 kNm requirement, recognising that there is benefit from the rotational spring stiffness at the top flange which has been neglected in the calculation.

Conclusions

1. The practice of assuming the point of contraflexure to be a virtual lateral restraint to the bottom flange is enshrined in the design standard for portal frames and confirmed by practice, but correctly prohibited for beams in buildings.
2. A simple buckling analysis shows that if the hogging length is assumed to be restrained at the point of contraflexure, the result is a significantly higher value of M_{cr} (i.e. an artificially high buckling resistance), and quite different to modelling the real condition.
3. If the member is bare steel, modelling the complete beam, with restraints (if any) is the straightforward and correct approach. Tools are available to calculate M_{cr}.
4. With a composite beam, the full process is complex. The codified simplified method is very limited in scope. Assuming the beam to be steel alone will be conservative.

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Fire resistance of light steel framing

Mark Lawson and Andrew Way of the Steel Construction Institute (SCI) discuss issues related to fire resistance of light steel framed buildings and introduce new guidance recently published by SCI.

Figure 1: Typical application of light steel loaded walls supporting a joisted floor

Figure 2: Light steel frame construction with metal decking for composite floors

Since the Grenfell fire disaster, the question of the fire safety of medium- and high-rise residential buildings has been heightened. Clients and checking authorities are understandably concerned about fire safety, particularly for buildings that exceed 18 m in height, and Regulations have been introduced to prevent the use of combustible materials in external walls. SCI has been working with members of the Light Steel Forum and other industry experts to update design guidance on the fire resistance of light steel framing, which is well established as a construction system for medium-rise residential and mixed-use buildings.

Steel has well-known properties at elevated temperatures and comprehensive design data is presented in BS EN 1993-1-2 and formerly in BS 5950-8 (dating from 1990). BS 5950-8 was the first fire engineering code worldwide and it influenced Eurocode developments. The critical temperature of structural steel beams and columns is taken as 550°C for the design of the fire protection to these members and this critical temperature increases as the proportionate loading (known as the load ratio) on the member reduces. Structural engineers are familiar with the design approach for structural steel but the application of methods for cold formed steel is the subject of the recent work by SCI.

Light steel framing has gained a market share because one of its benefits is that it is non-combustible and does not add to the fire load of the building. It may be used with joisted floors (Figure 1) or increasingly, with composite floor slabs that are supported by light steel load-bearing walls, as shown in Figure 2.

Strength retention of cold formed steel

Cold formed steel has slightly reduced strength retention properties at elevated temperatures compared to structural steel I and H sections and hollow sections because of the influence of local buckling of its thin profile. Nevertheless, the strength reduction factor (SRF) for Class 4 light steel sections at 550°C is still 0.41 of the nominal yield strength, as seen in Figure 3. This reduction in strength is broadly consistent with the reduction in load level at the fire limit state, which means a structure designed in the normal way at ambient temperatures is likely to be able to resist the reduced loads of the fire limit state.

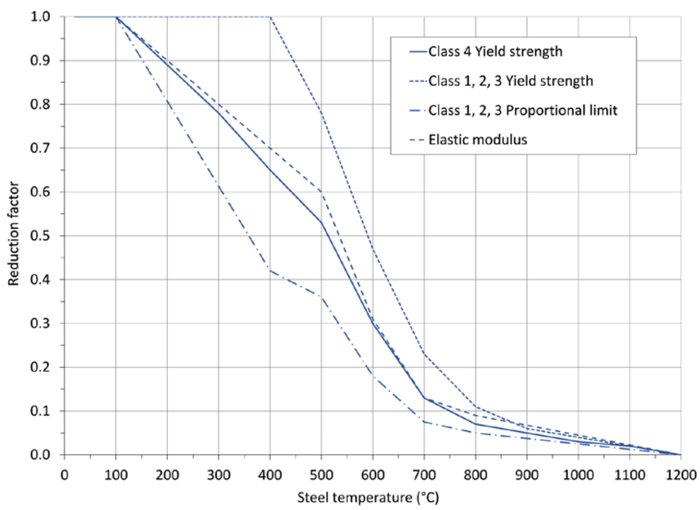

Figure 3: Strength and stiffness reduction factors for steel at elevated temperature

Light steel framing differs from structural steel in that it is a planar construction system. The 2D walls and floors are protected by layers of Type F or similar fire-rated plasterboards. In the last 3 years, an unprecedented number of loaded fire tests have been performed by light steel framing and plasterboard suppliers to satisfy 60, 90 and 120-minutes fire resistance requirements for loaded walls and floors.

A fire test on a loaded wall (Figure 4, opposite) is generally performed using the thinnest steel section in a range with the highest sensible load that

Figure 4:
Typical fire test arrangements for light steel loaded wall

can be applied by the test house. Temperatures are measured on the flanges and web on the 'C' sections at a number of positions, so that the critical temperatures can be related directly to the load that is applied for the particular wall build-up. This is the so-called 'load ratio' method.

With this test information, the design of a 'C' section with thicker steel or with a different wall height to that tested can be calculated using the method developed by SCI. The only issue that affects the design solution is then the effect of non-uniform heating through the 'C' section for fire on one side, which has two opposing effects: it causes some thermal bowing which adds to bending effects (or P-Δ effects); but on the beneficial side, the centre of resistance of the 'C' section moves towards the cooler unexposed flange. Although the two effects generally cancel each other for the normal range of wall lengths, both effects are taken into account in the design process.

Design Methodology for Loaded Walls

The formula that links the design resistance of a loaded 'C' section in a planar wall at the fire limit state to its buckling resistance in normal conditions is given by:

$$N_{b,Rd,fi} = k_1 N_{b,Rd} SRF(\theta_{ref})$$

$N_{b,Rd,fi}$	is the axial load that may be supported in fire.
$N_{b,Rd}$	is the buckling resistance of the 'C' section in normal conditions taking account of the effective length.
$SRF(\theta_{ref})$	is the strength reduction factor for a Class 4 cold formed steel section.
θ_{ref}	is the reference steel temperature for a non-uniformly heated section.
k_1	is a coefficient that takes account of thermal bowing effects and is typically 0.8 for walls supporting joisted floors or 0.9 for walls supporting composite (concrete) floors due to the greater restraint provided by the stiffer floor.

The procedure uses measured temperatures in a test and so it is important that this data is obtained as temperature versus time in order to be able to back-analyse the test. It is a pre-requisite that a valid test result is obtained for the particular wall build-up before the calculation method may be used.

The complete design guidance is presented in a new SCI publication P424. The guide includes numerical design examples and a wealth of construction details for walls, roofs, ceilings and junctions between elements.

External Fires on Loaded Walls

The same approach may be applied to external walls, although there are uncertainties about the severity of an external fire. At present, there is no agreement on this as logically it should be less severe than a fully developed fire within a compartment in a building. The approaches that have been proposed for an external fire are:

- A fully developed ISO fire curve, but with a cut-off temperature of 680°C as permitted by BS EN 1363-2 for external walls. With this limit, the fire endurance will be increased relative to an equivalent internal wall, but this test is rarely performed.
- A fully developed ISO fire curve, but with compliance for an external wall taken as a notional fire resistance of 60 minutes or alternatively the fire resistance period for the internal structure, reduced by 30 minutes. This is a simple way of recognising that a natural fire occurring outside a building or emanating from windows and radiating back onto the external wall has a lower effect than a fully developed fire internally, assuming adequate fire stopping around windows etc.
- A fully developed ISO fire curve without any reduction.

Figure 5: Typical fire test arrangements for a light steel loaded floor

The external sheathing boards that are used are very robust structurally but do not necessarily possess the inherent insulation characteristics of gypsum-based plasterboard. Furthermore, for buildings more than 18 m high (currently for England), non-combustible insulation and sheathing boards are required.

Composite floor slabs

Composite floor slabs can provide up to 120 minutes fire resistance without requiring a fire protected ceiling by virtue of the embedded reinforcing bars in the deck ribs. Guidance on the fire resistance of composite slabs is given in BS EN 1994-1-2 and in the former BS 5950-8, and SCI publication P375 - *Fire Resistance Design of Steel Framed Buildings*.

Design Methodology for Loaded Floors

Loaded floors differ from loaded walls in that the effects of thermal bowing do not add to the applied moments and the critical temperature is taken as the bottom flange temperature. Also, for floors, the plasterboard ceilings can become detached as they weaken in fire. The design approach for loaded floors is based on a similar approach to walls but a constant coefficient of 0.6 is used and the buckling resistance can take account of the restraint offered by the floor boarding, as follows:

$$N_{c,Rd,fi} = 0.6 N_{b,Rd}\, SRF(\theta_{exposed})$$

Most joisted floors (shown before a fire test in Figure 5) are designed for serviceability limits of deflection; their load ratio will generally be less than 0.4, meaning their performance at the fire limit state is likely to be satisfactory.

Conclusions

The new SCI publication presents detailed design guidance for light steel framing at the fire limit state. Fire tests are required, with the data used to extend the range of application to different steel thickness, size, loading and span. The publication has been circulated to SCI members; it may be downloaded from Steelbiz. ■

Impact on car park structures

Designing car park steelwork for impact has two aspects – the structural resistance and the selection of an appropriate steel sub-grade. David Brown of the SCI offers advice on both issues.

Advisory Desk 456 was prepared as a response to reports of designers circumnavigating the requirement to design internal columns for impact. Without turning to any design standard it seems entirely to be expected that accidents will happen within car parks leading to impact on any unprotected structural elements. There is plenty of evidence in car parks that vehicles can and do hit walls and barriers – regularly.

BS EN 1991-1-1 Annex B covers “Vehicle barriers and parapets for car parks” and gives a method for calculating the horizontal characteristic force on a barrier from vehicle impact. Logic demands that if a vehicle can hit a barrier, it can equally hit a column, if unprotected, so this Annex can be used to calculate the force applied to any unprotected element.

It seems that some designers are looking at the UK National Annex to BS EN 1991-1-7, and in particular at clause NA.2.16. In clause NA.2.16 the NA states that the equivalent static design force due to vehicular impact should be taken from Table 4.2 of BS EN 1991-1-7, unless the structure is Consequence Class 3. If the structure is Consequence Class 3, the National Annex directs the designer to Table NA.9.

Table 4.2 of BS EN 1991-1-7 includes traffic categories of motorways, main roads, country roads, courtyards and parking garages. It is absolutely clear that this table refers to impact on a structure from the outside – motorway velocity is not anticipated inside a multi-storey car park.

Car park structures become Consequence Class 3 if they have more than 6 storeys, when Table NA.9 applies. Table NA.9 also has classes of road including motorway, trunk roads etc – it is equally clear that this table applies to impact from outside a building. There can be no mistake – the note to the table states categorically that “these equivalent design forces are applicable outside a building; for columns inside any multi-storey building used for car parking the value must be taken from BS EN 1991-1-1 Annex B”.

It is reported that for Consequence Class 2 car park structures (those not exceeding 6 storeys), some designers suggest that they are not required to look at Table NA.9, and therefore avoid the note that internal columns should be designed for the forces in BS EN 1991-1-1 Annex B. This thought process does leave an unanswered question as to what forces should then be used. It seems that in this situation, designers are using the forces associated with external impact, from Table 4.2 of BS EN 1991-1-7, ignoring that fact that it is not applicable for impact internally and ignoring the inconvenient logic that if a vehicle can hit a barrier, it can also hit an unprotected internal column. The attraction of this thought process is that Table 4.2 of BS EN 1991-1-7 specifies a mere 75 kN for columns in “courtyards and parking garages”, in contrast to the higher forces determined from Annex B of BS EN 1991-1-1.

Annex B of BS EN 1991-1-1 should be used to determine the impact forces on unprotected elements within a multi-storey car park, whatever their Consequence Class.

Impact force according to Annex B of BS EN 1991-1-1

The characteristic impact force F is given by:

$F = 0.5mv^2 / (\delta_c + \delta_b)$

where:

- m is the mass of the vehicle in kg, taken as 1500 kg for vehicles with a gross mass not exceeding 2500 kg
- v is the velocity of the vehicle in m/s, taken to be 4.5 m/s
- δ_c is the deformation of the vehicle, taken to be 100 mm
- δ_b is the deformation of the barrier (or in this case, the column)

Clearly δ_b is a function of the member stiffness and the applied load, so some iteration is needed to find the force F.

Considering a column supporting four storeys above, the ultimate axial load $N_{b,Ed}$ is approximately 3850 kN. This value is based on a column grid of 7.2 m × 15.6 m, a variable action of 2.5 kN/m² and a permanent action of 3.56 kN/m². If the storey height is 3.5 m, a 305 UC 118 in S355 would be appropriate, with $N_{R,Ed} \approx 4115$ kN.

As a first guess, the deformation δ_b has been taken as 5 mm.

Then $F = 0.5 \times 1500 \times 4.5^2 / (100 + 5) = 145$ kN

Annex B specifies that the load is applied at 375 mm above floor level, so the column has been analysed as shown in Figure 1, with the load applied 825 mm from the node – based on half a 600 mm deep beam and a 150 mm slab, plus 375 mm. The ends of the analysis member have been modelled as fixed, which is considered to be a reasonable assumption for continuous columns and the sudden application of the load.

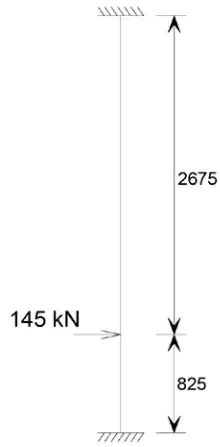

Figure 1: Analysis model

To make life easy, the member was modelled with a node at the point of load application, so the deflection could be extracted readily from the analysis results. Two cases must be considered – load applied to either axis.

In the major axis, under a load of 145 kN, the deflection at the point of load application is 0.21 mm.

The force F_{major} is revised to $0.5 \times 1500 \times 4.5^2 / (100 + 0.21) = 152$ kN.

Under this load, the deflection does not change significantly, so the characteristic force F is taken as 152 kN.

In the minor axis, under a load of 145 kN, the deflection at the point of load application is 0.64 mm

The force F_{minor} is revised to $0.5 \times 1500 \times 4.5^2 / (100 + 0.64) = 151$ kN.

Under this load, the deflection does not change significantly, so the characteristic force is taken as 151 kN

The bending moment due to this load is 73.2 kNm at the adjacent support (72.8 kNm in the minor axis)

Column verification

The impact on the column is an accidental situation (despite the frequency one sees damage in a car park) and therefore the column is verified under a combination of actions according to equation 6.11b of BS EN 1990.

The characteristic axial load from the permanent actions is 1600 kN, and from the variable actions is 1120 kN, leading to a total of 2720 kN.

From the UK NA to BS EN 1990, Table NA.A1.1 gives ψ_1 as 0.7

Thus the design combination axial load is 1600 + 0.7 × 1120 = 2384 kN

The accidental action is unfactored in equation 6.11b, so the design bending moment is 73.2 kNm in the major axis.

The column should be verified in combined bending and axial compression, using expressions 6.61 and 6.62 of BS EN 1993-1-1. This involves laborious determination of the interaction factors if proceeding with manual calculations, so to use a spreadsheet or other software would be a wise decision at this point.

Thankfully, there is a convenient software for combined axial compression and bending available on *steelconstruction.info*. Entering the input parameters, the results are shown in Figure 2 for the major axis.

The complication of calculating the C_1 factor was avoided by setting the moment at both ends to be 73.2 kNm. A uniform moment is the most onerous, so

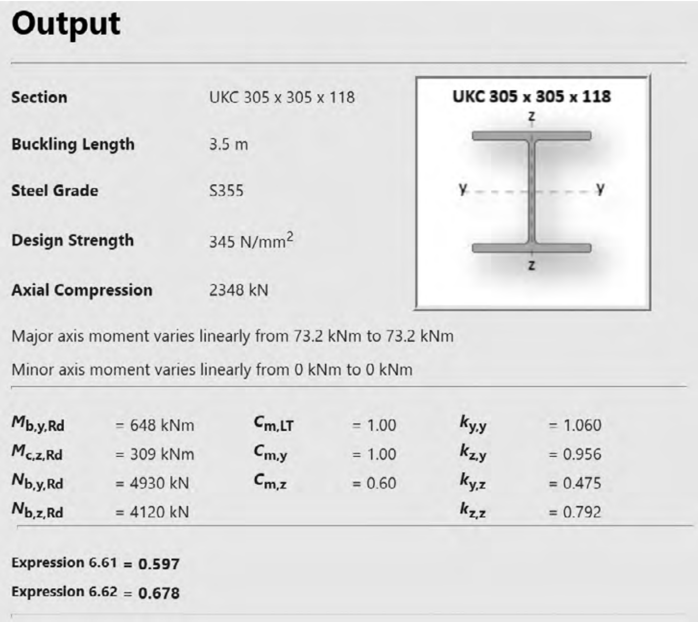

Figure 2: Column verification – major axis moment

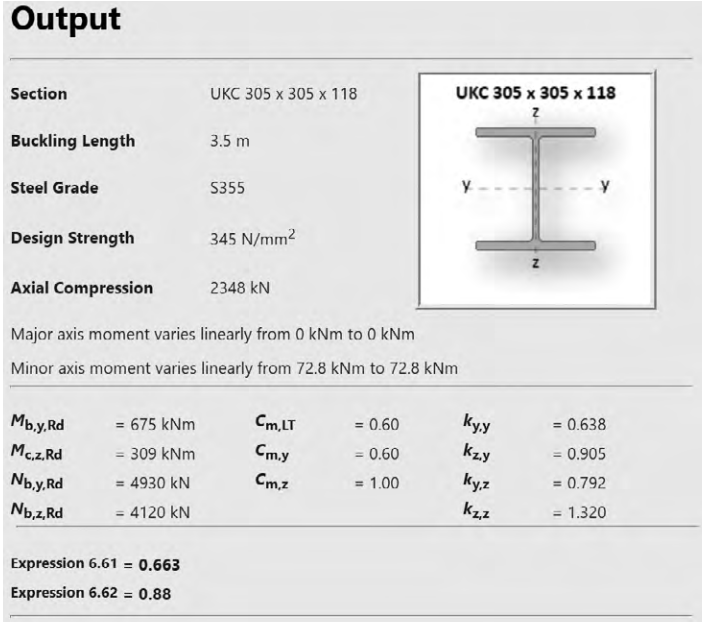

Figure 3: Column verification – minor axis moment

the approach is conservative. With both resulting utilization factors less than 1.0, the column is satisfactory.

The results for the minor axis are shown in Figure 3.

Impact and steel sub-grade

When specifying a steel sub-grade, designers will refer to BS EN 1993-1-10 and the UK National Annex. Hopefully, they will use PD 6695-1-10 as a much easier approach if fatigue is a design consideration, and SCI publication P419 if fatigue is not a concern. However, these resources only allow for a modest strain rate covering most transient and persistent design situations. In clause 2.3.1(2), BS EN 1993-1-10 notes that for other strain rates (e.g. impact loads), the tabulated values must be modified.

The first challenge is to determine the strain rate, so the equations of motion once learned in physics lessons finally have some use. The mass is known, and the force, so the acceleration can be determined from $F = m.a$

Knowing the acceleration, the initial and final velocities, the time can be determined from $v = u + a.t$

With some trivial rearrangement, $t = \frac{um}{F} = \frac{4.5 \times 1500}{152 \times 1000} = 0.044$ seconds.

The maximum stress due to the impact is when the force is applied in the minor axis.

The stress is $\frac{M}{W_{z,el}} = \frac{72.8 \times 10^6}{598 \times 10^3} = 123\ \text{N/mm}^2$

The strain is therefore $\frac{123.6}{210000} = 5.89 \times 10^{-4}$

and the strain rate $= \frac{5.89 \times 10^{-4}}{0.044} = 0.0134/\text{sec}$

Due to this high strain rate, the reference temperature in Table 2.1 of BS EN 1993-1-10 must be reduced by $\Delta T_{\dot{\varepsilon}}$ given by:

$$\Delta T_{\dot{\varepsilon}} = -\frac{1440 - f_y(t)}{550} \times \left(\ln\frac{\dot{\varepsilon}}{\dot{\varepsilon}_0}\right)^{1.5} = -\frac{1440 - 345}{350} \times \left(\ln\frac{0.0134}{1 \times 10^{-4}}\right)^{1.5} = -21.6°$$

It should be noted that in the above expression, $\dot{\varepsilon}_0$ has been taken as 1×10^{-4}/sec. This is not at all clear in clause 2.3.1 of BS EN 1993-1-10 where the value of $\dot{\varepsilon}_0 = 4 \times 10^{-4}$/sec appears. Designers would be forgiven for using this latter value, but this is the value allowed for in the tabulated values, not the value to be used to calculate $\Delta T_{\dot{\varepsilon}}$. Although not given in the code, the use of $\dot{\varepsilon}_0 = 1 \times 10^{-4}$/sec is explained in Reference 1, and confirmed in the draft prEN 1993-1-10.

The steel sub-grade may now be determined. Because of the temperature shift, immediate use of the final tables in either PD 6695-1-10 or P419 is not possible. The following example demonstrates the application of the UK National Annex provisions, assuming fatigue is not a design consideration, and therefore using information from P419. The NA references are to the UK NA to BS EN 1993-1-10.

Firstly, NA.2.1.2.2 specifies that in the UK, the use of Table 2.1 in the Eurocode is limited – only the section for $\sigma_{Ed} = 0.75 f_y(t)$ may be used. P419 presents data for an extended range of reference temperatures in Table 4.1 for $\sigma_{Ed} = 0.75 f_y(t)$, which will be used in this example.

It is assumed that the steelwork is external, so the service temperature T_{md} is -15°C.

It is assumed that the steel is welded generally, with no particular onerous details. From NA.2.1.1.2, $\Delta T_{RD} = 0°C$

It is assumed that there are no stress concentrations, so from NA.2.1.1.3, $\Delta T_{Rg} = 0°C$

If the steel is JR sub-grade, the test temperature is room temperature, 20°C. According to table NA.3, the difference between the test temperature and the service temperature is 20 – (-15) = 35°C and the value of $\Delta T_{RT} = -30°C$

Under the previous calculated axial load of 2348 kN and the moment of 72.8 kNm, the cross section is all in compression, so according to Table NA.5, $\Delta T_{R\sigma} = 30°C$

Allowing for the adjustment $\Delta T_{\dot{\varepsilon}} = -21.6°C$, the reference temperature becomes:

$T_{Ed} = -15 - 21.6 - 30 + 30 = -36.6°C$

An extract of Table 4.1 from P419 is shown in Figure 4.

Steel Grade	Sub-Grade	Charpy energy CVN at T (°C)	Charpy energy CVN J_{min}	Reference temperature T_{Ed} (°C), $\sigma_{Ed} = 0.75 f_y(t)$: 70	60	50	40	30	20	10	0	–10	–20	–30	–40	–50
	JR	20	27	200	200	200	200	200	200	200	177	114	77	54	40	30
	J0	0	27	200	200	200	200	200	200	200	200	200	177	114	77	54
S355	J2	–20	27	200	200	200	200	200	200	200	200	200	200	200	177	114

Figure 4: Extract from Table 4.1, P419

From Figure 4, the limiting thickness even at -40°C is 40 mm, compared to the actual flange thickness of 18.7 mm, so JR is satisfactory.

A design case could be considered where the vehicle strikes the column in an otherwise empty car park. The axial load is then reduced to 1600 kN. Under this load, the compression is 106 N/mm², so there is a net tension of 17.6 N/mm² on the extreme fibres.

Therefore, 17.6/345=0.05 and from Table NA.5, $\Delta T_{\dot{\varepsilon}} = 20°C$.

$T_{Ed} = -15 - 21.6 - 30 + 20 = -46.6°C$

At -50°C, the limiting thickness is 30 mm, still more than the flange of 18.7 mm and JR remains satisfactory. ■

1 Sedlacek, G, et al
Commentary and worked examples to EN 1993-1-10 "material toughness and through thickness properties" and other toughness oriented rules in EN 1993
Joint Research Centre, 2008

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Proposed revisions to lateral torsional buckling

Several of the revised Eurocode Parts are close to their final version – they are in a mature state and significant changes now would be a surprise. David Brown of the SCI commences a series of intermittent articles considering the proposed rules – starting with LTB.

Back in January 2005 New Steel Construction published an article with the enigmatic title *The Eurocodes are coming... but does the steel know?*[1] , accompanied by picture of the rather youthful author. The article introduced some of the significant changes from BS 5950 to the imminent introduction of the Eurocodes. Some 16 years later, revised Eurocodes are in preparation, more mature, like the author. The revised Eurocodes are still some way off. The core standards need to be finalised and made available, so that the work on revised National Annexes can proceed. There will also need to be decisions made about releasing the revised Eurocodes and their National Annexes in one lot, or in batches, and how the transfer from the current documents to the revised versions is to be implemented. The revision of the supporting publications, software and design tools will be a significant task to be undertaken in parallel.

For some Parts, the documents are quite mature. This includes EN 1993-1-1 and EN 1993-1-8, which are obviously important to steel designers. Between now and the date the Eurocodes become available for use it is anticipated that there will be a series of technical articles introducing the provisions of the revised standards – starting with this article on lateral torsional buckling.

Current provisions

In summary, there are currently two options to calculate the reduction factor χ_{LT} – either the general case covered in 6.3.2.2 or the special case for rolled sections in 6.3.2.3. The special case for rolled sections gives an increased resistance and is the basis for the resistances quoted in the Blue Book. A comparison between the two expressions is given in Table 1.

The imperfection factor, α_{LT} is a constant, depending on which buckling curve is appropriate. Within the special case for rolled sections, the calculation of $\chi_{LT,mod}$ is optional (it is conservative to neglect it) and completed as a subsequent modification to the calculated reduction factor.

Table 1: Current lateral torsional buckling rules

General case 6.3.2.2	Special case for rolled sections 6.3.2.3
$\chi_{LT} = \dfrac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \bar{\lambda}_{LT}^2}}$	$\chi_{LT} = \dfrac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \beta\bar{\lambda}_{LT}^2}}$
$\phi_{LT} = 0.5[1 + \alpha_{LT}(\bar{\lambda}_{LT} - 0.2) + \bar{\lambda}_{LT}^2]$	$\phi_{LT} = 0.5[1 + \alpha_{LT}(\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta\bar{\lambda}_{LT}^2]$
	For rolled sections, according to the UK NA: $\bar{\lambda}_{LT,0} = 0.4$ $\beta = 0.75$
	$\chi_{LT,mod} = \dfrac{\chi_{LT}}{f}$ where: $f = 1 - 0.5(1 - k_c)[1 - 2(\bar{\lambda}_{LT} - 0.8)^2]$

Revised provisions

The revised Eurocode has two options – a general case in 8.3.2.3(2) and a special case for doubly symmetric I and H sections in 8.3.2.3(3). The comprehensive renumbering of clauses is common to the revised Eurocode Parts – an additional complication to master when the revised standards are issued for use.

The general case is no different to that in the current standard, although presented in a slightly different way. Designers will have noticed that the current general case (Table 1) is precisely the same as the expression for flexural buckling, but with subscripts "LT" throughout. The revised Eurocode simply instructs designers to use the given expression for flexural buckling and make the appropriate substitutions.

The new LTB expressions

In contrast to the general case, there is considerable change in the expressions for doubly symmetric I and H sections (equivalent to the special case in Table 1).

In summary:

- The imperfection factor α_{LT} becomes a variable unique to the individual section, not a constant;
- The effects covered by the existing f factor (see Table 1) are addressed *within* the expressions – the modification is no longer optional;
- The non-dimensional slenderness in the minor axis, $\bar{\lambda}_z$ must be calculated.

The core expressions are:

$$\chi_{LT} = \frac{f_m}{\phi_{LT} + \sqrt{\phi_{LT}^2 - f_m \bar{\lambda}_{LT}^2}}$$

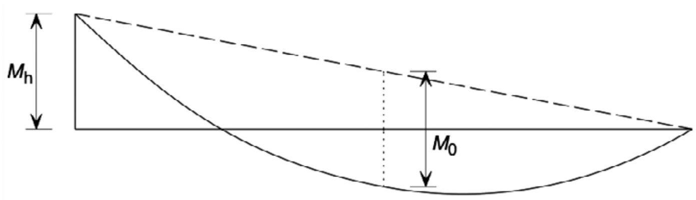
and

$$\phi_{LT} = 0.5\left[1 = f_m\left(\left(\frac{\bar{\lambda}_{LT}}{\bar{\lambda}_z}\right)^2 \alpha_{LT}(\bar{\lambda}_z - 0.2) + \bar{\lambda}_{LT}^2\right)\right]$$

Factor f_M

Factor f_M may conservatively be taken as 1.0, or from Table 8.6 in the standard. This table includes values of f_M for some orthodox shapes of bending moment diagram. The revised standard notes that f_M may be taken as 1.0 "in cases that cannot be approximated by the diagrams in Table 8.6", which seems an immediate opportunity for differences of view on what "approximately the same" means in practice. No general expression for f_M is given, despite requests from the UK (and no doubt others) at the comment stages of the draft development.

The conservative assumption that f_M = 1.0 might turn out to be attractive to many designers, as the calculation of f_M can be painful depending on the shape of the bending moment diagram. The entire Table 8.6 is too large to reproduce here, but a typical example is given in Figure 2 – a UDL on a member, with fixity at one end.



For $0 \le \dfrac{M_0}{M_h} < 1.47$: $f_M = 1.25 + 0.5\left(\dfrac{M_0}{M_h}\right)^2 - 0.275\left(\dfrac{M_0}{M_h}\right)^4$

For $\dfrac{M_0}{M_h} \ge 1.47$: $f_M = 1.05$

Figure 2: Example bending moment diagram and expressions for f_M

M_0 is the mid-span value of the free bending moment diagram (not the value of the bending moment at that point). In this instance, the value of M_0 is equal to the hogging moment M_h.

$$\frac{M_0}{M_h} = 1.0 \text{ so } f_m = 1.25 + 0.5\left(\frac{M_0}{M_h}\right)^2 - 0.275\left(\frac{M_0}{M_h}\right)^4 = 1.25 + 0.5 \times 1^2 - 0.275 \times 1^4 = 1.5$$

For a uniform bending moment diagram, $f_M = 1.0$

For a UDL on a member with pinned ends, $f_M = 1.05$

For a central point load on a member with pinned ends, $f_M = 1.10$

The values of f_M can be plotted against the C_1 factor, which also depends on the shape of the bending moment diagram, and is readily calculated, either by formulae or by software. Figure 3 shows the relationship plotted for some known points, indicating an irregular relationship between the two values. The inconsistency in the curve appears to be around the UDL values of C_1 = 1.13 and f_M = 1.05

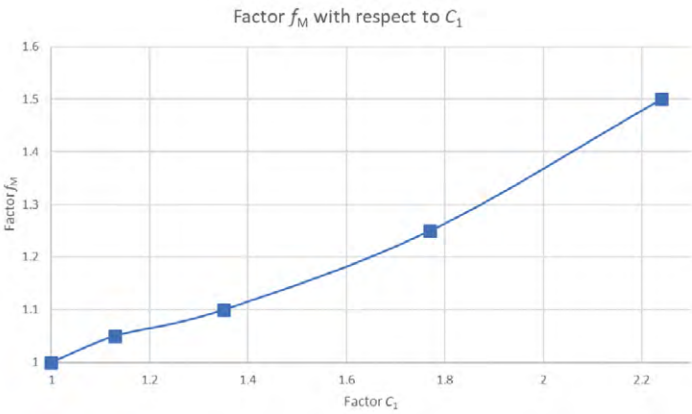


Figure 3: Relationship between f_M and C_1 at certain known points

Imperfection factor α_{LT}

The imperfection factor is given in Table 8.5. As an example, for a UB where h/b > 1.2 and $t_f \le 40$ mm, imperfection factor α_{LT} is given by:

$$\alpha_{LT} = 0.12\sqrt{\frac{W_{el,y}}{W_{el,z}}} \text{ but } \alpha_{LT} \le 0.34$$

For a 533 × 210 × 82 UB, $\alpha_{LT} = 0.12\sqrt{\frac{1800}{192}} = 0.367$, so the maximum of 0.34 applies.

For a relatively squat 305 × 165 × 40, h/b = 1.84 and α

$$\alpha_{LT} = 0.12\sqrt{\frac{560}{92.6}} = 0.295$$

A smaller value of α_{LT} results in an increased resistance, so the use of α_{LT} = 0.295 is an advantage.

Non-dimensional slenderness for minor axis flexural buckling

This is a new input into the formula for ϕ_{LT}, but a familiar term. The value can be computed from either:

$$\bar{\lambda}_z = \sqrt{\frac{Af_y}{N_{cr}}} \quad \text{or}$$

$$\bar{\lambda}_z = \frac{L_{cr}}{i}\frac{1}{\lambda_1}$$

Numerical comparisons

Table 2 contrasts the current approach with that in the revised standard, for a 533 × 210 × 82 UB in S355, subject to a UDL. The beam is 6 m long, with pinned ends.

Table 3 contrasts the current approach with that in the revised standard, for a 305 × 165 × 40 UB in S355, subject to a UDL. The beam is 6 m long, with pinned ends.

Table 2: Lateral torsional buckling resistance: 533 × 210 × 82 UB

Current standard	Proposed revisions
M_{cr} = 419 kNm	M_{cr} = 419 kNm
$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}} = \sqrt{\frac{2060 \times 10^3 \times 355}{419 \times 10^6}}$ $\bar{\lambda}_{LT} = 1.32$	$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}} = \sqrt{\frac{2060 \times 10^3 \times 355}{419 \times 10^6}}$ $\bar{\lambda}_{LT} = 1.32$
UK NA Table NA.1: Curve "c" α_{LT}=0.49	$\alpha_{LT} = 0.12\sqrt{\frac{1800}{192}} = 0.367$ but max 0.34 α_{LT}=0.34
$\phi_{LT} = 0.5[1 + 0.49(1.32 - 0.4) + 0.75 \times 1.32^2]$ $\phi_{LT} = 1.38$	$N_{cr,z} = \frac{\pi^2 EI_z}{L^2} = \frac{\pi^2 \times 210000 \times 2010 \times 10^4}{6000^2 \times 10^3}$ $N_{cr,z}$ = 1157 kN $\bar{\lambda}_z = \sqrt{\frac{10500 \times 355}{1157 \times 10^3}} = 1.80$ $f_M = 1.05$ $\phi_{LT} = 0.5\left[1 + 1.05\left(\left(\frac{1.32}{1.80}\right)^2 \times 0.34\ (1.80 - 0.2) + 1.32^2\right)\right]$ $\phi_{LT} = 1.57$
$\chi_{LT} = \frac{1}{1.38 + \sqrt{1.38^2 - 0.75 \times 1.32^2}} = 0.464$	$\chi_{LT} = \frac{1.05}{1.57 + \sqrt{1.57^2 - 1.05 \times 1.32^2}} = 0.444$
With a UDL, k_c = 0.94	
$f = 1 - 0.5(1 - 0.94)[1 - 2(1.32 - 0.8)^2]$ $f = 0.986$	
$\chi_{LT,mod} = \frac{0.464}{0.986} = 0.471$	
$M_{b,Rd} = \frac{0.471 \times 2060 \times 10^3 \times 355}{1 \times 10^6} = 344$ kNm (thankfully the same as the Blue Book!)	$M_{b,Rd} = \frac{0.444 \times 2060 \times 10^3 \times 355}{1 \times 10^6} = 325$ kNm

Table 3: Lateral torsional buckling resistance: 305 × 165 × 40 UB

Current standard	Proposed revisions
M_{cr} = 110.4 kNm	M_{cr} = 110.4 kNm
$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}} = \sqrt{\frac{623 \times 10^3 \times 355}{110.4 \times 10^6}}$ $\bar{\lambda}_{LT} = 1.42$	$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}} = \sqrt{\frac{623 \times 10^3 \times 355}{110.4 \times 10^6}}$ $\bar{\lambda}_{LT} = 1.42$
UK NA Table NA.1: Curve "b" α_{LT}=0.34	$\alpha_{LT} = 0.12\sqrt{\frac{560}{92.6}} = 0.295$
$\phi_{LT} = 0.5[1 + 0.34(1.42 - 0.4) + 0.75 \times 1.42^2]$ $\phi_{LT} = 1.43$	$N_{cr,z} = \frac{\pi^2 EI_z}{L^2} = \frac{\pi^2 \times 210000 \times 764 \times 10^4}{6000^2 \times 10^3}$ $N_{cr,z}$ = 440 kN $\bar{\lambda}_z = \sqrt{\frac{5130 \times 355}{440 \times 10^3}} = 2.03$ ϕ_{LT} = $0.5\left[1 + 1.05\left(\left(\frac{1.42}{2.03}\right)^2 \times 0.295\ (2.03 - 0.2) + 1.42^2\right)\right]$ $\phi_{LT} = 1.70$
$\chi_{LT} = \frac{1}{1.43 + \sqrt{1.43^2 - 0.75 \times 1.42^2}} = 0.463$	$\chi_{LT} = \frac{1.05}{1.70 + \sqrt{1.70^2 - 1.05 \times 1.42^2}} = 0.407$
With a UDL, $k_c = 0.94$	
$f = 1 - 0.5(1 - 0.94)[1 - 2(1.42 - 0.8)^2]$ $f = 0.99$	
$\chi_{LT,mod} = \frac{0.464}{0.99} = 0.468$	
$M_{b,Rd} = \frac{0.468 \times 623 \times 10^3 \times 355}{1 \times 10^6} = 103.5$ kNm (Blue Book has 103 kNm)	$M_{b,Rd} = \frac{0.407 \times 623 \times 10^3 \times 355}{1 \times 10^6} = 90$ kNm

Observations

Precise comparisons are difficult to make, as the situation changes with the shape of the bending moment diagram. For beams, the revised standard leads to a reduction of resistance which varies, up to approximately 14 % (the highest reduction is at longer lengths). For universal column sections used as beams, the reduction in resistance is less, but still significant.

With software, none of the additional calculation steps will really increase design effort. The only "missing link" is a general expression for f_M. Perhaps a general expression will emerge before the revised Eurocodes are released, as an alternative to curve fitting in a spreadsheet. As shown in Tables 2 and 3, manual calculations will be more involved, and it will be easier to make an arithmetical mistake. It will not be possible to have look-up tables for the reduction factor χ_{LT}, as the imperfection factor is unique to the section being considered.

The plateau length embedded in the formulae is not easy to define, since the value of 0.2 relates to $\bar{\lambda}_z$ and not $\bar{\lambda}_{LT}$. For the 533 × 210 × 82 UB, the plateau extends to a value of $\bar{\lambda}_{LT} = 0.32$ according to the formulae. For the 305 × 165 × 40, the plateau extends to a value of $\bar{\lambda}_{LT} = 0.34$. However, in a separate clause the revised standard states that no reduction in lateral torsional buckling resistance is needed if $\bar{\lambda}_{LT} \leq \bar{\lambda}_{LT,0}$ where $\bar{\lambda}_{LT,0}$ is recommended to be taken as 0.4. This is a nationally determined parameter, but if it stays as 0.4, it would mean there is a step in resistance at that point, as shown in Figure 4.

Designers should note that the Eurocodes are not yet "final". There may still be changes, and the work on the National Annex has not yet commenced. Only when this work is complete can the significant task or revising publications, design tools and software be undertaken. ■

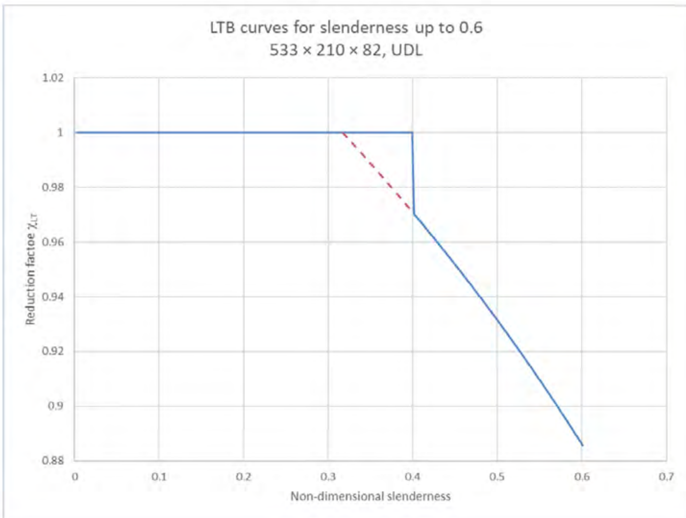


Figure 4: Reduction factor χ_{LT}

1 The Eurocodes are coming... but does the steel know?
Brown, D.G.
New Steel Construction, January 2005

Column strengthening

Re-use and adaptation of existing buildings may make column strengthening more common in the future. In this article, Richard Henderson of the SCI considers some of the issues and gives an example of a strengthening design.

1 Possible scenarios

Building refurbishment is likely to be carried out when a building is empty, particularly if the works are substantial and involve changing the internal arrangement of the structure and adding floors for plant rooms or other uses. This means that column loads during refurbishment are reduced significantly below their original design loads.

Other changes of use in multi-tenant buildings may involve the necessity for strengthening due to a change of use, but the presence of other tenants in occupation may limit the scope of work that is possible.

Columns can be strengthened by adding supplementary plates to provide additional area and enhance the other section properties, such that the strengthened section is capable of carrying the additional loads. The additional material can be welded or bolted to the original section. The form of the connection limits where the plates can be attached and affects the efficiency of the strengthening. Attachment by bolting can be done through the flanges of a UC section column, or through the web, but the most efficient strengthening arrangement for an open section element is by welding plates to the toes of the flanges to form a box section.

2 Design issues

The column supports the load present during the refurbishment and will be subject to an average compressive stress and a bending stress due to the compressive load multiplied by the amplified initial bow. Any strengthening plates provided to increase its resistance will be unloaded when they are fixed to the column. In the final state, the existing amplified bow will be further increased due to the additional load and the strengthened section will be subject to an average stress and a bending stress as already described. The original section will be subject to the sum of the stress present at the time of refurbishment and the stress due to the additional load.

3 Example

3.1 Requirement for strengthening

Consider a 305 UC 137 in grade S355 material supporting four storeys of 1092 kN each, with a system length of 4.0 m. According to the Blue Book, the compression resistance $N_{bz,Rd}$ = 4500 kN. the existing design load is 4368 kN and it is desired to add an additional load equivalent to another floor, making the final load equal to 5460 kN. The permanent load on the existing floors during refurbishment is 2.7 kN/m² and the column supports 108 m² of floor. A construction live load of 0.5 kN/m² on one floor is also present, so that the factored column load during refurbishment (and prior to adding the extra storey) is 1656 kN. (Note: a construction live load of 0.75 kN/m² is required by the loading code).

The relevant properties of the column are shown in the table.

Table 3.1 Column properties

Property	Units	Symbol	Value
height	mm	h	320.5
width	mm	b	309.2
flange thickness	mm	t_f	21.7
Area	cm²	A	174
minor axis second moment of area	cm⁴	I_z	10700
minor axis section modulus	cm³	W_z	692

3.2 Construction stage

The initial bow in the column is given by:

$$e_0 = \alpha(\bar{\lambda} - 0.2)\frac{W_z}{A}$$

where α is the imperfection factor for the relevant buckling curve and $\bar{\lambda}$ is the non-dimensional slenderness for flexural buckling. From EC3-1-1 Tables 6.1 and 6.2, buckling curve c applies and the value of α is 0.49. The elastic critical load is:

$$N_{cr} = \frac{\pi^2 EI_z}{L^2} = \frac{\pi^2 \times 210 \times 10^6 \times 1.07 \times 10^{-4}}{16} = 13861\text{kN}$$

The non-dimensional slenderness is:

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} = \sqrt{\frac{17400 \times 345 \times 10^{-3}}{13861}} = 0.658$$

Substituting values, e_0 = 8.93 mm

The maximum stress in the column during construction can be calculated using the axial load and the bending moment due to the amplified initial bow:

$$f_c = \frac{N_c}{A} + \frac{N_c e_0}{W_z}\frac{1}{\left(1 - {N_c}/{N_{cr}}\right)}$$

N_c is the design load during the construction stage.

$$f = \frac{1656 \times 10^3}{17400} + \frac{1656 \times 10^3 \times 8.93}{692 \times 10^3} \times \frac{1}{\left(1 - {1656}/{13861}\right)}$$

$$f = 95.2 + 21.4 \times 1.4 = 119.6\text{MPa}$$

This value is the maximum stress in the flange tips.

The original average design stress in the column is:

$$f = \frac{4368 \times 10^3}{17400} = 251.0\text{MPa}$$

3.3 Permanent stage

The additional load to be carried by the strengthened column is 5460 – 1656 = 3804kN. Assuming the stress in the strengthened column is 250MPa, the new area is:

$$\frac{5460 \times 10^3}{250} - 17400 = 4440\text{mm}^2$$

Consider plates welded to the flange toes to box out the section: the distance between the centrelines of the flanges is close to 300 mm. The limiting slenderness for class 3 internal compression elements is:

$$\frac{c}{t} \le 42\varepsilon = 34$$

The limiting thickness for class 3 is therefore 8.8 mm: use 10 mm plates.

The additional area is 6000 mm² and the area of the strengthened column is 234 cm² – See Figure 1.

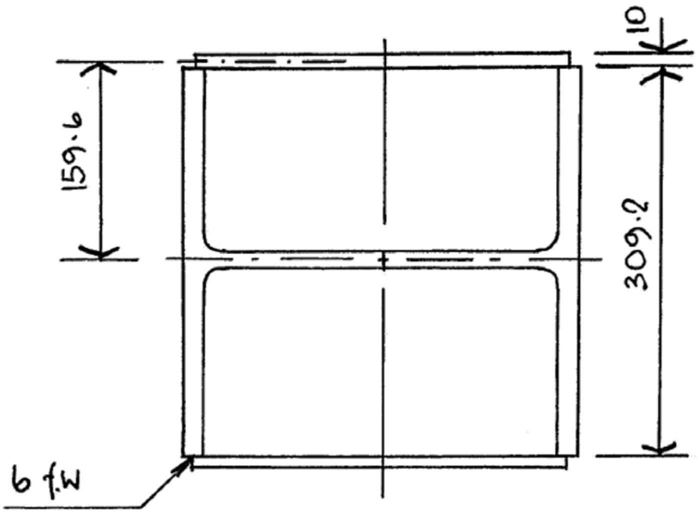


Figure 1: Cross section through strengthened column

The additional plates form a closed section with high torsional stiffness so by inspection, no check of torsional buckling is necessary.

The revised section properties of the column are:

$$I_z = 10700 \times 10^4 + 2 \times 3000 \times 159.6^2 = 2.598 \times 10^8 \text{ mm}^4$$

$$W_z = \frac{2.598 \times 10^8 \times 2}{329.2} = 1578000\text{mm}^3$$

The bow in the column at construction is the initial bow multiplied by the amplifier already calculated i.e. 8.93 × 1.14 = 10.2mm

The stress in the column due to the new load can be calculated as before, using the Euler load for the strengthened column. This is:

$$N_{cr} = \frac{\pi^2 EI_z}{L^2} = \frac{\pi^2 \times 210 \times 10^6 \times 2.598 \times 10^{-4}}{16} = 33654\text{kN}$$

The maximum stress in the column at the extreme fibre is:

$$f = \frac{3804 \times 10^3}{23400} + \frac{3804 \times 10^3 \times 10.2}{1578 \times 10^3} \times \frac{1}{\left(1 - \frac{3804}{33654}\right)}$$

$$f = 162.6 + 24.6 \times 1.13 = 190.4\text{MPa}$$

The maximum stress in the original column section is the stress at the construction stage plus the stress on the revised section from the additional load. The magnitude of this stress is approximately

$$f_{total} = 119.4 + 190.6 = 310\text{MPa} < 345\text{MPa}$$

The strengthening is satisfactory.

Based on the average axial stress, the load in the new plates is in proportion to their area:

$$\frac{A_{plts}}{A_{tot}} = \frac{6000}{23400} = 0.256$$

The load in each plate is therefore 12.8% of the additional load in the strengthened column i.e about 488 kN per plate. If the plates are site fillet welded between the top surface of the concrete slab and the underside of the beam above, connecting to the column web, the original column section in the ceiling zone must be able to carry the new load so a cross-section check is required. The cross-section resistance is 6000 kN so the column is satisfactory.

Welds are required to get the load into the strengthening plates and out again and it is appropriate to achieve this over a short length. Check the weld size required to develop the plate load over a length equal to the plate width i.e. about 300 mm. The design force per mm is therefore 488 / (2 × 300) = 0.813kN/mm. Using the Blue Book, a 5 mm fillet weld would be adequate. Using a common weld size, provide a 6 mm fillet weld over 270 mm on each side at the top and bottom of each plate. See Figure 2.

Once the force has been transferred into the strengthening plates, a connection between the plates and the column is required to transfer longitudinal shear into the plates due to the change in bending moment along the column and to prevent them from buckling under the axial load. Intermittent fillet welds could be used down the length of the column to achieve this.

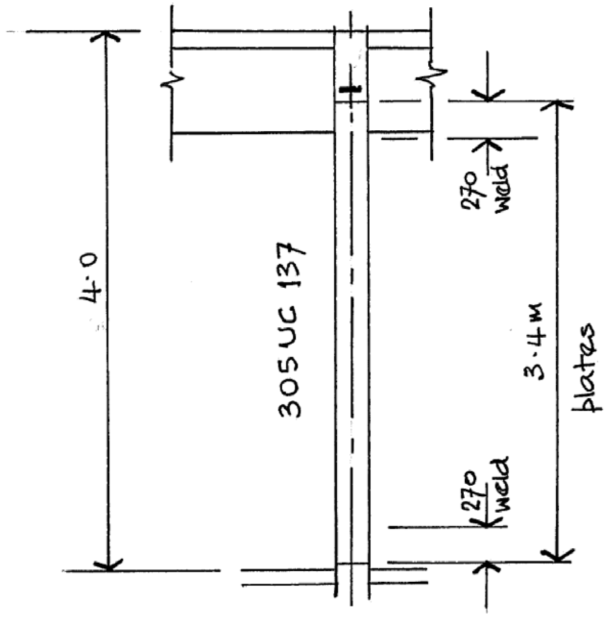


Figure 2 Elevation on strenthened column

3.4 Discussion

The arrangement of plates to strengthen the column was chosen to maximise the effect of the additional material. By boxing the column, the minor axis bending was increased significantly with plates of modest thickness. Alternatives such as adding plates to the flanges or webs would have been much less effective. For example, to achieve the same second moment of area, adding 300 mm wide plates to the flanges would have required 34 mm thick plates. Adding plates to the web would have required even thicker plates. Plating the flanges would have allowed the connection to have been achieved by bolting but this option does not seem to have a net benefit. ■

The buckling resistance of laced columns

David Brown of the SCI uses the example of a laced column to demonstrate useful approaches to member buckling.

Photo: Winvic Construction, SEGRO Logistics Park East Midlands Gateway Plot 12

Figure 1: 33 m-tall laced columns in a high bay warehouse

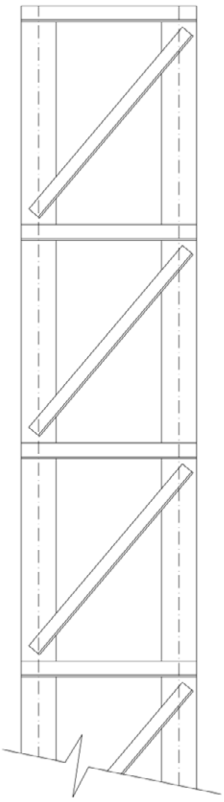


Figure 2: Laced column

Laced columns – a concept past its prime?

The short answer: mostly! A laced column consists of two main members – usually universal sections – acting as chords with a system of diagonal members in a 'N' or 'W' arrangement connecting the two chords, acting as the web of the compound section, as Figure 2. Laced columns involve significant fabrication effort, so enthusiasm for this form of column is influenced by the relative costs of labour and material. In previous decades built-up columns of this form were popular, but in the latest version of the Steel Designers' Manual they receive only a passing reference, perhaps indicating reduced enthusiasm. Laced columns *are* still used – they are very effective for high loads and tall columns found in some buildings, as illustrated in Figure 1. Laced columns may be useful when intermediate restraints are only possible to one axis of a tall, heavily-loaded column.

The primary purpose of this article is to illustrate important concepts relating to member buckling, which are demonstrated in the Eurocode rules.

Flexural buckling of laced compression members in BS 5950

All designers appreciate that buckling must be verified, usually about the two orthogonal axes of the member (a notable exception are angles). For minor axis buckling of a laced column (which is the major axis of the chord members, as shown in Figure 3), there is nothing new. The situation becomes more interesting for major axis buckling of the compound section.

BS 5950 clause 4.7.8 covers the design of laced struts and notes that the compound member may be designed as a single integral member. Table 23 allocates strut curve c when calculating the resistance of a laced strut, about either axis. In compression alone therefore, the process is straightforward. The use of a strut buckling curve allows for initial imperfections and second-order effects – notably the increase in the initial imperfection under the axial load.

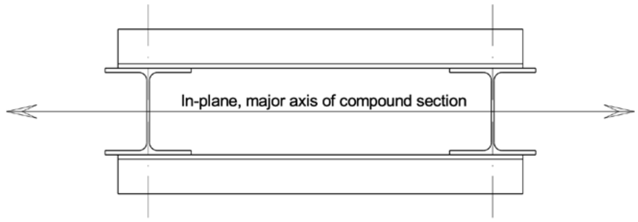


Figure 3: Laced column cross section

The challenge becomes much more complicated if the laced column is subject to an in-plane bending moment in addition to an axial compression. The expressions in section 4.8 all refer to the moment resistance of the section, being primarily suited to single rolled sections. It is not clear how to design a laced column under combined axial load and bending – designers are left to work from first principles.

The effect of shear stiffness

The shear stiffness of a laced column is significantly lower than that of a member with a solid web – much of the "web" is missing. The shear deformation of a member with a solid web is so small that it is usually ignored, but this could be significant in laced columns. Increased deformation leads to an increased moment at mid height due to the eccentricity of the cross section with respect to the ends of the member, and therefore a reduced resistance.

In BS 5950, there is no specific reference to allow for the additional deformation due to the shear flexibility. It could be that this effect is allowed for within the choice of curve c, in combination with the rules about local and overall slenderness.

Buckling of laced compression members in BS EN 1993-1-1

The Eurocode might be considered more helpful than BS 5950, since it has guidance for the design of laced columns subject to combined axial force and bending moment. The Eurocode also explicitly allows for the shear flexibility of a laced column. Perhaps of more interest is the design approach, which rather than using a buckling curve, demonstrates an alternative method to allow for imperfections and second-order effects. The following comments relate to in-plane buckling.

Usually, imperfections in members – in the form of an initial bow – and the amplification of that bow when load is applied, are dealt with through the choice of strut curve. An alternative approach is to determine the initial imperfection, amplify the imperfection and then simply complete a check of the cross section of the form:

$$\frac{\text{Force}}{\text{Area}} \times \frac{\text{Force} \times \text{final imperfection}}{\text{selection modulus}}$$

When this expression equates to the design strength of the section f_y, the buckling resistance has been established. The alternative approaches are described in clause 5.2.2.

Previous articles in New Steel Construction have reminded readers of the relationship between the initial imperfection e_0 and the final imperfection $\hat{e}$ α shown in Figure 4 which is given by:

$$\hat{e} = \frac{e_0}{\left(1 - \frac{N_{Ed}}{N_{cr}}\right)}$$

Clause 6.4 of BS EN 1993-1-8 uses this method to amplify an initial imperfection, and also to allow for the reduced shear stiffness of the laced column. Once the imperfections and second-order effects have been allowed for, all that remains are "local" checks. This approach is described in clause 5.2.2(7), where "the individual stability of members should be checked... for the effects not included in the global analysis". The effect remaining to be checked is the buckling of the chord between nodes of the lacing system.

Maximum design force in the chord

With two identical chords, the design force in one chord is obviously half the applied force – but the effects of the member initial imperfection, amplification, shear flexibility and any applied moment must all be added.

The design value of the internal moment (equivalent to Force × final imperfection above) is given by:

$$M_{\mathrm{Ed}} = \frac{N_{\mathrm{Ed}}e_0 + M_{\mathrm{Ed}}^{\mathrm{I}}}{1 - \frac{N_{\mathrm{Ed}}}{N_{\mathrm{cr}}} - \frac{N_{\mathrm{Ed}}}{S_{\mathrm{v}}}}$$

In the numerator, $N_{\mathrm{Ed}}e_0$ is the applied axial force multiplied by the *initial* eccentricity. This must be amplified, so the fundamental term becomes:

$$M_{\mathrm{Ed}} = \frac{N_{\mathrm{Ed}}e_0}{1 - \frac{N_{\mathrm{Ed}}}{N_{\mathrm{cr}}}}$$ which is the same relationship in the first part of the previous expression.

$M_{\mathrm{Ed}}^{\mathrm{I}}$ is the moment at the middle of the member (if any), without second-order effects. That too must be amplified.

In the denominator, the final term $\frac{N_{\mathrm{Ed}}}{S_{\mathrm{v}}}$ is the amplification of the deformation due to the shear stiffness of the lacings, S_{v}. Figure 6.9 of the Eurocode gives different values of S_{v} for various arrangements of lacing.

The design moment M_{Ed}, which allows for the effects described above, is converted into a force in the chord by dividing by the lever arm between the chords, to be added to half the applied compression. Expression 6.69 does not immediately appear to be so straightforward, since it is presented as:

$N_{\mathrm{ch,Ed}} = 0.5N_{\mathrm{Ed}} + \frac{M_{\mathrm{Ed}}h_o A_{\mathrm{ch}}}{2I_{\mathrm{eff}}}$, but since $I_{\mathrm{eff}} = 0.5h_0^2A_{\mathrm{ch}}$ the expression simplifies to $N_{\mathrm{ch,Ed}} = 0.5N_{\mathrm{Ed}} + \frac{M_{\mathrm{Ed}}}{h_o}$

Perhaps there was some good reason for the more complicated presentation.

How do the codes compare? – BS 5950

A convenient worked example of design to the Eurocode is contained in Reference 1. Each chord is a HE 220 A, in S355, spaced 800 mm apart. The member is 10 m long. Each chord has an area of 6430 mm². For each chord, $i_z = 55.1$ mm

The in-plane second moment of area I_{eff} of the compound section is 2058×106 mm^4

BS 5950 clause 4.7.8 places a limit on the slenderness of the chord between nodes, and on the slenderness of the overall member, so the best starting point is with a chord length between nodes.

Local buckling of chord

In Reference 1 the buckling length is 1125 mm

In the minor axis of the chord, $\lambda_c = \frac{1125}{55.1} = 20.4 < 50$, OK.

The chord flange is 11 mm, so $f_y = 355$ N/mm²

From Table 23, the strut curve is curve c (the section is a "H" profile)

From Table 24, $p_c = 345$ N/mm²

$P_c = 345 \times 6430 \times 10^{-3} = 2218$ kN per chord, 4436 kN in total.

Overall buckling of compound member

Radius of gyration in-plane = $\sqrt{\frac{2058 \times 10^6}{2 \times 6430}} = 400$ mm

Slenderness = $\frac{10000}{400} = 25$

However, the minimum value is $1.4\lambda_c = 1.4 \times 20.4 = 28.6$ (for convenience, take 30)

From Table 23, the strut curve is curve c

From Table 24, $p_c = 324$ N/mm²

$P_c = 324 \times 6430 \times 2 \times 10^{-3} = 4167$ kN

The resistance of the section in this axis is limited by the overall buckling, not the local buckling of the chords between nodes.

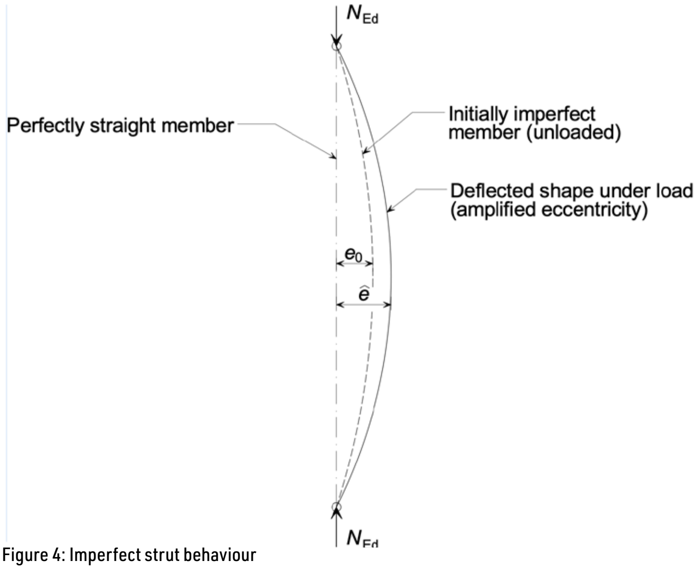


Figure 4: Imperfect strut behaviour

How do the codes compare? – BS EN 1993-1-1

From 6.4.1(1) the value of $e_0 = \frac{10000}{500} = 20$ mm

As given in the example, $N_{\mathrm{cr}} = 42650$ kN and $S_{\mathrm{v}} = 134100$ kN

Also as given, $N_{\mathrm{b,Rd}} = 2203$ kN (reassuringly similar to 2218 kN calculated above in accordance with BS 5950)

Assuming no externally applied moment, to make the example compatible,

$$M_{\mathrm{Ed}} = \frac{N_{\mathrm{Ed}} \times 0.02}{1 - \frac{N_{\mathrm{Ed}}}{42650} - \frac{N_{\mathrm{Ed}}}{134100}}$$

and $N_{\mathrm{ch,Ed}} = 0.5N_{\mathrm{Ed}} + \frac{M_{\mathrm{Ed}}}{0.8}$

The maximum resistance is when $N_{\mathrm{ch,Ed}} = N_{\mathrm{b,Rd}}$, so using "goal seek" within Excel for convenience, N_{Ed} is found to be 4167 kN

To check: $M_{\mathrm{Ed}} = \frac{4167 \times 0.02}{1 - \frac{4167}{42650} - \frac{4167}{134100}} = 95.66\mathrm{kNm}$

and $N_{\mathrm{ch,Ed}} = 0.5 \times 4167 + \frac{95.66}{0.8} = 2203$ kNm, OK.

Somewhat incredibly, the resistance according to BS EN 1993-1-1, is exactly the same as that calculated to BS 5950.

For interest, the terms in the denominator to calculate M_{Ed} are (1 - 0.098 - 0.03) which together lead to a 15% amplifier in the value of e_0. The values give some indication of the effect of shear flexibility – not very significant in this particular example. The shear flexibility obviously varies with the particular arrangement or lacing members proposed. American standards make a further distinction between lacing members that are welded or use preloaded assemblies, and those that are bolted with ordinary bolts in clearance holes. The latter introduces more flexibility into the system and reduces the overall resistance by up to 10%.

The out-of-plane buckling must be verified separately.

Conclusions

The primary purpose of this article was not to promote laced columns, but to demonstrate that buckling behaviour can be addressed either:

- within the member checks, (section 6.3 of the Eurocode) where member imperfections and second-order effects are automatically included, or
- by including the effects of imperfections and second-order effects within the analysis, leaving only local checks (usually just a cross sectional check but in the case of a laced column, the local check is still a buckling check).

A laced column is one example where the second approach is very helpful, as applied moments can be included in the design – a situation not covered in BS 5950. A second more common example is the in-plane buckling of portal frames. In-plane, imperfections and second-order effects are allowed for (if necessary) in the global analysis, meaning only a cross sectional check is needed. ■

1. Single-storey steel buildings Part 6: Detailed design of built-up columns available from arcelormittal.com

The "Blue Book" – quirks, hints and common questions

The Eurocode version of the Blue Book has been around since 2009 and exists in several different guises. David Brown of the SCI comments on the most common questions raised by users.

A long history

The Blue Book (to BS 5950) was first published in 1985 with major revisions associated with amendments to BS 5950 in 1990 and 2000. The familiar presentation of data and resistances was followed as much as possible when the Eurocode version was published in 2009. Minor amendments were made in 2015, correcting an error in the choice of curve for lateral torsion buckling of rectangular hollow sections and a corrigendum to BS EN 1993-1-1. Choosing the wrong curve for RHS was hardly significant – RHS only suffer from LTB at very long lengths if they are tall and narrow, so are almost certainly governed by deflection, not buckling. However it is important that such a widely-used resource, including online versions, is strictly correct. Occasionally, the SCI's Advisory Desk receives questions from structural engineers on values presented in the Blue Book – usually when they are preparing their own resistance calculations and they determine a different result. Often, the message will commence by advising us they have discovered an error. Hopefully, the following guidance will explain some of the commonly reported issues, point out some of the helpful – but rather hidden - features and comment on what with retrospect should have been included.

Reported mistakes

The most commonly reported "mistake" is typically the flexural buckling resistance of a member under axial compression. The "mistake" is that the Blue Book resistance is too low. The same claim is occasionally made about the quoted shear resistance.

The reason for this difference is the use of Table 3.1 of BS EN 1993-1-1 to determine the material strength of the steel. This table has the first reduction in strength when the thickness exceeds 40 mm and again at 80 mm (Figure 1).

Table 3.1: Nominal values of yield strength f_y and ultimate tensile strength f_u for hot rolled structural steel

Standard and steel grade	Nominal thickness of the element t [mm]			
	t ≤ 40 mm		40 mm < t ≤ 80 mm	
	f_y [N/mm²]	f_u [N/mm²]	f_y [N/mm²]	f_u [N/mm²]
EN 10025-2				
S 235	235	360	215	360
S 275	275	430	255	410
S 355	355	AC2 490 AC2	335	470
S 450	440	550	410	550

Figure 1: Extract from Table 3.1 from BS EN 1993-1-1

Permission to use this table is a nationally determined parameter – and the UK National Annex prohibits its use, insisting that material strengths are taken from the product standard. The product standard retains the stepped reductions at 16, 40, 63, 80 mm etc. Any section which (for example) has a flange between 16 mm and 40 mm would be credited with the higher strength according to Table 3.1 and therefore a higher resistance than allowed by the UK NA. The same issue arises with flanges between 40 mm and 80 mm.

The shear resistance calculation could suffer from exactly the same problem, but is more often linked to the material strength being set by the thickness of the thickest element in a cross section – the flange. If the web is (for example) less than 16 mm, yet the flange is more than 16 mm, the lower design strength is used for the entire cross section. Designers calculating their own resistance can easily miss this subtlety.

The next most commonly reported error is the shear resistance of M12 bolts. The usual comment is that the M12 resistance is too low, at 27.5 kN for property class 8.8 bolts in single shear, although the shear resistances for other diameters are correct. The subtlety here is that M12 bolts are *usually* used in M14 holes. According to Table 11 of BS EN 1090-2, the normal hole for an M12 should be M13, so when the bolt is used in an M14 hole, it is "oversize". Then the requirements of BS EN 1993–1–8 clause 3.6.1(5) apply, which says that M12 bolts may be used in M14 holes, but a factor of 0.85 should be applied to the calculated shear resistance.

Thus the calculated resistance for an M12, property class 8.8 bolt becomes:

$$F_{\mathrm{v,Rd}} = \frac{\alpha_{\mathrm{v}} f_{\mathrm{ub}} A}{\gamma_{\mathrm{M2}}} \times 0.85 = \frac{0.6 \times 800 \times 84.3}{1.25 \times 100} \times 0.85 = 27.5 \text{ kN as quoted}$$

The same issue would apply to M14 bolts, but these are not a common diameter and therefore not covered in the Blue Book.

Lateral torsional buckling resistances

LTB resistances appear in two locations in the Blue Book. They appear in tables for LTB resistance alone, and again in the tables for combined axial load and bending. Correspondents sometimes point out that the values are different.

In fact the values are identical – for $C_1 = 1$, which represents a uniform bending moment. Rather regrettably perhaps, the combined axial load and bending tables only have room for single lines of LTB resistances, so the most conservative values had to be chosen, which is for $C_1 = 1$. The tables for LTB alone have values for seven different values of C_1, covering standard cases such as a UDL on a pin-ended beam, and providing additional values to aid interpolation.

The situation becomes more complicated, since the combined axial load and bending tables have two rows for the LTB resistance, as shown in Figure 2. Both rows are indicated as $M_{\mathrm{b,Rd}}$.

Section Designation and Resistances (kN, kNm)	n Limit	Compression R			
					Varyin
		L (m)	1.0	1.5	2.0
* 406x178x67	0.752	$N_{b,y,Rd}$	3040	3040	3040
$N_{pl,Rd}$ = 3040		$N_{b,z,Rd}$	2890	2690	2450
$f_y W_{el,y}$ = 422	0.752	$M_{b,Rd}$	422	421	391
$f_y W_{el,z}$ = 54.3	**0.203**	$M_{b,Rd}$	478	470	433

Combined axial load and bending

Designation Cross section resistance (kNm) Classification	C_1 (1)			
		1.0	1.5	2.0
406x178x67	1.00	478	470	433
	1.13	478	478	454
$M_{c,y,Rd}$ = 478	1.35	478	478	478
$M_{c,z,Rd}$ = 84.1	1.50	478	478	478
	1.77	478	478	478
	2.00	478	478	478
Class = 1	2.50	478	478	478

LTB alone

Figure 2: Extract from the combined axial load and bending tables, and the complementary LTB tables

Section Designation	Axis	Compression resistance $N_{b,y,Rd}$, $N_{b,z,Rd}$, $N_{b,T,Rd}$ (kN) for Buckling lengths (m)												
		1.0	1.5	2.0	2.5	3.0	3.5	4.0	5.0	6.0	7.0	8.0	9.0	10.0
* 406x178x67	$N_{b,y,Rd}$	*2860*	*2860*	*2860*	*2860*	*2850*	*2820*	*2800*	*2750*	*2690*	*2620*	*2550*	*2460*	*2350*
	$N_{b,z,Rd}$	*2740*	*2560*	*2340*	2160	1840	1540	1280	897	654	496	388	311	255
	$N_{b,T,Rd}$	*2790*	*2650*	*2500*	*2350*	2190	2030	1890	1660	1490	1370	1280	1220	1170

Figure 3: Axial resistances from the compression (alone) tables

The lower row is for Class 1 and Class 2 sections. For these sections the plastic modulus, W_{pl} is used in the calculation. The upper row is for Class 3 sections, which use the elastic modulus W_{el} and thus a different resistance is computed. A section which was Class 1 or Class 2 can become Class 3 (and indeed Class 4) under increasing levels of axial load, so both LTB resistances had to be presented.

In contrast, all Universal Beams are Class 1 under bending alone, so the LTB tables show the resistance based on the plastic modulus.

The best advice is: If the section is Class 1 or Class 2, take the LTB resistances from the LTB tables using the appropriate value of C_1. If the section is Class 3, the LTB resistances from the combined axial load and bending tables are conservative. An improved resistance for Class 3 sections could be calculated separately if the C_1 value was much higher than 1.0.

The reasoning for the two rows only of LTB resistances in the combined tables really flows from the BS 5950 presentation. In BS 5950, the effect of a varying bending moment diagram was accounted for in the m_{LT} factor, but this was applied after calculating the resistance M_b (see clause 4.3.6.2 of BS 5950). Thus there was only a requirement to present M_b for the two classes of section in only two rows, leaving the designer to apply the m_{LT} factor.

Quick – what Class is the section?

The long-winded way to determine the section Class when a member is subject to combined axial load and bending is to classify the section using Table 5.2 of BS EN 1993-1-1, calculating the values of α and ψ , checking $^{c}/_{t}$ against the limits within the table.

The much easier way is to use the "n" limits already seen in Figure 2. The "n" limit is a proportion of the cross-sectional resistance, here given as $N_{pl,Rd}$ and indicates the axial load when a section becomes Class 2, and the axial load when a section becomes Class 3.

For the example shown in Figure 2:

0.203 × 3040 = 617 kN Thus at axial loads lower than 617 kN, the section is "at least" Class 2.

0.752 × 3040 = 2286 kN Thus for loads greater than 617 kN and less than 2286 kN, the section is Class 3. At axial loads greater than 2286 kN, the section becomes Class 4.

The reason for the "at least" in the above definition should become clear in Table 1. For Class 1 and Class 2 sections, the same section properties are used in both the calculation of the axial resistance and the bending resistance. It therefore does not matter - to know the section is "at least" Class 2 is sufficient.

The axial resistances are different?

Another subtle difference that gives rise to some questions is seen when comparing the axial resistances in Figure 2, (from the combined axial load and bending tables) with the complementary axial resistances in Figure 3, which is taken from the tables for axial load alone.

In Figure 3, the axial resistances are lower, and some are in italic font. Here the explanation is straightforward and given at the bottom of the tables. Italic font indicates the section is Class 4, and as seen in Table 1, the effective area is used in the calculation of the resistance. Inspection of the full tables shows that the values are identical when not Class 4 (not in italic font). The more onerous classification which affects the "compression alone" tables is for that precise reason – the entire cross section is in compression and the web becomes Class 4. As soon as a hint of bending is introduced, the web is not all in compression and is less enthusiastic to buckle. A more scientific explanation can be seen by appreciating that the relevant column in Table 5.2 of BS EN 1993-1-1 moves from "Part subject to bending" to "Part subject to bending and compression" and the limiting values are more generous.

Table 1. Section properties required for resistance calculations.

	Section property required	
Section Class	Axial resistance	Bending resistance
Class 1	A	W_{pl}
Class 2	A	W_{pl}
Class 3	A	W_{el}
Class 4	A_{eff}	W_{eff}

What could have been better?

A matter of regret is a small but important feature of the tables covering bolt resistances in tension. In BS 5950, two values of bolt tension resistance were defined. One value was the "full" resistance. If this value was used, described as the "Exact" capacity in the BS 5950 Blue Book, prying (which increases the force in the bolt) had to be taken into account. The second value, described as the "Nominal" capacity could be used if certain geometric limits were observed. When using the "Nominal" capacity, prying could be neglected, as only 80% of the "Exact" capacity was used.

In the Eurocode Blue Book, only the equivalent of the "Exact" value is presented, leaving designers to include prying in their calculations. Some years after the Blue Book was prepared, AD 354 was published, which offered exactly the same options as BS 5950. The current danger is that designers neglect prying, but take the published values as the "Nominal" resistances. With the benefit of hindsight, an additional column of resistances, being 80% of those currently presented, would have been very helpful. Perhaps designers might mark up their own copies! ■

Low carbon concrete – what you need to know

Graham Couchman (SCI) and Jenny Burridge (the Concrete Centre) discuss the specification of 'low carbon' concrete for use in composite construction

Globally, concrete is the second most used material after water. In the UK we produce about 109 Mt of ready mixed concrete and precast concrete products annually (2017 figures). As structural engineers we therefore specify a lot of concrete every year, even when specifying steel-framed solutions given that most steel-framed multi-storey buildings use composite floors. More of us are now trying to lower our carbon footprint and produce lower carbon intensive projects. One of the ways we can do this is to look at how we specify the concrete we use, to ensure the most appropriate material is adopted. Significant improvements are possible, so alternatives are well worth considering.

Concrete mix and embodied CO_2

Concrete is made from aggregate, cement, and water. Admixtures can be (and normally are) included in the mix. In terms of embodied carbon for the different elements, aggregates and water have very low embodied carbon. Locally sourced primary aggregates have an embodied carbon of about 4kgCO_2/tonne. It is the cement, forming about 10-15% of the mix, which holds most of the embodied carbon.

All concretes to BS 8500[1] are based on Portland cement, or CEM1, but mostly contain additions, or other cementitious materials. These include:

- Ground granulated blast-furnace slag (GGBS)
- Fly ash
- Silica fume
- Limestone powder
- Pozzalana

These additions have a much lower embodied carbon than CEM1. Significant savings can be made to the embodied carbon of concrete by specifying mixes that include additions. Table 1 gives an indication of the savings that can be achieved by specifying the different cements.

Broad designation of cement type in concrete	Percentage of addition	Embodied CO_2 kgCO_2/m³ of concrete
CEM1	0	283
IIA	6 - 20	228 - 277
IIB	21 - 35	186 - 236
IIIA	36 - 65 GGBS	120 - 198
IIIB	66 - 80 GGBS	82 - 123
IVB	36 - 65 fly ash or pozzalana	130 - 188

Table 1: Embodied CO_2 of UK concretes complying with BS 8500 (based on a cement content of 320kg/m³ of concrete)

It is also worth noting that higher strength concrete requires a larger proportion of cement, all other things being equal, although this can be more than offset if the higher strength allows a lower volume to be used. Superplasticiser admixtures can also help reduce the embodied carbon by reducing the water/cement ratio. This provides a stronger concrete for the same quantity of cement.

To supplement concretes in accordance with BS 8500, most of the larger concrete producers have low carbon proprietary concretes. These are formulated to keep the embodied carbon down to a given level, and may therefore be particularly interesting to the specifier. The producers are happy to provide advice on what can be achieved for the location and needs of the project, but it is important that the structural designer knows how to interpret the information they provide. This is discussed here.

Concrete properties that may affect structural behaviour

Although the potential benefits may be significant, care is needed when specifying alternatives to concrete covered by BS 8500. This is because concrete is specified on the basis of compressive strength alone (other than any special requirements for pouring etc). However numerous other concrete characteristics will, or could, affect the behaviour (short-term, long-term, fire) of composite construction, as considered below. The relationships between these various characteristics are only guaranteed, such that the material can be specified on the basis of compressive strength alone, for concrete mixes complying with BS 8500. It is worth noting that future versions of design software may therefore need the ability for certain properties to be inputted independently, to cover the presence of proprietary mixes on the UK market.

Mechanical behaviour

The strength and stiffness of the slab, and the mechanical interaction between steel and concrete *may*, and in some cases certainly *will*, depend on:

- Characteristic (compressive) cylinder strength f_{ck}
- Secant modulus E_{cm}
- Tensile strength f_{ctm} (which may be important for shear stud resistance as concrete 'cone' failure often governs in the presence of transverse trapezoidal decking)
- Crushing strain ε_{c1} – the upper fibres of concrete in compression must not lose strength before the steel decking reaches the anticipated level of stress

Figure 3.2 of BS EN 1992-1-1[2], reproduced here as Figure 1, shows the compressive stress-strain behaviour to be used in structural analysis, and defines some key variables.

Table 3.1 of BS EN 1992-1-1 defines certain rules that directly link various material properties. The key resulting values for a typical C30/37 (i.e. characteristic cylinder strength of 30 MPa) concrete are included in Table 2 here, for information and comparison purposes.

As noted above, these relationships mean that when a compressive strength is explicitly defined, the other properties are implicitly defined by the various formulae. Unless all those relationships are respected, or the derived properties are 'exceeded' (i.e. the value defined by the BS EN 1992-1-1 relationship is in fact conservative), for types of concrete not in compliance with BS 8500 the mechanical behaviour of the composite slab or beam could be adversely affected. Of course, performance could also be improved, depending on the concrete characteristics.

It is also worth noting that BS EN 1994-1-1[3] clause 9.8.2(4) allows explicit calculation of deflection to be ignored for composite slabs with a span to effective depth within certain limits. This relaxation inherently assumes a certain relationship between material strength and stiffness.

Long-term behaviour

The long-term behaviour of a composite floor is a function of the creep and

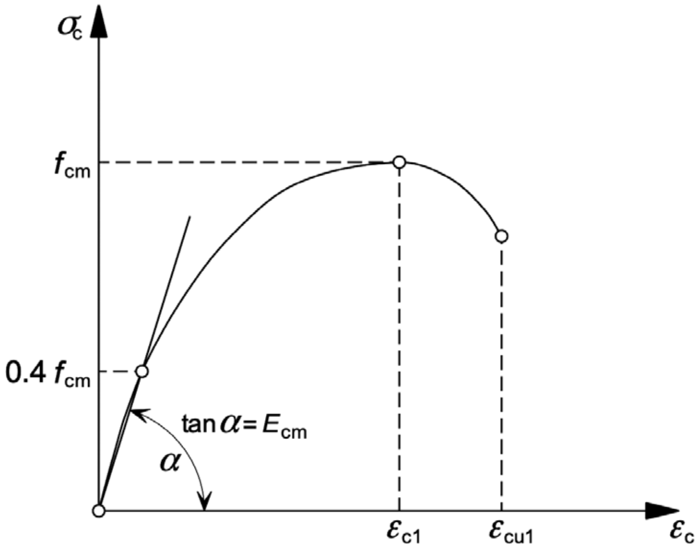


Figure 1: Schematic representation of the stress-strain relationship for structural analysis (the use of 0.4 f_{cm} for the definition of E_{cm} is approximate).

Property	Notation	Value for C30/37
Characteristic compressive cylinder strength at 28 days	f_{ck}	30 MPa
Mean value of concrete cylinder compressive strength	f_{cm}	38 MPa
Mean value of axial tensile strength	f_{ctm}	2.9 MPa
Secant modulus of elasticity	E_{cm}	33 GPa
Compressive strain in concrete at peak stress	ε_{c1}	2.2 °/₀₀
Ultimate compressive strain	ε_{cu1}	3.5 °/₀₀

Table 2: Material properties influencing mechanical behaviour for C30/37 concrete (according to BS EN 1992-1-1)

shrinkage characteristics of the concrete. Generally, these 'deteriorations' in the concrete are less significant with composite construction than reinforced concrete, because the steel elements resist the concrete strains. Relevant properties are:

- Creep coefficient ø. BS EN 1992-1-1 Fig 3.1 provides a simplified method, in the form of a number of graphs from which ø can be determined.
- Total free shrinkage strain ε_{cs} which is defined in BS EN 1994-1-1 Annex C as 325×10^{-6} for normal weight concrete in a dry environment.

BS EN 1994-1-1 clause 5.4.2.2 allows for both shrinkage and creep using a modular ratio approach for determining long-term deflections. The modular ratio increases with time as the steel modulus is unchanged, and the concrete modulus reduces (reducing the contribution of the concrete part of a composite cross-section). In the majority of cases, for buildings, a simplified approach is taken whereby the concrete properties described above are not explicitly considered (nor are inputs otherwise required, such as time of first load application and duration of loading). The modular ratio for a member under a mixture of short and long-term loading is taken as $2n_0$, where $n_0 = E_a/E_{cm}$ (i.e. the modular ratio at time zero). The validity of this assumed halving of the concrete stiffness with time should be justified, or otherwise, by considering the creep and shrinkage characteristics of any 'non-standard' concrete.

BS EN 1992-1-1 Annex B defines the relationship between modular ratio, and shrinkage and creep properties.

Fire behaviour

BS EN 1994-1-2 clause 3.2.2 Table 3.3 shows strength retention (it's actually reduction) with temperature. As an example, normal weight concrete has lost 25% of its strength at 400 degrees, and over half its strength at 600 degrees. Strain capacity also reduces with temperature. Current design software adopts these values, with no allowance for user modification. Any higher, or lower, rate of loss of strength with temperature of a given material would adversely, or beneficially, affect the mechanical resistance of a composite floor in fire.

BS EN 1994-1-2 clause 3.3.3 Fig 3.8 gives thermal conductivity values. Conductivity is important because the insulation provided by a concrete floor controls the temperature of the upper surface when the floor is exposed to fire from below. Fire tests on floors consider three failure criteria, one of which concerns the temperature achieved on the upper surface after the regulated period of fire exposure. Lower insulation would therefore invalidate a fire test result for a given slab. Clearly the existence of a relevant fire test, using the concrete material under consideration for substitution, would avoid the need for material properties to be defined.

Density

From a loading point of view, it is important to know dry density and wet density, and if stated values include an allowance for reinforcement (or are the concrete alone). Clearly the appropriate values must be used in any design software. The Eurocodes state 2600kg/m³ wet and 2500kg/m³ dry, but we reduce these by 50kg/m³ because composite slabs have less reinforcement than a typical RC slab.

Density may also affect the acoustic performance of a slab.

Rate of strength gain

An important point to note is that the higher the proportion of additions within the concrete, the slower the strength gain of the concrete. This might not influence the programme if the concrete does not need to be struck quickly or support load shortly after being cast. For composite construction, lower strength (and stiffness) gain may be less relevant than for in-situ reinforced concrete, because of the permanent formwork provided by the steel decking. However, it could impact on timing of removal of props, or application of loading. SCI publication P300 states that props should not be removed until a floor has reached 75% of its design strength, and suggests that this is normally achieved in seven or eight days. That indicative timing may no longer be valid, depending on the concrete type and the external temperature.

Conclusions

Any designer, contractor or manufacturer considering using 'non-standard' concrete (i.e. not covered by the scope of BS 8500) – whether to reduce carbon or for any other reason - should ensure that all relevant properties are known for the concrete they are considering, and justify the assumed performance of the composite construction. Doing this correctly, in consultation with all relevant parties involved in the design, material supply and construction of the project, should ensure that significant benefits are achieved without structural performance being compromised.

SCI offers a third-party assessment service whereby we will review the claimed performance characteristics of any proprietary concrete and confirm suitability for use (or advise how performance may be affected).

Acknowledgement

Content concerning the different types of concrete originally appeared in 'How to Specify Lower Carbon Concrete', authored by Jenny Burridge and published by The Institution of Structural Engineers (*https://www.istructe.org/resources/guidance/how-to-specify-lower-carbon-concrete/*). ■

1. BS 8500-1:2015 + A2:2019: Concrete – Complementary British Standard to BS EN 206. BSI, 2019
2 BS EN 1992-1-1: 2004: Eurocode 2: Design of concrete structures. General rules and rules for buildings (+A1:2014) (incorporating corrigenda January 2008, November 2010 and January 2014)
3 BS EN 1994-1-1:2004: Eurocode 4: Design of composite steel and concrete structures. General rules and rules for buildings (incorporating corrigendum April 2009)

Advisory Desk 2021

AD 455:
Design resistances for bespoke components in P358 (Green Book)

A reader has questioned the design resistances tabulated for bespoke components in P358 – the Green Book for nominally pinned joints to Eurocode 3.

The specific question related to the difference between the values quoted for Hollo-Bolts in Table G.60 and the data provided by the manufacturer.

The manufacturer provides *characteristic* resistances in their data for use with Eurocode designs. The Green Book tabulates *design* resistances, which are the characteristic resistance divided by the γ_{M2} factor, which in the UK National Annex is specified as 1.25

Typical values of the characteristic resistances provided by the manufacturer are 124 kN in tension and 211 kN in shear for an M20 Hollo-Bolt.

The design resistances in Table G.60 are 99.2 kN and 169 kN respectively, being the characteristic resistance divided by 1.25

Designers should note that the Green Book was first printed in 2014 and contains data appropriate at that time. It is quite possible that manufacturers may have subsequently changed material specification or component geometry, so checking with the manufacturer's latest data is advised.

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AD 458:
Web panel shear resistance

This AD relates to a bolted moment-resisting connection and the determination of the shear force in the column web panel, and in particular the selection of a lever arm as part of that calculation. In the following advice, the axial force in the beam is assumed to be zero, for simplicity.

In a moment resisting connection designed in accordance with BS EN 1993-1-8, the total tension, being the summation of the force in each bolt row, cannot exceed the resistance of the compression zone, or the shear resistance of the column web panel. If necessary, the forces in the bolt rows are reduced.

The moment resistance of the connection is then calculated by multiplying the resistances of the bolt rows (reduced if necessary) by their lever arms. The connection resistance is then compared to the applied moment.

It should be noted that in this process, the *resistance* of the connection is calculated.

If the resistance if the column web panel was limiting the development of tension in the bolt rows, and thus limiting the moment resistance of the connection, it may be appropriate to reinforce the column web panel. This requires knowledge of what the applied shear force is. Although this sounds straightforward, the process described above determines the resistances, not the applied forces.

In many cases, especially if the resistance of the connection is not greatly in excess of the applied moment, it would not be too conservative to assume the applied shear force to be equal to the summation of the bolt row resistances.

If more effort is worthwhile, BS EN 1993-1-8 clause 5.3(3) specifies that the applied shear force is given by the applied moment, divided by a lever arm, z. Note that this correctly relates to the applied moment, not the moment *resistance*.

The lever arm, z, is determined via a forward reference to clause 6.2.7, which in turn refers to Figure 6.15. The bottom row in Table 6.15 covers bolted connections with two or more bolt rows in tension – the common case. Two alternatives are given for the lever arm z:

1. The distance from the centre of the compression flange to mid-way between the two furthest bolt rows, and,
2. A "more accurate" value, taken as z_{eq} from the method described in clause 6.3.3.1

Clause 6.3.3.1 covers the calculation of joint stiffness; the calculation of z_{eq} is part of the process. It is very likely that the "more accurate" value z_{eq} is smaller than the dimension described in (1) above, and thus would produce a higher shear force.

If designers are calculating the shear force in the web panel based on clause 5.3(3) they should be careful to use the "more accurate" value, as use of the approximate value is not always conservative. Calculation of the "more accurate" value is not without its own challenges, so designers should remember that assuming the applied shear force to be equal to the summation of the bolt row *resistances* will be conservative, and probably economical in terms of design effort.

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AD 428A:
Lateral and torsional vibration of half-through truss footbridges

Revision Note

This AD note was first issued to provide interim guidance on the design of half-through footbridges. It has now been revised following publication of updated standards and is issued as AD428A.

Purpose of this guidance

This note alerts designers to the potential susceptibility of narrow half-through footbridges to excitation by pedestrians in a lateral-torsional mode. Until the recent publication of NA+A1:2020 to BS EN 1991-2:2003 Incorporating Corrigendum No. 1, Eurocodes and UK National Annexes did not fully address this mode of vibration, so there was a danger that it may have been discounted without proper consideration. This previous gap in the standards has led to the need to retrofit dampers and/or provide additional stiffening to some recently constructed footbridges where excitation occurred due to pedestrians walking eccentric to the deck centreline and, more significantly, from deliberate shaking of the deck.

Affected mode of vibration

Half-through footbridges, without plan bracing to the top chord, often have as their lowest natural mode of vibration a lateral-torsional mode. A typical example is shown in Figure 1. The mode occurs because the open bridge cross-section has a low torsional stiffness with a shear centre below the deck level about which axis the rotation occurs.

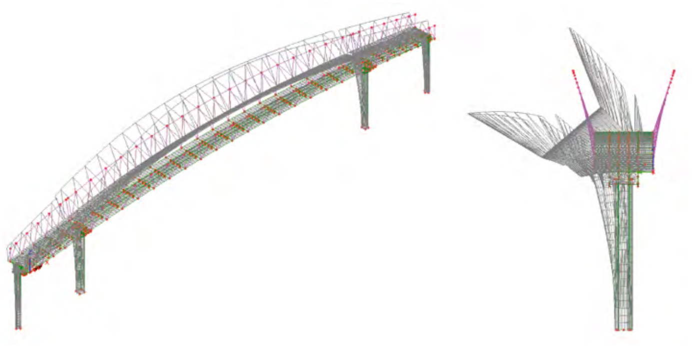

Figure 1: Lateral and torsional mode of vibration

UK design criteria prior to issue of "NA+A1:2020 to BS EN 1991-2:2003 Incorporating Corrigendum No. 1"

The criteria for assessing the dynamic behaviour of footbridges were outlined in the following Eurocodes (BS EN) and BSI Published Documents (PD):

- BS EN 1990:2002+A1:2005 as modified by UK National Annex
- BS EN 1991-2:2003 as modified by UK National Annex
- PD 6688-2: 2011

They contained the following requirements:

- Eurocode EN 1990 clause A2.4.3.2(2) requires comfort to be verified if the natural frequency is lower than 2.5 Hz for lateral and torsional modes;
- BS EN 1990 clause A2.4.3.2(1) states that comfort criteria should be defined in terms of maximum acceptable acceleration and proposes a horizontal limit for lateral and torsional vibrations of 0.2 ms^{-2} under normal use and 0.4 ms^{-2} for exceptional conditions, but makes these values nationally determined parameters;
- Clause NA.2.3.10 of the UK National Annex to BS EN 1990 states that the pedestrian comfort criteria should be as given in NA.2.44 of the UK National Annex to BS EN 1991-2. However, this clause does not specify a maximum acceptable acceleration for horizontal movement under normal use – it (and PD 6688-2) only address synchronous lateral vibration caused by lateral forces from footfall and does not address lateral and torsional modes excited by vertical loading.

None of the documents provided limiting horizontal accelerations for deliberate lateral shaking of the bridge.

A literal reading of all the applicable clauses therefore led to the conclusion that a lateral-torsional mode with frequency less than 2.5 Hz should be verified for horizontal acceleration as EN 1990 clause A2.4.3.2 (2) still applies. However, no acceleration limit was provided as EN 1990 clause A2.4.3.2(1) was modified by the UK NA to BS EN 1991-2 which, itself, did not provide a limit.

Updated provisions in NA+A1:2020 to BS EN 1991-2:2003 Incorporating Corrigendum No. 1

The following requirements have been made in NA+A1:2020 to BS EN 1991-2:2003 Incorporating Corrigendum No. 1 to address the original problems noted above:

i. The design should conform to the requirements of BS EN 1990 clause A2.4.3.2(2) i.e. a verification of the comfort criteria should be performed if the fundamental frequency of the deck is less than 5 Hz for vertical vibrations, and 2.5 Hz for horizontal (lateral) and torsional vibrations.
ii. The maximum acceptable acceleration for horizontal movement under normal use should be taken as the recommended value given in BS EN 1990 clause A2.4.3.2(1) [i.e. 0.2 ms^{-2}], measured at the level of the deck. The acceleration should be calculated under the vertical load models of NA.2.44 considering walking paths offset from the bridge centreline as necessary.
iii. Where the fundamental frequency of the bridge is less than 3 Hz for horizontal (lateral) and torsional vibrations, consideration should be given to making provision in the design, in discussion with the client, for possible installation of dampers to the bridge after its completion. [This recommendation makes some allowance for uncertainty in the value of damping and other parameters used in the calculations and also provides some potential remedy for unacceptable horizontal accelerations from deliberate shaking should they occur].
iv. Any further limiting criteria for pedestrian comfort, such as under deliberate shaking, should be determined on a project-by-project basis and agreed with the client.
v. The potential for unstable lateral responses (synchronous lateral vibration) should still also be checked using NA.2.44.7 of the UK National Annex to BS EN 1991-2.

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AD 460: Amendment A2 to EN 1993-1-4

In February 2021, the second amendment to the 2006 version of the stainless steel Eurocode was published, EN 1993-1-4:2006+A2:2020.

The amendment consists of one revision – Table 5.3, which lists the imperfection coefficient (α) and the plateau length ($\bar{\lambda}_0$) that are used in the buckling curves for flexural, torsional and torsional-flexural buckling.

The revised table in this new amendment is presented as Table 1 (over page).

The reason for the amendment is that since the original buckling curves were developed (more than 25 years ago), a considerable amount of additional experimental data on stainless steel compression members has been generated. The test data, supplemented by extensive numerical analyses

Buckling mode	Type of member	Axis of buckling	Austenitic & austenitic-ferritic (Duplex)		Ferritic	
			α	$\bar{\lambda}_0$	α	$\bar{\lambda}_0$
Flexural	Cold formed angles and channels	Any	0.76	0.2	0.76	0.2
	Cold formed lipped channels	Any	0.49	0.2	0.49	0.2
	Cold formed rectangular hollow sections	Any	0.49	0.3	0.49	0.2
	Cold formed circular hollow sections	Any	0.49	0.2	0.49	0.2
	Hot finished rectangular hollow sections	Any	0.49	0.2	0.34	0.2
	Hot finished circular hollow sections	Any	0.49	0.2	0.34	0.2
	Hot rolled sections & welded open or box sections	Major	0.49	0.2	0.49	0.2
		Minor	0.76	0.2	0.76	0.2
Torsional and torsional-flexural	All members	The values of α and $\bar{\lambda}_0$ for minor axis flexural buckling apply.				

Table 1 Values of α and $\bar{\lambda}_0$ for flexural, torsional and torsional-flexural buckling (Table 5.3 of EN 1993-1-4:2006+A2:2020)

and reliability assessments, indicate that for cold formed hollow and open sections, the existing buckling curves were too optimistic. Assessment of the data also showed that in some cases, a different buckling curve was justified for ferritic stainless steel due to its less non-linear stress-strain characteristics. Additionally, it was possible to make a distinction between rectangular and circular hollow sections, as well as lipped channels and plain channels. New data on hot finished hollow sections also enabled the addition of buckling curves for this product form. Welded box sections were conservatively assigned the same buckling curve as hot rolled and welded open sections.

Further information on the new flexural buckling curves is given in the Commentary to the *Design Manual for Structural Stainless Steel*, available from *www.steel-stainless.org/designmanual*

For torsional and torsional-flexural buckling, new test data has also suggested that the original buckling curve is too optimistic for channel section columns. In the absence of data for other open cross-sections susceptible to torsional or torsional-flexural buckling, the minor axis flexural buckling curve is recommended as a safe approximation. The same approach is used in prEN 1993-1-1:2020.

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AD 461: Anchorage of bars in the troughs of composite slabs

Introduction

Traditional practice in the UK uses continuity over the supports, combined with a small contribution from the decking in the span, to provide composite slab hogging and sagging moment resistances that are sufficient to support the loads in a fire situation. Fire tests have validated such an approach. The hogging resistance is significant under fire conditions because the reinforcement (normally in the form of fabric mesh) in the upper part of the slab remains relatively cool and has a reasonable lever arm. The contribution of the decking is small because, acting as external reinforcement, it is directly exposed to the fire and so loses much of its strength. An apparent idiosyncrasy of this approach is that end continuity is normally ignored for ambient temperature design (although it is relatively much less significant in that condition).

When there is no physical continuity at the slab ends the sagging resistance alone will not be sufficient to resist loads in a fire unless bars are placed in the troughs to act as the lower layer of reinforcement, and ensure adequate sagging resistance. Because they are insulated through the provision of concrete cover they remain relatively cool and thus retain significant, if not full, strength. The provision of bars in the troughs is common practice across Europe for all composite slabs, a situation that is reflected in EN 1994-1-1 Annex D (an Informative Annex that is not adopted according to the UK National Annex).

Anchorage of reinforcing bars

Any reinforcing bar requires anchoring before it can resist a tensile force. Anchorage is typically, and most easily, achieved by having sufficient length of bar surrounded by concrete. Eurocode 2 gives rules for anchorage lengths. To ensure the most up-to-date information, although it is not yet publicly available the table below is taken from the latest draft of the 'new' prEN 1992-1-1. It is worth noting that anchorage lengths may be reduced when bars are subject to a lower level of stress.

Ø [mm]	Anchorage length l_{bd} / Ø							
	f_{ck} [MPa]							
	20	25	30	35	40	45	50	60
≤ 12	47	42	38	36	33	31	30	27
14	50	44	41	38	35	33	31	29
16	52	46	42	39	37	35	33	30
20	56	50	46	42	40	37	35	32
25	60	54	49	46	43	40	38	35
28	63	56	51	47	44	42	40	36
32	65	58	53	49	46	44	41	38

NOTE: the values of l_{bd} / Ø are valid for a rebar cover greater than $c_d \geq 1.5$ Ø and for rebars with a design strength of σ_{sd} = 435 Mpa in good bond conditions. For bars in poor bond conditions the values should be multiplied by 1.20. For the cases where σ_{sd} < 435 Mpa the values may be multiplied by (σ_{sd} / 435), but l_{bd} / Ø ≥ 10.

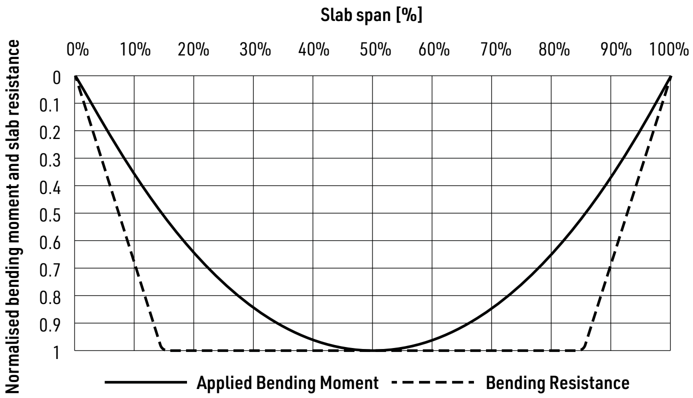
The 'new' prEN 1992-1-2 says that the values given above are also applicable to fire conditions, and when there is no shear reinforcement (as is the case with a composite slab) anchorage lengths should be determined assuming poor bond condition, i.e. increased by 20% as per the note in the table above. However, we propose that this increase need not be applied in the case of composite slabs as it reflects the tendency of concrete to spall when an RC slab is exposed to fire, and the presence of decking prevents such spalling.

Example

For a situation with a 12 mm bar (which is at the upper end of the typical range) in concrete with a characteristic strength of 35 N/mm² (also at the upper end of typical), using information from the table below left the anchorage length needed to achieve yielding of the bar is 12 × 36 = 432 mm. For a 3 m span slab the full strength of the bar could therefore be relied upon anywhere in the middle 70% of the span. For the 15% at either end the sagging resistance will build up linearly from zero at the support. This development of resistance (giving a tri-linear envelope) can be compared to the development of applied moment to see whether the bar anchorage will be adequate. For uniformly distributed loading the applied moment envelope is of course parabolic. The table below shows applied moment and moment resistance at certain distances from the support. The table shows that at only 5% of the way 'into' the span, say 150 mm, the bars would already have sufficient anchorage to generate over one-third of their resistance as ambient temperature, which is more than adequate to resist the applied moment. Values of applied moment and moment resistance are also plotted in the figure below, as a function of span.

Distance into span (%)	Applied moment[1]	Moment resistance[2]
5	0.19	0.35
10	0.36	0.69
20	0.64	1.00

1. expressed as a proportion of the mid-span applied moment
2. expressed as a proportion of the mid-span moment resistance



Conclusion

For situations with uniform loading, or predominantly uniform loading, it can be concluded that straight bars with no extra provision for anchorage will be adequate. As with all composite elements, when the loading is heavily non-uniform, specific checks should be carried out using the principles given above. Resistance could be increased by using larger bars and/or increasing anchorage for example by forming the bar ends into hooks (if space allows).

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AD 463: Corrections to BS 5950-5:1998

Although this Code of practice was withdrawn in 2010, many designers of cold formed thin gauge sections still use it to verify members. During the course of some recent work, we have noticed two problems with Table D1. This table is useful, as it gives expressions for the position of the shear centre and for the Warping constant (known as C_w in BS 5950, and I_w in the Eurocode suite).

In the fourth row of the table, expressions for a lipped 'C' section are given. The lips are facing inward as shown in Figure 1(a). The expression for the Warping constant should only have positive terms within the bracketed part of the equation. The correct expression is:

$$C_w = \frac{b^2 t}{6}(4b_L^3 + 3d^2 b_L + 6db_L^2 + bd^2) - I_x e^2$$

The next row has a lipped 'C' section with the lips facing outwards, as shown in Figure 1(b), with a very similar expression for C_w. These sections are sometimes known as "top hat" sections. The expression for C_w for this section does have a negative term within the bracket and is correct.

The next row in the table (which is over the page in the code) has a diagram of precisely the same 'top hat' section, but very different formulae – it is clear that the diagram is incorrect. The correct shape is a lipped angle, as shown with the appropriate labelling of the elements in Figure 2.

Designers are recommended to review Annex C of BS EN 1993-1-3 which presents a general approach to calculate the Warping constant I_w and the Torsional constant I_t for thin-walled open sections. The advantage of this method is that it is applicable to any shape of cross section, only requiring the centre line co-ordinates of the node points between flat elements of the cross section. The method is appropriate for a spreadsheet

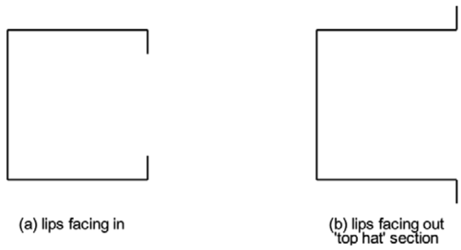

Figure 1: Lipped 'C' sections

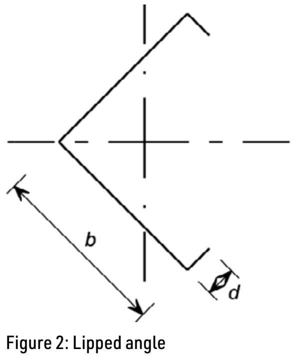

Figure 2: Lipped angle

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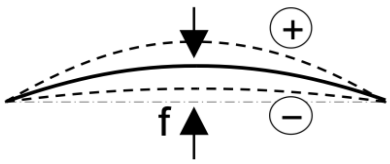
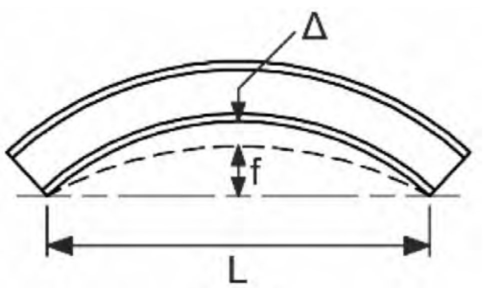
AD 465:
Amendment to clauses on negative tolerances on cambers in the National Structural Steelwork Specification

Engineers generally specify cambers on steel beams to reduce deflection under self-weight. However, when specifying cambers, they should carefully consider the tolerances, especially the negative tolerance ($-\Delta$). For example, in the case of long span beams the negative tolerance can be so large that the beam loses its theoretical camber (f) and is curved in the opposite direction.



In the 6th Edition of the National Structural Steelwork Specification (NSSS), Clauses 7.2.6, 7.4.9 and 7.5.8 required the tolerances on camber to be $\pm L/500$ or 6mm whichever is greater. During the development of the 7th Edition of the NSSS it was recognised that a tighter negative tolerance is necessary to achieve the required camber, and consequently the 7th Edition specified a zero negative tolerance on cambers. An extract from clauses 7.2.6, 7.4.9 and 7.5.8 of the 7th Edition is given below:



Deviation from intended curve or camber f at middle of length L of curved portion when measured with the web horizontal.

$-\Delta = 0$

$+\Delta = L/500$ or 6mm whichever is greater

This is a more onerous requirement than criterion No. 4 in Table B.6 of BS EN 1090-2:2018, which stipulates the following functional tolerances:

- For Class 1, $-\Delta = L/500$ or 6mm whichever is greater
- For Class 2, $-\Delta = L/1000$ or 4mm whichever is greater

Steelwork contractors are finding that in many cases the zero negative tolerance is difficult to achieve and requires more expensive cambering techniques. For long-span beams the Class 1 negative tolerances given in BS EN 1090-2 may not be appropriate because, as explained above, it can result in the beam cambering in the wrong direction. To avoid this situation the more stringent class 2 tolerances from Table B.6 of BS EN 1090-2:2018 for the negative tolerance limit should be adopted in Clauses 7.2.6, 7.4.9 and 7.5.8 of the 7th Edition of the NSSS. The change to the negative tolerance in 7.2.6, 7.4.9 and 7.5.8 is given below.

$-\Delta = L/1000$ or 4 mm, whichever is greater

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AD 466:
Probability factors applied to characteristic wind and snow loads for non-standard return periods

Wind Loads

The SCI receives queries from time to time on the determination of design actions for wind load for design lives other than the usual 50 years. According to BS EN 1991-1-4:2005[1] para. 4.2, the basic wind velocity v_b is multiplied by the probability factor c_{prob} to give the 10 minutes mean wind velocity having the probability p for a given annual exceedance. Equation 4.2 in the para. named above is:

$$c_{prob} = \left(\frac{1 - K \times \ln\left(-\ln\left(1 - p\right)\right)}{1 - K \times \ln\left(-\ln\left(0.98\right)\right)} \right)^{n}$$

The recommended values for K and n are 0.2 and 0.5 respectively. In substituting these values, the denominator gives the constant multiplier 0.75 and the formula becomes:

$$c_{prob} = 0.75\sqrt{1 - 0.2\ln[-\ln(1-p)]}$$

This is as given in SCI publication P394[2] Appendix B, where it is applied to the wind speed derived from the wind map for the UK (Figure NA.1 in the UK National Annex[3]) along with other factors to arrive at the design wind pressure q_p.

BS EN 1991-1-6[4] gives appropriate return periods for the design of structures during execution which may be shorter than 50 years. Example values of the probability factor for other return periods are for a 10 year return period ($p = 0.1$), $c_{prob} = 0.9$ and for 60 years ($p = 0.0167$), $c_{prob} = 1.01$, leading to factors on wind loading equal to 0.82 and 1.02 respectively.

Snow Loads

The adjustment of snow loads for different return periods is also allowed according to BS EN 1993-1-3:2003[5] Annex D, but only for return periods longer than five years, according to the UK National Annex. Here, the characteristic snow load is adjusted for a recurrence interval different from that for the characteristic snow load s_k which is based on an annual probability of exceedance of 0.02 ie a return period of 50 years. The formula for snow load with a return period of n years is given in Annex D as:

$$s_n = s_k \left(\frac{1 - V\frac{\sqrt{6}}{\pi}\left[\ln\left(-\ln\left(1 - P_n\right)\right) + 0.57722\right]}{(1 + 2.5923V)} \right)$$

V is the coefficient of variation for the probability distribution. In the UK V varies depending on location. When determining ψ factors for the UK National Annex to BS EN 1990, a range of values for V were considered[6]. Example values for the factor on characteristic snow load for specific return periods and coefficients of variation are given in the table.

Coefficient of variation V	10 year return period	60 year return period
0.1	0.9	1.01
0.3	0.8	1.02

For site-specific queries, the Meteorological Office should be contacted. The Met. Office suggests using the contact form on the web-page: *https://www.metoffice.gov.uk/services/research-consulting*

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1 BS EN 1991-1-4:2005+A1:2010 *Eurocode 1 - Actions on Structures Part 1-4: General actions – Wind actions*. BSI, 2011
2 A F Hughes, *Wind Actions to BS EN 1991-1-4*, SCI P394, 2014
3 *UK National Annex to Eurocode 1 - Actions on Structures Part 1-4: General actions – Wind actions*. BSI, 2011
4 BS EN 1991-1-6:2005 *Eurocode 1 - Actions on Structures Part 1-6: General actions – Actions during execution*. BSI, 2013
5 BS EN 1991-1-3:2003+A1:2015 *Eurocode 1 - Actions on Structures Part 1-3: General actions – Snow loads*. BSI, 2004
6 Brettle, M E, Currie, D M, Cook N J, *Snow loading in the UK and Eire: Combination of actions given in the Eurocodes*, The Structural Engineer, 18 June 2002

AD 469:
P385 Design of Steel Beams in Torsion – Error in Example 3

An SCI member has identified further errors in SCI publication P385 Design of Steel Beams in torsion.

In example 3 section 3.5.1 Total torque, the total torque on the beam being considered is calculated for four different cases. In each case, the expression given sets out the torque per metre but omits multiplying this quantity by the length of the beam to arrive at the total torque. The stated values for the total torque in each case are in fact correct so the remainder of the example uses the appropriate values.

(See also *AD 452: Errata in P385*)

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AD 473:
Holes in beams for temporary lifting attachments

The SCI has been asked to consider the requirement of clause 6.2.5(6) in BS EN 1993-1-1, which covers the allowance for fastener holes when calculating cross sectional resistance in bending. The clause states that ordinary fastener holes need not be allowed for, provided they are filled by fasteners.

This requirement can lead to problems when – for example – bolts must be placed in holes used for temporary lifting brackets, which then prevents other components such as precast units or decking sitting correctly on the top flange.

BS 5950 presents a less restrictive rule for members in bending in clause 4.2.5.5. According to BS 5950, no allowance need be made for bolt holes in a compression flange in bending.

SCI recommend that, within limits, bolt holes in the compression flange of beams used for temporary attachments need not be allowed for and need not be filled with bolts. In an element with holes subject to compression, if the flange yields locally, the strength of the material increases as the cross section deforms, due to strain hardening.

Some limitation on the reduction in cross section is appropriate, to prevent multiple holes in a cross section being neglected on the basis of the above recommendation.

SCI consider there is no requirement to apply the material factor $\gamma_{M2} = 1.1$ (from the UK NA, used in the net area tension checks) when calculating the compression resistance. SCI recommend that the resistance of the net section of the flange in compression may be based on the ultimate strength.

At full utilisation, the assumed design resistance of the flange is $f_y A_g$

The resistance of the net area in compression may be taken as $f_u A_{net}$

No allowance for bolt holes need be made when

$$A_{net} > \frac{f_y A_g}{f_u}$$

If the member is not fully utilised, the design resistance of the flange may be based on a reduced stress when completing the above verification.

In the final condition, for example in a composite beam, holes in the top flange for temporary lifting attachments have little impact.

It should be noted that this advice contradicts the specific requirements of the Eurocode, so should be agreed with the designer with overall responsibility for the structure. In due course it is hoped that this advice will be presented in the NSSS.

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