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The EC3 approach to the design of columns, beams and beam-columns

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Abstract. Procedures given in Eurocode 3 Part 1.1 (EN 1993-1-1) for design of the main types of structural member under given systems of loading are presented and described. Whereas some of these e.g. the procedure for axially loaded columns, are little changed from the early concept that appeared more than 25 years ago in the European Recommendations and have subsequently been adopted in many other steel codes of the world, others such as the interaction formulae for beam-columns are new, with aspects of the provisions still under development. For each type of member the basis of the procedure is described and some comparative comments made.

Key words: beams; beam-columns; buckling curves; columns; EN 1993; Eurocode 3; structural design.

1. Introduction

The first package of the Structural Eurocodes dealing with steel construction (EN 1993, also referred to as EC3), including the basic Part 1.1, is expected to be released during the latter part of 2004. It is the result of a long process, originating from the 1977 ECCS recommendations and moving through the original draft EC3 of 1983, the ENV of 1992 and culminating in the final set of EN documents. These are, of course, the outcome of numerous discussions and negotiations over the content, the depth of treatment and the style of presentation considered most appropriate and have finally been agreed on a pan-European basis. Essentially, the documents represent a scientifically correct set of procedures - rather than material prepared principally so that it is easy to use in practice.

Although a series of background documents giving the scientific basis e.g. comparing the provisions with experimental data, has been promised, the actual appearance of such material has been spasmodic. Scientific papers have been written, describing different aspects of the provisions, at various stages - but as the details of these provisions change during the preparation process, so the value of such papers reduces because they do not necessarily accord precisely with the latest version of the specific provisions.

It is the expectation that each member country within Europe will, at the appropriate time, replace their National Codes with the equivalent Eurocodes. In cases where no comparable document already

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exists, this is, of course, a relatively straightforward process. However, several European countries already have a well established network of National Codes that has, in turn spawned a large portfolio of the supporting material regularly used by designers in their day to day activities. It is a significant challenge for such countries to devise ways in which their existing portfolio based on their National Standards can migrate to a similarly supportive portfolio based on the Eurocodes. For the UK a project designed to map the essentials of such a process has recently been undertaken by a committee convened by the Institution of Structural Engineers acting on behalf of the Office of the Deputy Prime Minister.

This paper is restricted to certain technical provisions from the basic Part 1.1 document. In particular, it deals with the design of columns, beams and beam-columns by presenting the main features of the design approach and giving some background information and comments. It draws on materials contained within the Designers' Guide to EC3 (Gardner and Nethercot 2005) written by the authors as one of the series being published in the UK.

2. Column design

The basic relationship between strength and stability for a centrally loaded column has its origins in the ECCS column curves. Since first being proposed (Beer and Schulz 1970), these have undergone several, relatively minor, refinements to give the set of five curves shown as Fig. 1. They are described by a Perry-Robertson type of formula, in which a constant plateau length corresponding to $\bar{\lambda}$ = 0.2 is used together with a selection of imperfection coefficients α (given by Table 1), each of which defines a different curve.

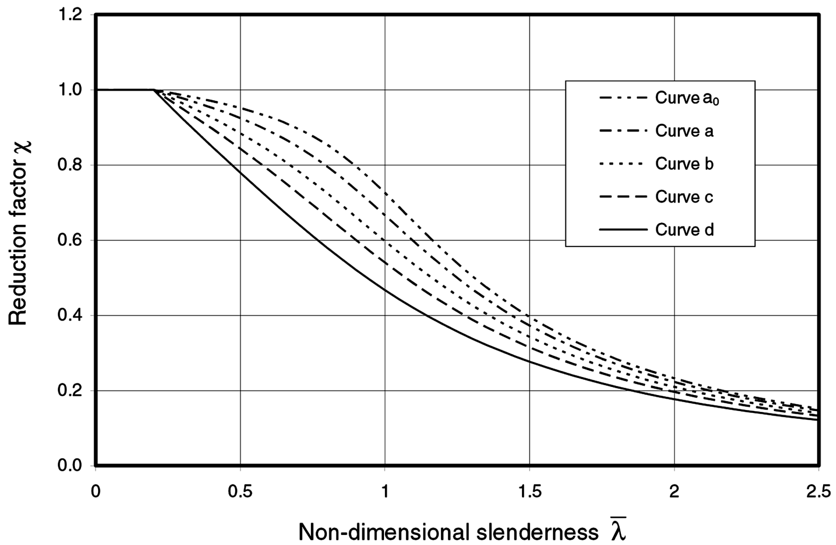
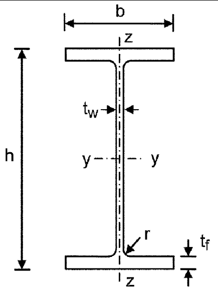
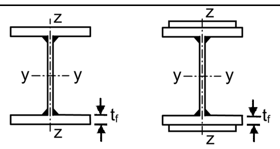
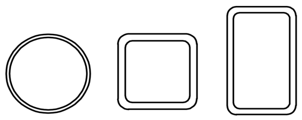
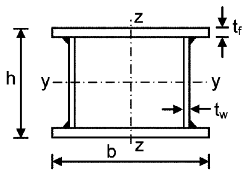
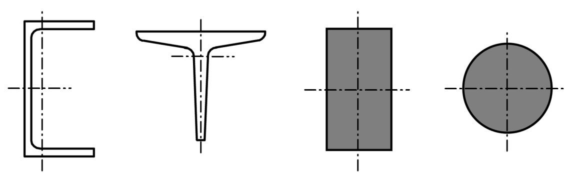
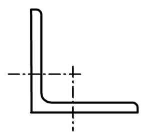


Fig. 1 Eurocode 3 flexural buckling curves

Table 1 Imperfection factors for buckling curves (Table 6.1 of EN 1993-1-1)

Buckling curve	a_0	a	b	c	d
Imperfection factor α	0.13	0.21	0.34	0.49	0.76

Table 2 Selection of buckling curve for a cross-section (Table 6.2 of EN 1993-1-1)

Cross-section		Limits		Buckling about axis	Buckling curve: S 235, S 275, S 355, S 420	Buckling curve: S 460
Rolled I-sections		$h/b > 1.2$	$t_f \le 40$ mm	y - y	a	a_0
				z - z	b	a_0
			40 mm $< t_f \le 100$ mm	y - y	b	a
				z - z	c	a
		$h/b \le 1.2$	$t_f \le 100$ mm	y - y	b	a
				z - z	c	a
			$t_f > 100$ mm	y - y	d	c
				z - z	d	c
Welded I-sections			$t_f \le 40$ mm	y - y	b	b
				z - z	c	c
			$t_f > 40$ mm	y - y	c	c
				z - z	d	d
Hollow sections			hot finished	any	a	a_0
			cold formed	any	c	c
Welded box sections			generally (except as below)	any	b	b
			thick welds: $a > 0.5\, t_f$ $b/t_f < 30$ $h/t_w < 30$	any	c	c
U-, T- and solid sections				any	c	c
L-sections				any	b	b

Different types of cross-section are allocated to the appropriate column curves by means of the selection table shown in Table 2. Following section classification to determine whether or not the cross-section is slender (Class 4), the design steps therefore correspond to:

1. Determine non-dimensional slenderness:

$$\bar{\lambda} = \sqrt{A f_y / N_{cr}} \tag{1}$$

in which A is to be replaced by an effective value A_{eff} suitably reduced to allow for loss of effectiveness due to local plate element buckling for Class 4 sections.

2. Calculate the intermediate factor:

$$\Phi = 0.5[1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2] \tag{2}$$

3. Calculate the reduction factor:

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \text{ but } \chi \leq 1.0 \tag{3}$$

4. Obtain the design buckling resistance:

$$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}} \quad \left(\text{or } \frac{\chi A_{eff} f_y}{\gamma_{M1}} \text{ for Class 4 sections}\right) \tag{4}$$

This process is also applicable for situations in which torsional or torsional-flexural buckling is likely to govern e.g. open sections composed of thin plates to form an unsymmetrical shape, with the proviso that N_{cr} in step 1 must now relate to the most critical mode of buckling. Whilst this form of buckling will not often govern for normal hot-rolled or welded sections, the formulation has distinct advantages for cold-formed sections (governed by the Part 1.3) for which these other modes are more likely.

Unlike many Codes, EC3 provides very little direct guidance on determining N_{cr} for various conditions of end restraint e.g. through the provision of effective length factors. This is a consequence of the basic philosophy adopted for the document that it should not provide 'textbook material'. Users accustomed, for example, to the style of BS 5950 will find this unfamiliar and will need to consult other guidance material - where previously they could expect to find all they needed in a single document.

3. Beam design

The first consideration in the design of beams is by now almost universally used process of cross-section classification. EC3 uses the normal 4 classes with some refinement in defining the limits used for class boundaries. Unlike most Codes it utilises flat widths of plating i.e., distances between radii, fillets, welds etc. Its provisions are generally a little more liberal than those in other current (or older) Codes. The effective width method is used when dealing with Class 4 cross-sections. A special procedure is possible if only the web of a section falls within Class 3 (and flanges are either Class 1 or Class 2) to permit design to be based on a Class 2 plastic distribution but with the use of an effective web, as shown in Fig. 2.

For laterally unrestrained beams, the buckling resistance moment $M_{b,Rd}$ is obtained from a parallel process to that given above for columns, with the following variations: the definition of slenderness $\bar{\lambda}_{LT}$ as $\sqrt{W_y f_y / M_{cr}}$, the selection of the imperfection factor α_{LT}, and the final determination of $M_{b,Rd}$ as $\chi_{LT} W_y f_y / \gamma_{M1}$. W_y should be taken as the plastic, elastic or effective section modulus for Class 1 and 2, Class 3 or Class 4 cross-sections respectively.

Eurocode 3 defines lateral torsional buckling curves for two cases:

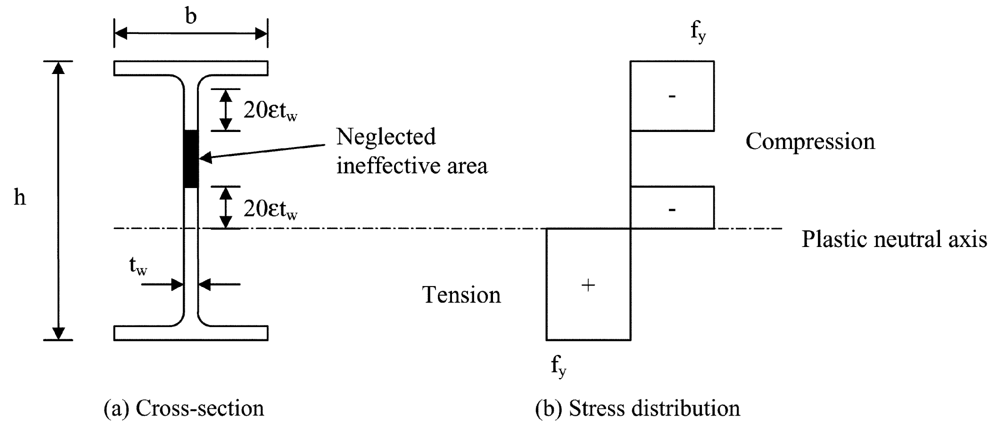


Fig. 2 Procedure for Class 3 beam webs

- General case (clause 6.3.2.2 of EN 1993-1-1)
- Rolled sections or equivalent welded sections (clause 6.3.2.3 of EN 1993-1-1)

The general case may be applied to all common section types, including rolled sections, but unlike clause 6.3.2.3 of EN 1993-1-1, it may also be applied outside the standard range of rolled sections. For example, it may be applied to plate girders (of larger dimensions than standard rolled sections), to castellated and to cellular beams.

Four curves (curve a_0 is not utilised for lateral-torsional buckling) are given for different types of cross-sections, allocated as indicated in Table 3. Note that there is a different allocation of buckling curves for the two methods, but for a given buckling curve, the imperfection factor for lateral-torsional buckling α_{LT} is the same between the two methods and the same as that for column buckling α.

No assistance is provided when determining the elastic critical moment for lateral torsional buckling M_{cr} i.e., the Code user is expected to have obtained this from their own knowledge e.g. by reference to suitable texts (Galambos 1998).

For the general case, the formulations that define the buckling curves (Eq. (2) and Eq. (3)) are as for column buckling (except with $\bar{\lambda}_{LT}$ used in place of $\bar{\lambda}$ and α_{LT} used in place of α). For the case of 'Rolled sections or equivalent welded sections', the values of Φ_{LT} and χ_{LT} are calculated from a slightly modified expression, where the 0.2 plateau slenderness is replaced by a variable $\bar{\lambda}_{LT,0}$ and by inserting a factor β. Expressions for Φ_{LT} and χ_{LT} are given by Eq. (5) and Eq. (6), respectively. Values of $\bar{\lambda}_{LT,0}$ and β are left to national choice subject to a maximum value of 0.4 and a minimum value of 0.75 respectively.

Table 3 Buckling curve selection for laterally unrestrained beams

Cross-section	Limits	General case		Rolled or equivalent welded case	
		Buckling curve	α_{LT}	Buckling curve	α_{LT}
Rolled I-sections	$h/b \leq 2$	*a*	0.21	*b*	0.34
	$h/b > 2$	*b*	0.34	*c*	0.49
Welded I-sections	$h/b \leq 2$	*c*	0.49	*c*	0.49
	$h/b > 2$	*d*	0.76	*d*	0.76
Other cross-sections	-	*d*	0.76	*d*	0.76

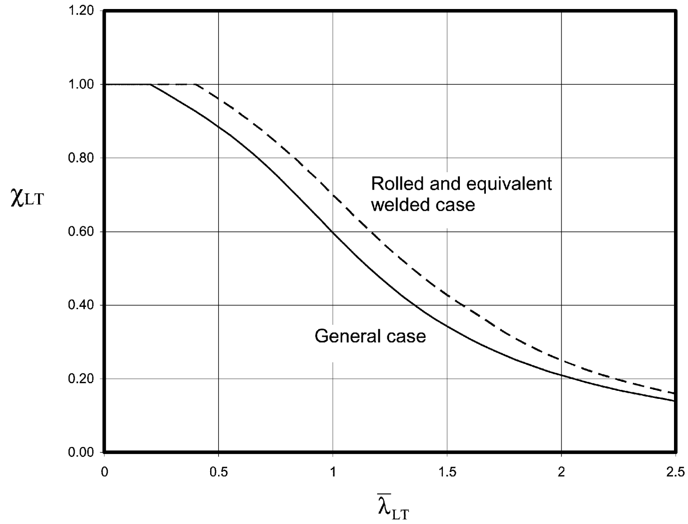


Fig. 3 Lateral torsional buckling curves for the general case and for rolled sections or equivalent welded sections

$$\Phi_{LT} = 0.5[1 + \alpha_{LT}(\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta\bar{\lambda}_{LT}^2] \quad (5)$$

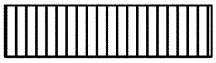
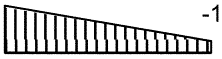
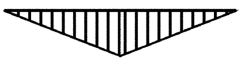
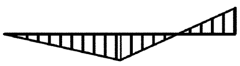
$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \beta\bar{\lambda}_{LT}^2}} \quad \text{but} \quad \chi_{LT} \leq 1.0 \quad \text{and} \quad \chi_{LT} \leq \frac{1}{\bar{\lambda}_{LT}^2} \quad (6)$$

Fig. 3 compares the lateral torsional buckling curves of the general case and the case for rolled sections or equivalent welded sections. The imperfection factor α_{LT} for buckling curve *b* has been used for the comparison. Overall, it may be seen that the curve for the rolled and equivalent welded case is more favourable than that for the general case, but of particular interest is the plateau length of the curves. Since no lateral torsional buckling checks are required within this plateau length (and resistance may simply be based on the in-plane cross-section strength), the rolled sections or equivalent welded sections case may offer significant savings in calculation effort for some arrangements.

The provision of multiple buckling curves and the allocation of different cross-section types to an appropriate curve has been the subject of much debate - especially by the membership of ECCS TC8. On the one hand, test data with its inevitable scatter due to the response of nominally identical specimens exhibiting some spread because of variation in properties between specimens such as: material strength, lack of straightness, residual stress distribution and magnitude etc., plus tacit recognition that virtually all beams in practical situations benefit from some degree of restraint, leads to the pragmatic view of a single curve and a longer plateau (Georgescu and Dubina 2002). Careful numerical analysis on the other hand (Salzgeber and Greiner 2000) with controlled variation in input parameters permits consistent differences to be identified, thus supporting the use of more than one curve. The assumption of ideal support arrangements removes the influence of any 'accidental restraint'.

A procedure to allow for the shape of the bending moment diagram is also included for the rolled sections and equivalent welded sections case. This provides a factor *f* by which the basic χ_{LT} value should be divided. Values of *f* are obtained from Eq. (7) and k_c is determined from Table 4.

Table 4 Correction factors k_c

Bending moment distribution	k_c
 $\psi = +1$	1.0
 $-1 \leq \psi \leq 1$	$\dfrac{1}{1.33 - 0.33\,\psi}$
	0.94
	0.90
	0.91
	0.86
	0.77
	0.82

$$f = 1 - 0.5(1 - k_c)\,[1 - 2.0\,(\bar{\lambda}_{LT} - 0.8)^2] \tag{7}$$

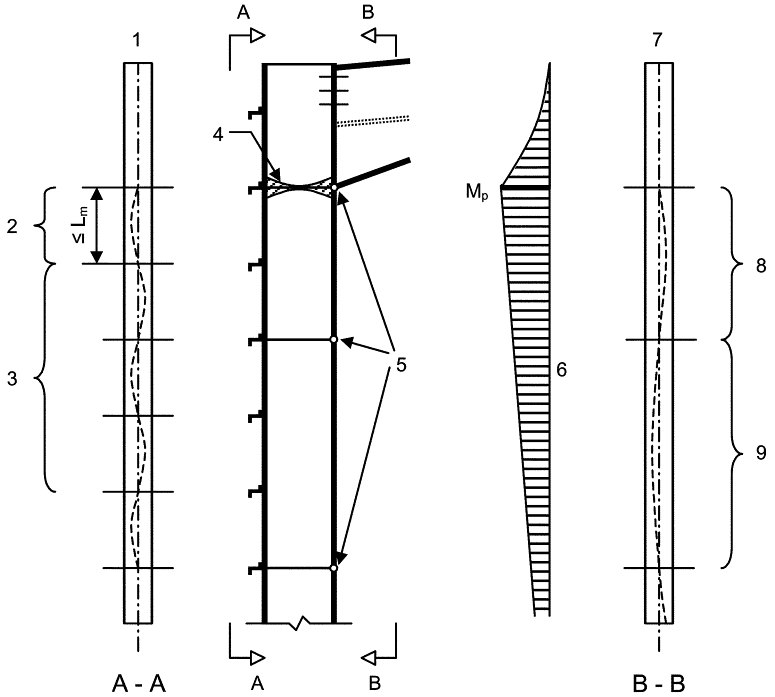
A further simplified method for designing laterally unrestrained beams is also included, in which the length of compression flange between lateral restraints is checked as a strut.

Guidance for special circumstances is provided in the informative Annex BB of EN 1993-1-1 that covers:

- required shear stiffness of sheeting to permit the use of $\chi_{LT} = 1.0$
- required rotational stiffness of sheeting to permit the use of $\chi_{LT} = 1.0$
- stable lengths of segments between points of restraint for uniform or tapered members containing plastic hinges.

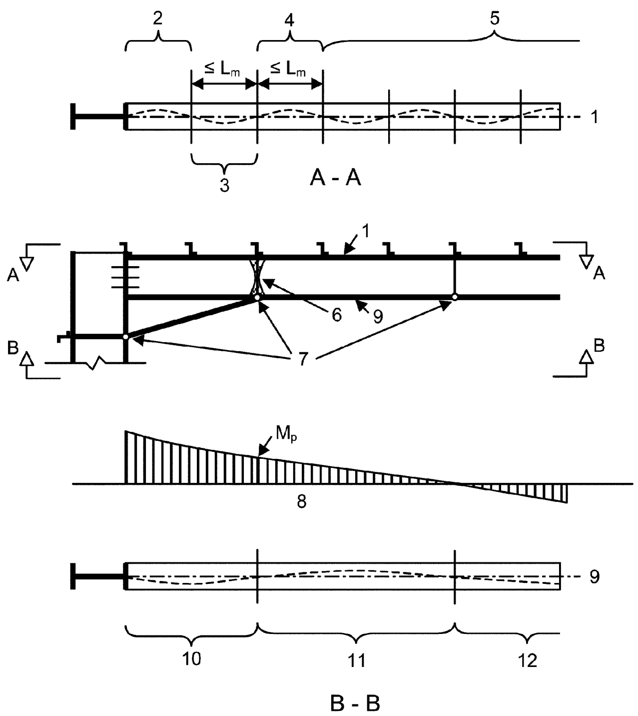
This last provision is intended for application when designing portal frames so as to check column and rafter members as illustrated in Figs. 4 and 5.

The basis of the rules governing the restraint provided by sheeting is largely the work constructed at the Technical University of Berlin (Lindner 1998, Lindner and Gregull 1986) and Budapest (Vrany 2001), whilst the provisions for plastically designed members draws heavily upon British work (Davies and Brown 1996).



1. tension flange
2. plastic stable length
3. elastic section
4. plastic hinge
5. restraints
6. bending moment diagram
7. compression flange
8. plastic with tension flange restraint, stable length = L_s
9. elastic with tension flange restraint, χ and χ_{LT} from N_{cr} and M_{cr} including tension flange restraint

Fig. 4 Checks in a member without a haunch



1. tension flange
2. elastic section
3. plastic stable length
4. plastic stable length
5. elastic section
6. plastic hinge
7. restraints
8. bending moment diagram
9. compression flange
10. plastic stable length
11. plastic stable length
12. elastic section, χ and χ_{LT} from N_{cr} and M_{cr} including tension flange restraint

Fig. 5 Checks in a member with a three flange haunch

4. Beam-columns

General provisions are included in EC3 to use second order analysis (including consideration of out-of-plane effects) for the design of beam-columns, in which the geometrical imperfections prescribed in clause 5.3.2 of the Code are to be used.

Also included is the more familiar interaction method for checking individual members between points of appropriate restraint. This is done using the pair of formulae given by Eqs. (8) and (9), whereby Eq. (8) is for failure about the *y-y* (major) axis and Eq. (9) for the *z-z* (minor) axis.

$$\frac{N_{Ed}}{\frac{\chi_y N_{RK}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,RK}}{\gamma_{M1}}} + k_{yz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,RK}}{\gamma_{M1}}} \leq 1 \tag{8}$$

$$\frac{N_{Ed}}{\frac{\chi_z N_{RK}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,RK}}{\gamma_{M1}}} + k_{zz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,RK}}{\gamma_{M1}}} \leq 1 \tag{9}$$

in which:

N_{Ed}, $M_{y.Ed}$ and $M_{z.Ed}$	are the design values of the compression force and the maximum moments about the *y-y* and the *z-z* axes along the member, respectively
$\Delta M_{y,Ed}$, $\Delta M_{z,Ed}$	are moments due to the shift of the centroidal axis for Class 4 sections
N_{Rk}, $M_{y.Rk}$ and $M_{z.Rk}$	are the characteristic values of the compression resistance of the cross-section and the bending moment resistances of the cross-section about the *y-y* and the *z-z* axes, respectively
χ_y and χ_z	are the reduction factors due to flexural buckling
χ_{LT}	is the reduction factor due to lateral torsional buckling, taken as unity for members that are not susceptible to torsional deformation
k_{yy}, k_{yz}, k_{zy} and k_{zz}	are the interaction factors k_{ij}

The characteristic values of the cross-sectional resistances N_{Rk}, $M_{y,Rk}$ and $M_{z,Rk}$ may be calculated as for the design resistances, but without dividing by the partial γ_M factor.

Values for the interaction factors k_{ij} are to be obtained from one of two methods given in Annex A of EN 1993-1-1 (alternative method 1) or Annex B of EN 1993-1-1 (alternative method 2). These originate from two different approaches to the beam-column interaction problem - enhancing the elastic resistance taking account of buckling effects to include partial plastification of the cross-section or reducing the plastic cross-sectional resistance to allow for instability effects. Both approaches distinguish between cross-sections susceptible or not susceptible to torsion, as well as between elastic (for Class 3 and 4 cross-sections) and plastic (for Class 1 and 2 cross-sections) properties. Which of the two alternative methods (Annex A or Annex B) to be used is left as a matter for National Choice.

Both approaches involve the use of a large number of coefficients and do not provide an easy appreciation of cause and effect, i.e., it is not possible for the user to readily decide how an inappropriate design selection that is either unsafe or too conservative should be modified so as to get closer to satisfying the design formulae. Spreadsheets or similar automated calculation techniques would appear to be necessary when using this procedure.

The approach given in Annex A provides more complex and more exact interaction formulae for general applications (Lindner 2003). Its origins lie in work conducted at the Universities of Liege and Clermont-Ferrand (Boissonnade *et al.* 2002). Its basis is a rigorous second-order elastic analysis incorporating the concepts of amplification, equivalent moment and buckling length. It is assumed these are still valid in the inelastic range.

Ignoring Class 4 cross-sections, dropping the safety factor γ_{M1} and considering only 'members not susceptible to torsional deformations', the general form for the pair of controlling interaction formulae becomes:

$$\frac{N_{Ed}}{\chi_y N_{p1,Rd}} + \mu_y \left[\frac{C_{my,0} M_{y,Ed}}{\left(1 - \frac{N_{Ed}}{N_{cr,y}}\right) C_{yy} M_{p1,y,Rd}} + \alpha^* \frac{C_{mz,0} M_{y,Ed}}{\left(1 - \frac{N_{Ed}}{N_{cr,z}}\right) C_{yz} M_{p1,z,Rd}} \right] \leq 1 \tag{10}$$

$$\frac{N_{Ed}}{\chi_z N_{p1,Rd}} + \mu_z \left(\beta^* \frac{C_{my,0} M_{y,Ed}}{\left(1 - \frac{N_{Ed}}{N_{cr,y}}\right) C_{zy} M_{p1,y,Rd}} + \frac{C_{mz,0} M_{z,Ed}}{\left(1 - \frac{N_{Ed}}{N_{cr,z}}\right) C_{zz} M_{p1,z,Rd}} \right) \leq 1 \tag{11}$$

in which α^* and β^* are coefficients to account for the plastic interaction between major and minor moments, given by:

$$\alpha^* = 0.6 \sqrt{\frac{w_z}{w_y}} \tag{12}$$

$$\beta^* = 0.6 \sqrt{\frac{w_y}{w_z}} \tag{13}$$

$C_{my,0}$ and $C_{mz,0}$ are equivalent uniform moment factors, defined in Table A.2 of EN 1993-1-1.

The C_{ii} (C_{yy} and C_{zz}) and C_{ij} (C_{yz} and C_{zy}) factors are included to account for the combined effects of instability and plasticity and are defined as Eqs. (14) and (15). For Class 3 and 4 cross-sections, these factors are equal to unity.

$$C_{ii} = 1 + (w_i - 1) \left[2 - \frac{1.6}{w_i} C_{mi}^2 (\bar{\lambda}_{\max} + \bar{\lambda}_{\max}^2) \right] \frac{N_{Ed}}{N_{p1,Rd}} \geq \frac{W_{e1,i}}{W_{p1,i}} \tag{14}$$

$$C_{ij} = 1 + (w_j - 1) \left[2 - 14 \frac{C_{mj}^2 \bar{\lambda}_{\max}^2}{w_j^5} \right] \frac{N_{Ed}}{N_{p1,Rd}} \geq 0.6 \sqrt{\frac{w_j}{w_i}} \frac{W_{e1,j}}{W_{p1,j}} \tag{15}$$

When lateral-torsional bucking is possible (i.e., for 'members susceptible to torsional deformations') Eqs. (10) and (11) become further expanded with the introduction of five factors (a_{LT}, b_{LT}, c_{LT}, d_{LT} and e_{LT}). For the case of members not susceptible to torsional deformation these factors were suppressed by a zero value for the non-dimensional slenderness for lateral-torsional buckling due to uniform moment $\bar{\lambda}_0$.

The simpler of the two alternative methods for the determination of the four interaction factors k_{ij} is set out in Annex B of EN 1993-1-1 and has been described by Lindner (2003). It contains more straightforward interaction formulae for standard cases, and is especially suitable for calculation by hand. Its origins lie in work conducted at the Technical Universities of Berlin and Graz (Lindner 2003). The interaction factors for the case of 'members not susceptible to torsional deformations', for use in Eqs. (8) and (9) are as follows:

$k_{yy} = C_{my}(1 + [\bar{\lambda}_y - 0.2]n_y) \leq C_{my}(1 + 0.8n_y)$ for Class 1 and 2 sections (16a)

$k_{yy} = C_{my}(1 + 0.6\bar{\lambda}_y n_y) \leq C_{my}(1 + 0.6n_y)$ for Class 3 and 4 sections (16b)

$k_{yz} = 0.6k_{zz}$ for Class 1 and 2 sections (17a)

$k_{yz} = k_{zz}$ for Class 3 and 4 sections (17a)

$k_{zy} = 0.6k_{yy}$ for Class 1 and 2 sections (18a)

$k_{zy} = 0.8k_{yy}$ for Class 3 and 4 sections (18a)

$k_{zz} = C_{mz}(1 + [2\bar{\lambda}_z - 0.6]n_z) \leq C_{mz}(1 + 1.4n_z)$ for Class 1 and 2 I-sections (19a)

$k_{zz} = C_{mz}(1 + [\bar{\lambda}_z - 0.2]n_z) \leq C_{mz}(1 + 0.8n_z)$ for Class 1 and 2 RHS (19b)

$k_{zz} = C_{mz}(1 + 0.6\bar{\lambda}_z n_z) \leq C_{mz}(1 + 0.6n_z)$ for Class 3 and 4 sections (19c)

And for 'members that are susceptible to torsional deformations', the expressions are as above, except for k_{zy} which is given by Eq. (20).

For Class 1 and 2 sections with $\bar{\lambda}_z \geq 0.4$:

$$k_{zy} = 1 - [(0.1\bar{\lambda}_z n_z)/(C_{mLT} - 0.25)] \geq 1 - [(0.1n_z)/(C_{mLT} - 0.25)] \tag{20a}$$

For Class 1 and 2 sections with $\bar{\lambda}_z < 0.4$:

$$k_{zy} = 0.6 + \bar{\lambda}_z \leq 1 - [(0.1\bar{\lambda}_z n_z)/(C_{mLT} - 0.25)] \tag{20b}$$

For Class 3 and 4 sections:

$$k_{zy} = 1 - [(0.05\bar{\lambda}_z n_z)/(C_{mLT} - 0.25)] \geq 1 - [(0.05n_z)/(C_{mLT} - 0.25)] \tag{20c}$$

where $n_y = \dfrac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}$ and $n_z = \dfrac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}$

C_{my}, C_{mz} and C_{mLT} are equivalent uniform moment factors determined from Table B.3 of EN 1993-1-1; C_{my} relates to in-plane major axis bending; C_{mz} relates to in-plane minor axis bending; and C_{mLT} relates to out-of-plane buckling. For end moment loading, the factors may be determined from Eq. (21), where ψ is the ratio of the end moments. Note that a minimum value of 0.4 is specified.

$$C_{mi} = 0.6 + 0.4\psi \geq 0.4 \tag{21}$$

Current studies in Germany (Lindner 2004) are seeking to further extend the scope of these provisions to include the effects of torsional loading, both with and without the presence of an axial load.

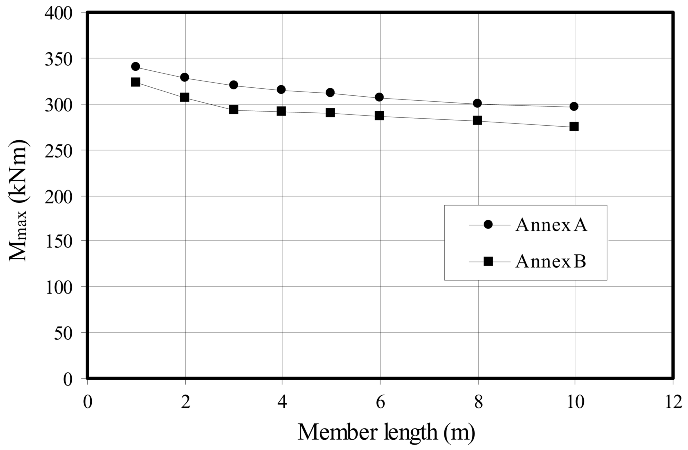


Fig. 6 Maximum sustainable moments for Case 1 ($M_{y,Ed} = M_{z,Ed}$)

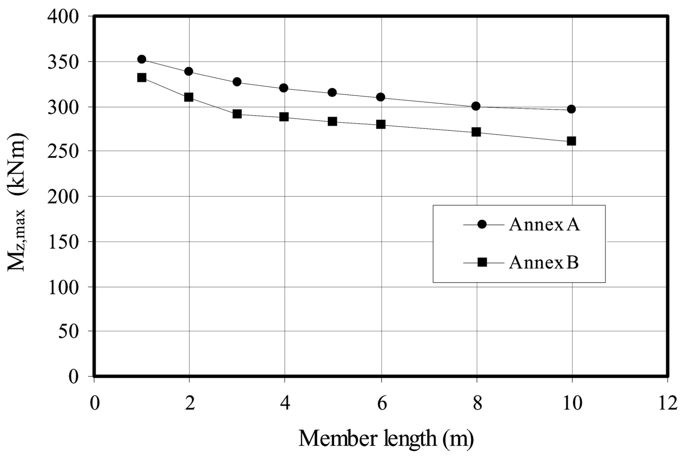


Fig. 7 Maximum sustainable $M_{z,Ed}$ for Case 2 (in presence of $M_{y,Ed}$ = 300 kNm)

5. Application of the beam-column interaction formulae

A simple study was conducted in order to assess the implications on design efficiency of adopting either the Annex A or the Annex B method for determining the interaction factors k_{ij}. Throughout the study, a 305×305×198 UC section was examined, and pin-end conditions and no torsional restraints were assumed.

Three cases were considered. For Case 1, equal uniform end moments were applied about the major and the minor axes (i.e., $M_{y,Ed} = M_{z,Ed}$), and the maximum moments that could be sustained using the Annex A and the Annex B interaction factors were determined. The results for Case 1 are shown in Fig. 6.

For Case 2, the maximum minor axis moments that could be sustained in the presence of a major axis moment of 300 kNm (i.e., $M_{y,Ed}$ = 300 kNm) using the Annex A and the Annex B interaction factors were determined. The results for Case 2 are shown in Fig. 7.

For Case 3, the maximum axial loads that could be sustained in the presence of major and minor axis

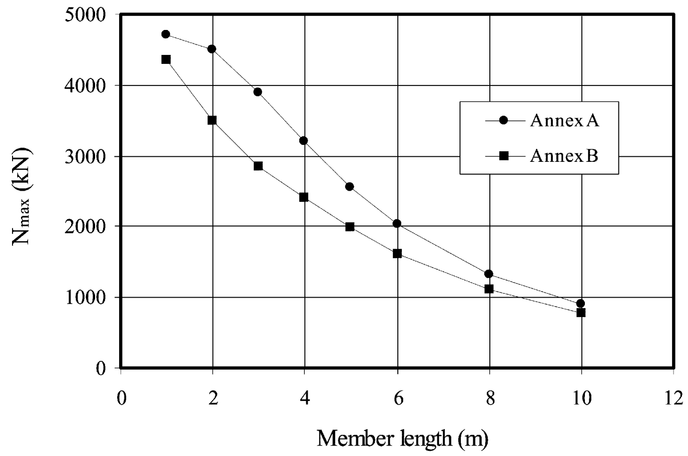


Fig. 8 Maximum sustainable axial load for Case 3 (in presence of $M_{y,Ed} = 0.25M_{c,y,Rd}$ and $M_{z,Ed} = 0.25M_{c,z,Rd}$)

moments of 25% of their respective cross-section resistances (i.e., $M_{y,Ed} = 0.25M_{c,y,Rd}$ and $M_{z,Ed} = 0.25M_{c,z,Rd}$) using the Annex A and the Annex B interaction factors were determined. The results for Case 3 are shown in Fig. 8.

For the cases considered, the results indicate that the Annex A method provides more competitive solutions than those derived on the basis of the Annex B interaction factors k_{ij}. On average, approximately 10% gains in resistance may be observed. This is to be expected, since the Annex A method is presented as the 'more exact' one, and requires the greater calculation effort. It is suggested that for initial design purposes, Annex B would provide a quicker and more conservative solution, whilst for detailed design, or to check whether small increases in loading can be tolerated, the Annex A method would be adopted.

6. Conclusions

The procedures of EC3 - Part 1.1 for the design of columns, beams and beam-columns have been presented together with a brief description of their basis. Much of the material is comparatively recent in origin, with the result that users will need access to detailed explanations if they are to be able to use the new procedures in a suitable fashion. The complex nature of several of the equations employed makes this difficult, especially as their evaluation is likely to require the use of automated calculation methods (particularly for the beam-column interaction factors). For the cases considered, the beam-column interaction factors derived from Annex A of EN 1993-1-1 offer more competitive solutions than those from Annex B.

Acknowledgements

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