Eurocode 3: Design of steel structures

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Introduction

Structural engineers should be encouraged that at least in steel, design conforming to Eurocode 3 (BS EN 1993-1-1) is not significantly different to BS 5950. In fact, if the changes in presentation are stripped away, in most areas design is almost identical to both standards. This is of course to be expected, as the steel itself knows no difference, and the UK has enjoyed the benefits of mature design standards for decades – it would be very surprising if there were dramatic changes from previous UK practice. There are well-known changes:

- the nomenclature is different, with more Greek symbols;
- subscripts are important, and informative;
- the Eurocode is arranged by structural phenomena, not design routine;
- most checks are presented as expressions, not graphs or look-up tables;
- the ‘simple’ approaches found in BS 5950 are generally missing – the Eurocode presents the rigorous methods.

The importance of the national annex

This chapter presents an overview of the key areas of steel design to the Eurocode. In addition to highlighting the more interesting design issues, one general point about all Eurocode design should be made, which is that the national annex is crucially important. The national annex allows countries to set various parameters, but these are not limited to a modest list of factors. The national annex may also define which methods are allowed, or set limits
on a range of application. In practice, the national annex often embraces the opportunity to define national parameters with enthusiasm, providing revised methods, new tables and important limitations. In the core Eurocode document, the possibility that the national annex might have an impact is identified with a Note to the clause. The Notes are easy to miss, and ignoring the impact of the national annex could be very significant. A very strong recommendation is to review each national annex, and mark the core Eurocode with the important changes. Each and every Eurocode Part has a national annex, so this is no small task. The national annex to be used is that for the country where the structure is to be built.

**Loading**

The loading Parts of the Eurocode are independent from the resistance Parts, but the determination of actions is nevertheless a key part of the design process. Designers are offered four ultimate limit states in BS EN 1990 – EQU, STR, GEO and FAT, covering equilibrium, strength, geotechnics and fatigue. STR will be the most relevant for building designers, where there is a choice of how the combination of factored loads (called ‘the design value of the combination of actions’) can be calculated:

- using expression 6.10; or
- using the most onerous of expressions 6.10a and 6.10b.

Both expressions 6.10a and 6.10b produce a lower ULS design value than 6.10, so are recommended for greatest economy. Modest experience will conclude that expression 6.10b is almost always the critical expression. Use of expression 6.10 will always be a little conservative, so can always be used.

Using expression 6.10b, members designed for vertical load only, such as beams and many columns, will be designed for $1.25 \times$ permanent actions $+ 1.5 \times$ variable actions, which is an immediate attraction of around 8% compared to BS 5950.

The stability systems will be subject to higher loads, as they must carry the lateral loads (wind) factored by 1.5, plus modest imperfection forces. This compares with 1.4 $\times$ wind in BS 5950, with no notional horizontal forces.

BS EN 1991-1-4 deserves a special mention. The national annex to this Part is a very substantial document, and should be consulted. For designers, a number of important points should be noted.
The national annex recommends that roof coefficients be taken from BS 6399-2, not the Eurocode.

Internal pressures may be calculated, based on opening ratios, or the two cases of +0.2 and –0.3 should be considered.

The Eurocode comments on the common practice of so-called ‘elective’ dominant openings – those that would be dominant but are considered shut at ULS. An additional accidental case must be considered with the dominant opening.

BS EN 1990 also refers to SLS – but then directs the designer to the material standard – in this case BS EN 1993-1-1. The designer is then referred to the national annex, which in the UK simply confirms the status quo – horizontal and vertical deflections are to be checked under unfactored variable actions and permanent actions need not be included. The same familiar deflection limits in BS 5950 (loved and loathed in equal measure by those who consider them attractively vague/too definitive) reappear in the national annex.

Member resistance

In most cases, calculated resistances are close to those calculated using BS 5950, and the design process is very similar. The fundamental structural mechanics have not changed and dramatic changes in resistance should ring alarm bells. Experienced designers will have a feel for the sorts of member sizes they anticipate and that experience is equally appropriate to Eurocode designs as it is to BS 5950.

Steel strengths

Table 3.1 of BS EN 1993-1-1 provides steel strengths for thicknesses up to 40 mm, and over 40 mm. This is a good example of the subtle influence of the national annex, which is invited to allow the use of Table 3.1, or to take the steel strengths from the product standard. The UK national annex adopts the latter, so in the UK steel strengths will continue to change at thicknesses of 16 mm, 40 mm, 63 mm etc. The UK national annex also notes that for ultimate strengths, where a range is given in the product standard, the lowest value in the range should be adopted. Again this is subtle, but for S275 steels, this means that the ultimate strength, $f_{\text{u}}$, must be taken as 410 N/mm$^2$ rather
than the more familiar 430 N/mm². The impact is modest, but will affect tying resistances, where ultimate strengths are used.

**Steel sub-grade**

Choice of steel sub-grade is very important to ensure that brittle failure does not occur. Although this issue is addressed in BS EN 1993-1-10, the strong advice is to obtain PD 6695-10 from BSI (‘PD’ stands for published document). The PD takes the complicated approach in the standard, and presents it is an altogether more amenable form – use of the PD is recommended. In the PD, tables of limiting thicknesses are presented for internal environments (–5 °C) and external environments (–15 °C). The limiting thickness depends on the type of fabrication (considering stress raisers and residual stresses) and the state of stress.

**Imperfections**

BS 5950 introduced designers to imperfections – the notional horizontal forces (NHF) are used ‘to allow for the effects of practical imperfections such as a lack of verticality’. The Eurocode deals with the same issue by ‘equivalent horizontal forces’ (EHF). The NHF were only applied in the ‘gravity load’ combination, whereas unless the externally applied lateral actions are more that 15% of the vertical actions, the EHF appear in every load combination. In practice, the 15% rule means that for multi-storey frames, expect that EHF will appear. In portal frames, where the vertical loads are modest, the EHF will appear in the ‘gravity’ combination, but probably not in other combinations.

BS EN 1993-1-1 describes three types of imperfection:

a) frame imperfections (as discussed previously);

b) member imperfections;

c) bracing imperfections.

Frame imperfections are allowed for by the EHF, as discussed previously. The value of the EHF is given as a proportion, φ, of the factored vertical loads, where \( \phi = \phi_0 \alpha_h \alpha_m \). The basic value of \( \phi_0 \) is 1/200, or the 0.5% of BS 5950. Factors \( \phi_h \) and \( \alpha_m \) can safely be set to 1.0, but allow for the height of the structure and the number of columns that contribute to the force on
the bracing system. (Eagle-eyed designers will appreciate that the definition of $\alpha_\text{cr}$ has been taken from BS EN 1992-1-1, rather than the (inappropriate) definition in BS EN 1993-1-1.)

Designers should not worry about member imperfections. Member imperfections have always been allowed for in the member design checks found in the standard, so no change in practice is needed. Some engineers wish to design members from first principles, accounting for real material behaviour, residual stresses etc., and the Eurocode reminds such experts to allow for the inevitable member imperfections.

Bracing imperfections will be new to UK designers – but the principle is not new. If frames are imperfect, bracing systems will also be imperfect. The Eurocode allows the imperfection to be allowed for by small additional forces applied at the nodes. The result will be slightly larger forces in the bracing members. Designing for these larger forces allows for the second order effects.

**Frame stability**

The Eurocode approach will be familiar to UK designers. The only difference is that the check for sensitivity to second order effects is carried out under all the lateral loads – the externally applied loads and the EHF. Under BS 5950 the check was carried out under NHF alone. The result is almost identical, since the BS 5950 expression was a particular instance of the general expression used in BS EN 1993-1-1:

$$\alpha_\text{cr} = \frac{H_{Ed}}{V_{Ed}} \left( \frac{b}{\delta_{H,Ed}} \right)$$

The fact that the wind loads are included makes little difference to the outcome, because as the horizontal action, $H_{Ed}$, increases, so the horizontal deflection, $\delta_{H,Ed}$, increases.

Second order effects are small enough to be ignored if $\alpha_\text{cr} > 10$, and if second order effects need to be allowed for, the Eurocode offers an amplifier which is identical to $k_{\text{amp}}$ in BS 5950.
Cross-sectional resistance

The only significant change is to the shear area for rolled sections, and then the change is largely cosmetic. The shear area of a rolled section according to BS 5950 was \( D_t \), but the Eurocode changes this to a rather more complex area, as illustrated in Figure 3.1.

The effect is modest, as can be seen from the typical examples in Table 3.1.

The advantageous effect of the increased shear area according to EN 1993-1-1 is somewhat offset by the formula for the shear resistance. According to the Eurocode, the shear resistance \( V_{pl,Rd} \) is

\[
V_{pl,Rd} = \frac{A_v\left(\frac{f_y}{\sqrt{3}}\right)}{\gamma_{M0}}
\]

which incorporates \( 1/\sqrt{3} \) compared to the familiar figure of 0.6 in BS 5950.

![Figure 3.1. Shear areas for rolled sections in BS 5950 and BS EN 1993-1-1](image)

<table>
<thead>
<tr>
<th>Section</th>
<th>Shear area (mm(^2))</th>
<th>Shear resistance S275 (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BS 5950</td>
<td>BS EN 1993-1-1</td>
<td>BS 5950</td>
</tr>
<tr>
<td>533 × 201 × 92 UKB</td>
<td>5072</td>
<td>837</td>
</tr>
<tr>
<td>356 × 171 × 57 UKB</td>
<td>2900</td>
<td>479</td>
</tr>
<tr>
<td>203 × 133 × 23 UKB</td>
<td>1158</td>
<td>191</td>
</tr>
<tr>
<td>BS 5950</td>
<td>BS EN 1993-1-1</td>
<td>BS 5950</td>
</tr>
<tr>
<td>5072</td>
<td>5450</td>
<td>909</td>
</tr>
<tr>
<td>2900</td>
<td>3193</td>
<td>501</td>
</tr>
<tr>
<td>1158</td>
<td>1285</td>
<td>197</td>
</tr>
</tbody>
</table>
Buckling of compression members

In addition to flexural buckling, the Eurocode covers torsional buckling and torsional-flexural buckling, which are illustrated in Figure 3.2. These forms of buckling are uncommon, as they involve a bi-symmetric cruciform section or an asymmetric section used as a compression member.

The most significant change in the buckling section is the presentation of slenderness. In the Eurocode, the general expression is that the slenderness, \( \bar{\lambda} \) is given by

\[
\bar{\lambda} = \sqrt{\frac{A f_y}{N_{cr}}}
\]

where

\( N_{cr} \) is the elastic critical buckling load for the buckling mode being considered

For the common case of flexural buckling, \( N_{cr} \) is more commonly known as the Euler load, given by

\[
\frac{\pi^2 EI}{L^2}
\]

For flexural buckling, the calculation of \( N_{cr} \) is not the only route to determine \( \bar{\lambda} \), because

\[
\bar{\lambda} = \frac{BS 5950 slenderness (L / r_y)}{a \text{ factor}}
\]

which is an approach that may appeal to many designers. The factor varies with steel grade.

Having calculated the slenderness, the designer must select which curve to use (what type of member? Which axis of buckling? Etc.). The proportion of yield strength to be used when calculating the resistance (there is no direct equivalent of the compressive strength) can be found from a graph (Figure 3.3) or by calculation.
Figure 3.2. Torsional and torsional-flexural buckling. (a) Torsional buckling. (b) Torsional-flexural buckling
Lateral torsional buckling

Having seen the presentation of flexural buckling, designers will immediately recognize the similar presentation of lateral torsional buckling. The slenderness, \( \lambda_{LT} \), is given by

\[
\lambda_{LT} = \frac{W_{y}f_{y}}{\sqrt{M_{cr}}}
\]

in which bending terms have replaced the axial terms. One unfathomable omission from the Eurocode is any expression to calculate \( M_{cr} \). Sources of non-conflicting complementary information (NCCI) can be used, such as http://www.access-steel.com or the useful (and free) programme \( LTBeam \) from CTICM in France. In BS 5950, the beneficial effects of a non-uniform bending moment diagram are accounted for by the factor \( m_{LT} \), which is applied to the calculated buckling resistance \( M_{b} \). Thus it is straightforward to prepare look-up tables which contain single values of \( M_{b} \) – the \( m_{LT} \) adjustment is ‘outside’ the calculation of \( M_{b} \). The Eurocode takes a different approach and allows for the effect of a non-uniform bending moment using a \( C_{1} \) factor within the calculation of \( M_{cr} \). This means that look-up tables, such as those in the ‘Blue Book’ must present values of buckling resistance for different \( C_{1} \) values, as shown in Table 3.2.

![Figure 3.3. Buckling curves](image-url)
### Table 3.2. Typical look-up tables for lateral torsional buckling

<table>
<thead>
<tr>
<th>Designation</th>
<th>( C_1 )</th>
<th>( M_{b,Rd} ) (kNm)</th>
<th>( M_{c,y,Rd} ) = 692</th>
<th>( M_{c,z,Rd} ) = 106</th>
<th>Class = 1</th>
<th>Second Moment of Area ( y-y ) axis ( l_y )</th>
</tr>
</thead>
<tbody>
<tr>
<td>533 × 210 × 101</td>
<td>1.00</td>
<td>681 602 523 450 387 336 295 262 236 214 196 181 168</td>
<td>61 500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.50</td>
<td>692 692 644 581 519 462 411 367 329 297 269 250 233</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.00</td>
<td>692 692 692 670 618 565 514 467 425 387 354 324 298</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.50</td>
<td>692 692 692 692 689 643 598 553 511 471 435 402 373</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.75</td>
<td>692 692 692 692 692 675 632 590 549 510 473 440 409</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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The importance to the national annex can again be demonstrated within this area of the standard. The core Eurocode defines which buckling curve to use based on the $h/b$ ratio (section height/section breadth) and proposes that an expression can be used to calculate the buckling resistance of rolled sections and 'equivalent welded sections'. The national annex is given the opportunity to influence the curves and the buckling resistance calculation. The UK national annex takes this opportunity, defining additional $h/b$ limits. The UK national annex also specifies the constants to be used in the calculation of buckling resistance, accepting the recommended values for rolled sections, but specifying values that downgrade the resistance of welded sections.

Many designers will remember beam design according to BS 449, and look back with fondness to the simple tables in that standard. Because the physics has not changed, it is equally possible to present information in a similar way, as shown in Table 3.3.

In Table 3.3, the $h/t_f$ is the same as $D/t$ in BS 449, and the slenderness is identical. The main body of the table gives the proportion of the yield strength to be used when calculating the buckling resistance. There is some conservatism, since tables like this assume the most onerous value of $C_1$, but the result is a very simple table.

In practice, most designers use member resistance tables or software – which will deal with any (apparent) complexity with ease.

**Buckling resistances – the outcomes**

The theory is interesting, but what is the result? For flexural buckling, the calculated resistances are almost identical. For lateral torsional buckling, the resistance calculated to the Eurocode can be considerably higher than that according to BS 5950 – some 25% increase for a 7m 533 × 210 UKB. This increased resistance may not be significant if deflection or other SLS criteria govern, but it is a significant advantage when the buckling resistance is the governing check.

**Combined bending and axial load**

The expression to verify in-plane and out-of-plane buckling are not for the faint-hearted. They are equivalent to the ‘more exact’ approaches in BS 5950,
Table 3.3. Presentation of lateral torsional buckling tables

$$\lambda_z = L/i_z$$

<table>
<thead>
<tr>
<th>$\lambda_z$</th>
<th>$\bar{\lambda}_z$</th>
<th>$h/t_f$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5</td>
<td>10</td>
</tr>
<tr>
<td>30</td>
<td>0.35</td>
<td>1.00</td>
</tr>
<tr>
<td>40</td>
<td>0.46</td>
<td>1.00</td>
</tr>
<tr>
<td>50</td>
<td>0.58</td>
<td>1.00</td>
</tr>
<tr>
<td>60</td>
<td>0.69</td>
<td>1.00</td>
</tr>
<tr>
<td>70</td>
<td>0.81</td>
<td>1.00</td>
</tr>
<tr>
<td>80</td>
<td>0.92</td>
<td>0.98</td>
</tr>
<tr>
<td>90</td>
<td>1.04</td>
<td>0.97</td>
</tr>
<tr>
<td>100</td>
<td>1.15</td>
<td>0.95</td>
</tr>
</tbody>
</table>
The essential guide to Eurocodes transition

and on inspection designers will see the same fundamentals in both standards. Unfortunately, the Eurocodes in general are not noted for ‘simple’ alternatives – the expressions for combined bending and axial load are one area where simple expressions would be welcome, at least for manual design. In UK practice, the most common occurrence of combined bending and axial load is when designing columns in simple construction. In this situation, http://www.access-steel.com provides NCCI that makes the verification straightforward, as the interaction expression becomes:

$$\frac{N_{Ed}}{N_{min,b,Rd}} + \frac{M_{y,Ed}}{M_{b,Rd}} + 1.5 \frac{M_{z,Ed}}{M_{ch,z,Rd}} \leq 1$$

In most orthodox cases, the moments are either zero (no net moment) or small and the column verification is dominated by the axial load. Software will be used by most designers using the general expressions in the Eurocode, and there are a number of design aids available from the SCI at www.steelbiz.org.

Connections

There are no significant changes in connection component strengths designed according to BS EN 1991-1-8. Bolts are as strong as they always have been (92 kN for an M20 8.8 conforming to BS 5950 becomes 94 kN in the Eurocode). Welds also have about the same resistance. There are dramatic increases in the calculated bearing resistance. BS 5950 provided a bearing strength that was chosen to limit the deformation at working load to 1.5 mm. The Eurocode has no such restriction, and therefore the resistances are much higher. The UK national annex notes that there may be some circumstances where reduced deformation is desirable and provides an alternative γ factor – bringing the bearing resistance close to the BS 5950 value.

In a very significant departure for UK practice, the Eurocode requires that connections be classified – meaning that for example, designers should demonstrate that their connection detail really is nominally pinned, if that has been assumed. The Eurocode reflects practice in some other European countries, where connection stiffness is calculated. Thankfully, there is some relief in that the Eurocode states that connections may be classified on the basis of previous satisfactory performance as an alternative to calculation. UK designers will be relieved that the UK national annex goes further and states that connections designed in accordance with the ‘Green Books’ on simple and moment connections can be considered as simple and rigid respectively.
Caution is urged for connections outside the familiar details in those books, as a demonstration of connection stiffness by calculation may be required. This is not a trivial calculation and usually demonstrates that a simple connection is not as ‘pinned’ as was hoped.

The UK methods of designing simple connections have been largely followed in European guides, which calculate the resistance of each component and identify the minimum resistance. An emerging issue is that tying requirements depend on building classification (this is covered in the UK Building Regulations and BS EN 1991-1-7) and this in turn means that some relatively low storey structures must accommodate relatively high tying forces through the beam to column connection. The much-loved partial depth flexible end plate has a relatively low resistance in tying, leading to the use of full depth end plates or thicker end plates. Additional standard details have been prepared which comprise slightly thicker end plates, welded to both flanges. The (enhanced) resistance of these new details have been calculated in accordance with BS EN 1993-1-1, and will be published during 2010.

Support tools

A wealth of support is already available, including NCCI, design guides, worked examples books of member resistance (the ‘Blue’ and ‘Red’ books) and concise guides. A number of publishers have guidance available. As might be expected, the Steel Construction Institute has published a range of guides, with support from BCSA and Tata. The ‘Concise Guide’ deserves a special mention as it tries to offer guidance on the common cases of design, from loading through to the detailed design checks. More guidance, covering composite members, simple connections, bridge design, fire engineering and other topics will be published in 2010 – most orthodox issues will be covered. The steel sector maintains the NCCI\textsuperscript{1} website (http://www.steel-ncci.co.uk) for steel-related information. Whilst this material is likely to be technically

\textsuperscript{1}Whilst this material is likely to be technically authoritative, not all of it has been reviewed by the UK national committee and users should satisfy themselves of its fitness for their particular purpose. In particular, they should be aware that material indicated as not having been endorsed by the committee might contain elements that are in conflict with the Eurocode. (Source National Annex to EN 1993-1-1:2005)
authoritative, not all of it has been reviewed by the UK national committee, and users should satisfy themselves of its fitness for their particular purpose. In particular, they should be aware that material indicated as not having been endorsed by the committee might contain elements that are in conflict with the Eurocode.

As the Eurocodes are used, more NCCI will be required and will be added to this website.

Conclusions

This chapter has introduced some of the technical issues surrounding the design of steel structures. The author’s conclusion is that there are no significant technical challenges – once designers have amassed modest experience they will appreciate that the design processes are straightforward and familiar, albeit dressed in slightly different wrapping to existing standards. There will be issues of familiarity (‘which Eurocode Part, and which clause?’) but these will diminish with time. Dealing with the sheer numbers of documents, each with a national annex, and the management of the change, especially across several materials, is likely to be the larger challenge.

The second conclusion is that the influence of the national annex should not be underestimated. The examples referred to in this chapter are not meant to be exhaustive or to imply that they are the most significant – they are simply examples to demonstrate the importance of the national annex. A careful review of each national annex and its impact is strongly recommended.

The third conclusion is that there are some economic advantages in designing in accordance with the Eurocodes (based on the technical changes alone). Loads are reduced, and resistance is increased in some areas. For multi-storey frames the reduction in loads will be a significant saving. For lighter structures significantly affected by wind actions, such as portal frames, the jury is still out.

The final observation is that designers already have a great deal of support available, which will ease the transition. Much of this support is online, and entirely free.