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Template for comments and secretariat observations

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Do any clauses require editorial or technical correction?							
BE 1		All		general	EN 1993-1-1 cannot miss the goal: assist designers in their daily practice. Though it is expected to reflect the best knowledge acquired in the field, it should contain only material ready in the format of (as) simple (as possible) design rules or application rules and thus excluding what still look like scientist's development.		2
BE 2		All		general	The number of nationally determined parameters should be decreased.		2
BE 3		1	Fig 1-1		Main geometrical symbols for cross-sections are given in Figure 1.1. Symbol h_w that is a key value for the shear resistance is missing.	Implement Figure 1.1 with h_w symbol for sections with constant and sections with tapered flanges.	1 See SC3 Doc N1895
BE 4		5		general	Whole contents of Section 5 could be revisited so as to result in a more comprehensive presentation of it.		2
BE 5		5		te, ed	Actually EN1993-1-1 allows several alternatives for designing structures. But most designers do not read it accordingly and mishandle consequence of the choice of a global analysis on the type of further checks. That interrelation needs to be more clearly stated to	Rewrite Section 5 and Introduce summary Tables stressing the interrelations between the type of global analysis and the still required section/member verifications.	2

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					prevent the designers' feeling that - compared to former national codes - EN1993-1-1 increases the number and complexity of the design tasks. Such summary tables do exist in the literature. See for instance: *Frame design including joint behavior, RFCS Report, EUR 18563 EN, European Commission, 1988.*		
BE 6		5.2.1(3) in relation with 5.3.2(4)	equ (5.1) in relation with equ (5.7)	te	Investigations by Merchant et al suggested the so-called Merchant-Rankine formula, according to which the ultimate load amplifier (thus including the effects of material yielding) is very close to the 1st order plastic amplifier as long as it is less than 10% of the elastic critical amplifier, thus F_{cr}/F_{Ed}. > 10. A lot of later studies supported that limit of 10, even when semi-rigid and partial strength joints are significantly yielded at the ultimate load (See: *Jaspart, J.P. : « Etude de la semi-rigidité des nœuds poutre-colonne et son influence sur la résistance et la stabilité des ossatures en acier », PhD thesis, Liège University, 1991*). In contrast, scientific documents justifying the limit value of 15 are not available and NOTE is not actually convincing.	Give equation (5.1) with a single limit value of 10 irrespectively of the type - "elastic" or "plastic" - of global analysis and remove NOTE. This solves the problem of inconsistency de facto. If the doublet of values 10 and 15 remains, then a consistency needs to be restored between equations (5.1) and (5.7).	1, see DE 12 4, equation (5.7) is independent of criteria (5.1); (5.7) is a criteria for imperfections, (5.1) a criteria for first or second order theory

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					Moreover eq (5.7) in 5.3.2(4) is provided as a general rule and therefore applies whatever "elastic" or "plastic" analysis; however its derivation is based on considerations that refer to a single F_{cr}/F_{Ed} = 10. There is thus inconsistency between equs (5.1) and (5.7).		
BE 7		5.2.1(4)B		ed	Symbol h represents the storey height while it represents the section depth in Figure 1-1.	Care for this possible confusion.	1
BE 8		5.2.1(4)B		ed	Figure 5.1 and 5.2.1(4) text: Symbol V_{Ed} represents the total design vertical load on the structure on the bottom of the storey but is in general used as design value of the shear force V_{Ed} (6.2.6)	Care for this possible confusion.	1
BE 9		5.2.1(4)B	equ (5.2)	te, ed	It is a matter of fact that α_{cr} or/and $(\alpha_{cr})_i$ depend only on the gravity loads (principle of elastic stability). Writing (5.2) as it is may appear in contradiction with that principle. It should be noticed that the ratio $H_{Ed}/\delta_{H,Ed}$ – that is nothing else than the lateral stiffness K of the structure/storey - is indeed independent	Write (5.2) so as to introduce the lateral stiffness K and express K = $H_{Ed}/\delta_{H,Ed}$ in the definition below.	2, see FR1

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					of H_{Ed} because of first order analysis.		
BE 10		5.2.1(4)B	Equ (5.2)	te, ed	In 5.2.1(4)B, equation (5.2) is confusing. It concerns one storey while the same symbol α_{cr} is used in equ (5.1) to characterize the whole structure. Clarification is required.	Write: "…..is satisfied for each storey. Then the critical amplifier α_{cr} of the portal frame is given as the lowest of the $\alpha_{cr,i}$ values calculated for all the storeys. For each storey i, $\alpha_{cr,i}$ may be calculated using the following approximative formula, provided that the axial compression in the beams or rafters is not significant: $\alpha_{cr,i} = K_i\ h_i / (V_{Ed})_i$ with $K_i = (H_{Ed})_i/(\delta_{H,Ed})_i$ the lateral stiffness of the storey and where $(H_{Ed})_i$ is the design value of the horizontal reaction at the bottom of the storey i to the horizontal loads and fictitious horizontal loads, see 5.3.2(7) $(V_{Ed})_i$ is the total design vertical load on the structure on the bottom of the storey i $(\delta_{H,Ed})_i$ is the horizontal displacement at the top of the storey i, relative to the bottom of the storey i, when the frame is loaded with horizontal loads (e.g. wind) and fictitious horizontal loads which are applied at each floor level	2, see FR 1

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						h_i is the height of the storey i	
BE 11		5.2.2(7)	b)	ed	The last sentence could lead to an uneconomic design by disregarding to consider a more favorable buckling length.	Write the last sentence as follows: … This verification should take account of end moments and forces from the global analysis of the structure, including global second order effects and global imperfections (see 5.3.2) when relevant and be based on a buckling length corresponding to a non-sway mode. Conservatively and for sake of simplicity this buckling length may be taken to the system length.	2
BE 12		5.2.2(8)			Even if this method is satisfactory in many cases, it is basically not correct to check the section resistance on the basis of internal forces resulting from first order theory, i.e. without any second order amplification.	Amplify the first order internal forces by a fixed value e.g. as it was in clause 5.2.6.2(8) of ENV 1993-1-1.	2, for traditional equivalent column method more detailed rules should be given, see for example German NA or former DIN
BE 13		5.3.2(3)		ed	Symbol h represents the height of the structure while it represents the section depth in Figure 1-1 and the storey height in 5.2.1(4)B .	Care for this possible confusion.	1

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BE 14		5.3.2(3)	Table 5.1	te, ed	In Table 5.1, the magnitude of the equivalent geometric imperfection is shown as depending on “elastic “or “plastic” analysis. The wording “elastic“ or “plastic” addresses the way the verification of the member is conducted and not the type of analysis used for the structure. Therefore the wording “analysis” is in appropriate. <u>Note:</u> In Belgian NA, “analysis” is replaced by “section verification” and the values in Table 1 are said normative.	In the heading of both 2[nd] and 3[rd] columns in Table 5.1, the wording “check” (or “verification”) should be substituted for the wording “analysis”. Whatever the values given in Table 5.1, it is expected they become normative.	1, see DE 20 4
BE 15		5.3.2(3) in relation with 6.3.1.2		te, ed	The “equivalent geometric imperfection” is expected to produce same effects as those of structural and geometric imperfections (see clause 5.3.4(2) of EN1993-1-1). Its values were proposed for the first time by ECCS TWG8/1-2; see: *European Convention for Constructional Steelwork: Ultimate limit state calculations of sway frames with rigid joints. ECCS, Brussels, publ. n°33, 1984*): a_0: L/750, a: L/500, b: L/250, c: L/200, d: L/150. Values of Table 5.1 are not consistent with those originally published and publications/background documents	The magnitudes of the equivalent geometric imperfection given in Table 5.1 are worth being revisited and possibly changed. Whatever the values given in Table 5.1 – possibly after alterations –, it is not conceivable they may be chosen in national annexes; they should thus be made normative.	6, see DE 19 4

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					justifying them are missing. Some designers argued (See: *Private conversations with de Ville de Goyet, BEG, Liège)* that calculations in accordance with EN buckling curves and numerical simulations using the equivalent geometric imperfection of Table 5.1 in accordance with 5.3.4(3) may significantly diverge. The magnitude of the so-called «equivalent geometric imperfection» given in Table 5.1 is thus a much disputable matter.		
BE 16		5.3.2(3) in relation with 5.3.2(6)		te, ed	Plastic second order analysis – which incorporates the plastic section check - is far from being used in daily practice. Therefore let us provisionally disregard that possibility. Then values given in Table 5.1 are likely to be regularly used in two situations: 1. Where the initial bow of compressions elements MUST be modeled (in accordance with §5.3.2(6) of EN 1993-1-1) in order get internal forces that include the effects of exceptionally flexible columns. 2. Where the elastic/plastic check of the compression member is made with a	1. For sake of simplicity, and <u>for the purpose of global analysis only,</u> in 5.3.2(3): a) Replace Table 5.1 and the attached note by a *single* fixed and rough value of the initial imperfection e_0 (for instance $L/250$ or something similar) independent of the type of global analysis (“elastic” or “plastic”). b) Specify that for a plastic second order analysis, the structure shall be modeled with magnitude of initial bow of compression members given in “new” clause	4

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					second order analysis (see 5.3.4(2)) of the member instead of using the buckling curves. Consequently: • Circumstances where values of Table 5.1 are directly useful are very restricted. • The respective purposes of these values in the two above situations are quite different and not especially univocally linked. Use the values of Table 5.1 to get, with a very great accuracy, same carrying capacity as the buckling curves is hopeless; especially because independently of the magnitude of the equivalent initial imperfection, there is an effect of the slenderness that is not reflected in Table 5.1.	5.3.4(2) (see below). 2. In clause 5.3.4(2): a) Introduce <u>for the purpose of verification with second order elastic analysis,</u> a table similar to Table 5-1 - with possibly slightly revised values -. b) Make a clear statement that carrying capacity derived from elastic second order analysis of compression members (possibly taking into account rotational restraints) is only a *good estimate* compared to the one obtained with the buckling curves.	
BE 17		5.3.2(6) in relation with 5.2.2(5)		te, ed	In the context of a compression member, equation (5.8) corresponds to a limit ratio of 4 between both critical and applied axial loads (if $N_{Ed} = N_{c,Rd}$). That condition may be expected in relation with the boundary of 3 given in Clause 5.2.2(5)B for a similar ratio.	For sake of simplicity, replace $\alpha_{cr} \geq 3$ by $\alpha_{cr} \geq 4$ in 5.2.2(5)B to restore consistency with equation (5.8).	4, Is it necessary to have consistency between 5.3.2(6) and 5.2.2(5)?
BE 18		5.3.3	(2) and Figure 5.6	ed	There could be confusion between stabilizing forces (for the beam) and	Replace twice “equivalent stabilizing force” by “equivalent force”	1

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					destabilizing forces (for the bracing).		
BE 19		5.3.4		te	Background of this clause is missing.	Clarification is needed.	4
BE 20		5.4.2		te	It should be useful to give some guidance regarding the section properties to be used in global analysis according to the classification of the cross section.	Elastic global analysis may be conducted with the elastic properties of the gross cross-sections. If the global analysis is not yet explicitly taking into account local effects (shear lag and plate buckling), the resistance check must be based on the effective cross-section.	3, 5.2.1(5) is sufficient
BE 21		5.4.2(3)		te, ed	In accordance with EN 1993-1-1, the global analysis of a structure made of cross sections the resistance of which is governed by local buckling (Class 4 sections) MUST be “elastic”. The wording “may also” used in 5.4.2(3) makes the sentence questionable.	Rewrite 5.4.2(3). Substitute “shall be” for “may also”;	1, but should instead of shall
BE 22		5.5			No comment is made on what to do when sections are subjected to combined major and minor axial bending with or without additional axial force. Tables 5.2 focuses on mono-axial bending with or without additional axial force.	Some general guidance should be given when biaxial bending with or without axial force.	4
BE 23		5.5.2	Table 5.2	te, ed	The general relation between the reduced slenderness and the b/t ratio is	Revisit the b/t limit ratios of Table 5.2 in order to remove some inconsistencies	5, see N1898

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					(see EN1993-1-5): $\bar{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr}}} = \frac{\bar{b}/t}{28{,}4\,\varepsilon\,\sqrt{k_\sigma}}$ The length $\bar{\lambda}_{p0}$ of the yield plateau present in the normalized plate buckling curves represents the range of full effectiveness; it is drawn from (corrigenda to EN1993-1-5): $\bar{\lambda}_p \leq 0{,}5 + \sqrt{0{,}085 - 0{,}055\psi}$ where the sign "equal" is substituted for the inequality. For Class 3, one gets accordingly λ_{po} = 0,673 for pure compression (ψ = 1) and 0,874 for pure bending (ψ = -1). Only few people know that the lengths λ_{po} of the plateau for Class 2 and Class 1 limits were respectively fixed at 0,6 and 0,5. See *ESDEP: Lectures 7.2 and 7.3.* Several *b/t* limit ratios specified in Table 5-2 show inconsistencies compared those that would be derived from that background.	between EN1993-1-1 and EN1993-1-5. In 5.5.2, introduce a clause where the background of the b/t limit ratios for Class 1, Class 2 and Class 3 is briefly given and mention that rounded values are given in Tables 5.2 for use in practice. Note: It is quite easy to give the background for flat plate elements. It is more questionable for circular cross-sections (see below)	3, explanation in the text of the code should be avoided
BE		5.5	Table 5.2 Sheet 3	te, ed	Surprisingly the *d/t* limits for circular hollow section of Class 3 are not derived	Give the background of the *d/t* limit ratios for Class1, Class 2 and Class 3 circular	

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					from the elastic shell buckling theory with due consideration of some plateau length. Background for Class 1 and Class 2 circular hollow sections are the more questionable. Search for documents justifying values given in Table 5.2-Sheet 3 is vain.	hollow sections. If necessary, change accordingly some *d/t* limit values for circular hollow sections after revisiting.	6, see comments by the piling group
BE 24		6.2.1(8) and 6.2.1(9) in relation with 5.5.2(1)		te, ed	Though expressed differently, the contents of Clauses 6.2.1(8) and 6.2.1(9) are redundant with some parts of 5.5.2(1). Designers appreciate the statement of NOTE in 6.2.1(9).	Remove text of Clauses 6.2.1(8) and 6.2.1(9) but not the NOTE. and Convert NOTE in 6.2.1(9) in a clause replacing 3.2.1(8) and 6.2.1(9) and write it as follows: "For Class 3 and Class 4 cross-sections, the extreme fibres may be assumed at the mid-plane of the flanges for ULS checks. For fatigue see EN 1993-1-9."	2, see al-so FI41 4, see also German comments
BE 25		6.2.1(10)		te, ed	According to 6.2.1(10), Class 3 sections are allowed to benefit from plastic reserves of the tension zone where yielding first occurs on the tension side. Such an allowance is not offered to Class 4 sections. That discrimination is source of discontinuity in the resistance magnitude at the borderline between Class 3 and	Write 6.2.1(10) by substituting "Class 3 or Class 4 cross-section" for "Class 3 cross-section".	4

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					Class 4 sections.		
BE 26		6.2.6(3)		Te, ed	A clear definition of h_w is missing. What is "depth of a web"? See also Figure 1.1. What to do if the flanges are tapered?	H_w should be the distance between the flanges.	5, see N1895
BE 27		6.3.1.2			European buckling curves are characteristic curves, established based on: i) characteristic distribution and magnitude of measured residual stresses, and ii) characteristic sinusoidal initial bow amounting (*L*/1000). See : *[1] Beer H. et Schulz G. : Base théoriques des courbes européennes de flambement. Construction Métallique, n° 3, 1970, [2] European Convention for Constructional Steelwork: Manual on Stability of steel structures. ECCS, Brussels, n°22, 1976, and* [3] *European Convention for Constructional Steelwork: Ultimate limit state calculations of sway frames with rigid joints. ECCS, Brussels, n°33, 1984.* More especially the geometric imperfection L/1000 is worth being mentioned for two reasons: a) stress the effects of residual stresses by comparing with the "equivalent" geometric imperfection given in Table 5.1, and b) compare with the fabrication tolerance.	In 6.3.1.2, general information on the structural and geometric imperfections used to derive the buckling curves should be provided in a dedicated clause. Regarding the geometric imperfection the magnitude of *L*/1000 should be clearly stated.	3, - rules for tolerances can be found in EN 1090 - background information belongs into a commentary

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BE 28		6.3.1.2 in relation with 2.4.2(2)		te, ed	The tolerance on the initial bow indicated in EN 1090 is *L*/750. It exceeds significantly the initial bow *L*/1000 implicitly assumed to establish the buckling curves (see above). It is however expected (see 2.4.2(2)) that effects of fabrication tolerance on the carrying capacity of members are covered by the application rules (more especially by the European buckling curves). Thus an unacceptable inconsistency exists between EN1090 and EN1993-1-1.	Either Give due justification of the inconsistency between EN1090 and EN1993-1-1 or Remove the inconsistency by: • Either specifying the fabrication tolerance (given in EN1990) equal to *L*/1000 and keep §6.3.1.2 as it is presently, • Or, alternatively, letting the fabrication tolerance as it is in EN1990 (*L*/750) under the reservation that values of the imperfection parameter α given in Table 6.1 are increased accordingly. <u>Notes:</u> 1. A simple mean to evaluate the "new" α values does exist (See: *Rondal J. and Maquoi R.: Le flambement des colonnes en acier. Notice de la Chambre Syndicale des Fabricants de Tubes d'Acier, 1980*). 2. Take care that increasing values of parameter α will result in a penalty on the carrying capacity.	see above
BE 29		6.3.1.2(2)	Tables 6.1 and	ed	Originally buckling curve a_0 was established for hot finished hollow	In Table 6.2, allocate • Buckling curve a_0 to hot finished	

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			6.2		sections made of high strength steel, with a L/1000 characteristic initial bow magnitude but without residual stresses. See: [*1] 2nd International Colloquium on Stability of Steel Structures, Introductory Report, Liège 1977*, and *[2] European Convention for Constructional Steelwork: Manual on Stability of steel structures. ECCS, Brussels, publ. n°22, 1976* At that time, the index *0* in a_0 meant the (almost) absence of residual stresses. Also "high strength" at that time meant smaller strength than the same wording used today. In the EN1993-1-1, it is especially referred to curve a_0 for rolled sections made of S460 steel grade though such sections are well affected by residual stresses (That is clearly but indirectly reflected through the magnitude of the equivalent imperfection relative to curve a_0 : L/300 or L/350 in Table 5.1). The fact that a_0 can be allocated to sections with residual stresses is not disputed; it is simply the consequence of peak values of the residual stress-to-yield stress ratio significantly less for	hollow sections (as it is presently) • Where necessary, buckling curve a_{hs} (with, for instance, index *hs* for "high strength") – instead of a_0 – to rolled sections made of S460 grade. In Table 6.1, characterize both a_0 and a_{hs} curves by a same value of the α parameter.	6

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					S460 grade than for mild steel. It remains that the physical meaning of curve a_0 is henceforth biased.		
BE 30		6.3.2.2 and 6.3.2.3			6.3.2.2 and 6.3.2.3 are a set of more confusing than strictly useful documents. • Two methods coexist; they address respectively general use and a restricted field of application. In contrast with what should be expected, both methods are not compatible whatever the choice made of the values of $\lambda_{LT,0}$ and β in 6.3.2.3(1) NOTE. • Discrepancy and confusion are mainly due to the fact that 6.3.2.2 and 6.3.2.3 do not refer to a same analytical expression of the LTB curves and a same selection table for these latter. While equ (6.56) has clearly a physical background, equ (6.57) has not; it simply results from an arbitrary modification of equ (6.56) governed by the will to get a larger plateau length $\lambda_{LT,0}$; but that trick needs to be counterbalanced by less favourable LTB curves. • Undoubtedly a plateau length of 0,2 in the LTB buckling curves is fully	• Shorten the heading of 6.3.2.2 as follows: "6.3.2.2 Lateral torsional buckling curves " • Keep 6.3.2.2(1) as it is. • Keep 6.3.2.2(2) as it is but remove the NOTE in 6.3.2.2(2) so as to make the detailed procedure normative. • Consistently the headings of Table 6.3 and Table 6.4 become: " Table 6.3: Values for imperfection factors for lateral torsional buckling curves " " Table 6.4: Selection of lateral torsional buckling curves " and the sentence below Table 6.3 writes: " Buckling curves are given in Table 6.4." • Move the clause 6.3.2.3(2) – with the NOTE made normative – as an	5, decided new version in SC3, see N1898

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					supported by the results of experiments conducted all around the world on specimens without lateral restraints at the beam ends ("fork" support conditions) (see *Sedlacek, G., Ungermann, D., Kuck, J., Maquoi, R., Janss, J.: Eurocode 3 - Part 1, Background Documentation Chapter 5 - Document 5.03 (partim): Evaluation of test results on beams with cross sectional classes 1-3 in order to obtain strength functions and suitable model factors. Eurocode 3 – Editorial Group, 1984).* That being, it is agreed that the presence of such restraints is likely to increase the plateau length beyond 0,2. However, a realistic evaluation of the effects of restraints is often all but easy so that some guidance for a simplified approach is welcome. • What should distinguish 6.3.2.2 and 6.3.2.3 is less the scope of application (as it is specified through the respective headings of 6.3.2.2 and 6.3.2.3) than the effects which are accounted for when calculating the elastic LTB critical moment M_{cr}. • Heading of Table 6.4 is unappropriate; it should address the selection of the LTB curves.	additional clause in 6.3.2.2. • In 6.3.2.2 add one clause: " For beams or parts of beams in buildings, made of rolled sections or equivalent welded sections and the ends of which are more than fork support conditions because of restraints at their fully braced ends, a slenderness ratio $\lambda_{LT,0}$ up to 0,4 (instead of 0,2) may be allowed under the condition that the effects of these restraints are disregarded when calculating M_{cr}. " • Remove the whole section 6.3.2.3. • Numbering of 6.3.2.4 becomes 6.3.2.3.	

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					• 6.3.2.2(3) refers to Figure 6.4 and is normative; in contrast, reference to the same figure below Table 6.3 is introduced as "recommendation". That results in inconsistency. The proposal is to reduce 6.3.2.2 and 6.3.2.3 to a single section in which the design procedure has a scientific and physical background , but which opens the way to a larger plateau length of the LTB curves, to consider effects that are possibly not accounted for when calculating M_{cr}.		
BE 31		6.3.2.2 and 6.3.2.3		ed	According to 6.3.2.3, the (alternative) method applicable to rolled sections and equivalent sections may take benefit of the *f* factor that has a favorable effect on the penalty factor χ_{LT}. The general method 6.3.2.2 may not while it is yet more conservative than 6.3.2.3. That discrimination is not acceptable. See *N. Boissonnade, R. Greiner, J.P. Jaspart, J. Lindner: "Rules for member stability in EN1993-1-1. Background documentation and design guidelines", ECCS TC8 publication N°119, Bruxelles, 2006.*	Introduce the *f* factor so that both methods 6.3.2.2 and 6.3.2.3 may take advantage of it.	5, see above

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BE 32		6.3.2.3(1)				Il convient de considérer les valeurs de $\bar{\lambda}_{LT,0}$ et de β respectivement égales à 0,2 et 1,0 si M_{cr} est déterminé en considérant les propriétés de la section transversale brute et en tenant compte des combinaisons des actions, de la distribution réelle des moments et des restreintes. Pour les poutres de bâtiments avec des restreintes, il convient d'adopter les valeurs 0,4 et 0,75 respectivement pour $\bar{\lambda}_{LT,0}$ et β, à condition que les restreintes soient totalement ignorées pour la détermination de M_{cr}. The values of $\bar{\lambda}_{LT,O}$ and β are equal to 0,2 and 1,0 with M_{cr} determined on the basis of gross cross sectional properties and due account taken of loading combinations, the real moment distribution and the lateral restraints. Alternatively, for beams with restraints in buildings values of $\bar{\lambda}_{LT,O}$ and β equal to 0,4 and 0,75 may be adopted provided that these restraints are fully disregarded in the determination of M_{cr}.	5, see above
BE 33		6.3.3		te, ed	Clearly 6.3.3 addresses combined bending and axial compression. What about combined bending and axial tensile force?	Either • Implement the code with clauses addressing combined bending and tension; or • Write sufficient guideline so as to prevent from extrapolating (by simply changing the sign of N) the rules of 6.3.3 to bending and tension.	3 Not available so far N (tension) has to be neglected N (compression) is clearly addressed in the headline

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Which clauses would benefit from improvements in clarity?							
BE 34		All		general	Strict requirements imposed by CEN for the drafting of Eurocodes frequently prevent from producing self explaining design rules. Many designers complain about the lack of clarity of many specifications.	Basic and rather condensed commentaries should be prepared to serve as companion in an appendix to the code properly.	3 not a task of the code, maybe in a design guide ECCS?
BE 35		Annexes A and B			Methods described in these Annexes addressed a “member” to be extracted from the structure. Guidance should be given on how to extract that member.	Material is available (See *ECCS 119: New Design Rules in EN1993-1-1 for Member Stability, 2006*)	3, reduction of volume
Where should the scope of the EN be extended?							
BE 36		6 (mostly)		general	The code mainly addresses steel structures made of members with uniform section. As a consequence, L little attention is paid to the members made of built-up welded sections with (linearly) variable depth – that is nowadays an excellent solution in the field of industrial building. Regarding the stability of such members, the general method depicted in 6.3.4 is proposed in the code but some more guidance should be given to make the latter applicable.		4

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BE 37		In relation with 5 and 6 (mostly)		general	For aesthetical and technical reasons, girders of uniform depth but with large web openings are frequently used. The code remains silent on such topic. Clauses relative to the design of such girders with *uniformly distributed* web openings over the length should be gathered possibly not in the main part of the code but in an Annex (as done in the ENV1993-1-1). Also some guidance regarding the stiffening of *localized* web openings would be useful.		6
BE 38		6.2.7		te, ed	In ENV1993-1-1 torsion was mostly ignored. In EN1993-1-1, torsion is present but all the questions raised by meticulous designers are not answered. It is paradoxical that direct stresses induced by warping torsion are just mentioned under the heading “Torsion” and are never referred to in sections where direct stresses (or their resultants) are the main concern.		6
BE 39		6.3.3		te, ed	Stability verifications under combined bending and axial force suppose implicitly that the axial force is a *compression* force. Combined bending and axial *tensile* force is not at all treated.	Prepare additional clause(s) providing rules/guidelines for the design of members subjected to combined bending and *tensile* axial force.	3, so far not available

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BE 40		6		te, ed	The main results of elastic structural stability are supposed to be well-known by designers. It should be but it is not. It is a matter of fact that some elementary principles/formulae are largely unknown or misunderstood, such lateral torsional buckling (with all the parameters in the phenomenon), torsional buckling and, almost, flexural torsional buckling. Those results are not subject for standardization. But simply the conversion of the symbols used in the bibliographical references relative to elastic instability to the symbols used in the Eurocodes is risky. Also managing all the symbols is sometimes complex.	Relevant material with main formulae of elastic stability should be reminded, and written with the symbols used in EN1993-1-1, in an informative annex.	3, the aim is to reduce the volume but check of all symbols

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Where could the EN be shortened?							
BE 41		All		General	It is paradoxical to hear that the "code" is too voluminous while designers comply that many clauses need to be clarified or further commented on.	Shortening the EN should be accompanied with the preparation of Commentaries which are a separate part of the code properly.	3, commentaries should not be a part of the code but given separatly
BE 42		All		General	Presently, the code opens the way to plastic 1st or 2nd order plastic or elastic analysis. It is a matter of fact that plastic analysis is rarely practiced and should only be contemplated for rather special purposes (justifying a structure for updated higher loads, explain and justify accidents or structural collapse, single storey industrial buildings in UK, ...)	Make the normative part of EN1933-1-1, beyond section 5, focus on elastic 1st or 2nd analysis only. Remove all the clauses addressing aspects linked with plastic analysis of statically indeterminate structures with the consequence of less voluminous code properly but improved clarity and readability. Aspects linked with plastic analysis might possibly be collected in an informative Annex.	3, option plastic analysis should be kept as partially used also in everyday life (purlins)
BE 43		6.3.2		ed	6.3.2.2 and 6.3.2.3 are altogether redundant and confusing. Section 6.3.2.4 is an efficient way for preliminary investigation of the location of full restraints; it looks more as a conceptual	Section 6.3.2 is likely to be significantly shortened (see previous proposal for fusing 6.3.2.2 and 6.3.2.3 into a single section).	5, new solution for 6.3.2.2 + 6.3.2.3 3, 6.3.2.4 is widley used in e.g. bridge design or elsewhere where rolled sections are not usual

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					than as a proper design rule.		
		BB3		ed	Looks more as “cooking recipes” then as duly demonstrated design rules.	Remove BB3 in its whole.	2
Are there any clauses whose application leads to uneconomic construction?							
BE 44		General		general	As such the question is almost irrelevant. By nature, the clauses are hopefully the best ones at present time. The question should be: are you of the opinion that some clauses could be improved – while remaining simple and, of course, correct – so as to get more economical construction? Very few places - where specifications are somewhat arbitrary – give opportunities for relaxing supposed too strict requirements.		No comment
BE 45		5.2.2(7)	b)	ed	The last sentence could lead to an uneconomic design by disregarding to consider a more favorable buckling length.	Write the last sentence as follows: … This verification should take account of end moments and forces from the global analysis of the structure, including global second order effects and global imperfections (see 5.3.2) when relevant and be based on a buckling length corresponding to a non-sway mode. Conservatively and for sake of simplicity this buckling length may be taken to the	2, see BE11

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						system length.	
BE 46		6.4.4	Table 6.9	te, ed	Maximum spacing of interconnections given in Table 6.9 looks particularly severe compared to the North American practice.	Revisit the subject in order to possibly increase the maximum spacing values given in Table 6.9.	4 See FR37
Are there any clauses whose application necessitates excessive design effort?							
BE 47		All		general	No designer is still working by full hand calculations. He is thus supposed to use commercial software – at least for global analysis – and almost Excel sheets of his fabrication for verifications. For the latter, his personal check tools must of course be adapted to the design rules of EN1993-1-1; this adaptation is required but the intellectual investment is made once. What may look complex and lengthy for a pure by hand calculation becomes very fast and the computing time irrespective of the complexity of the check. With that in mind, what does the question mean?		no comment
BE 48		5.3.2(11)		ed	That clause looks more as engineering science than engineering art. It is hardly consistent with the approaches to which it is given as an alternative; indeed, in a given situation,	In Belgium, it is specified that this clause is only applicable for bow imperfections of individual elements of a structure.	6

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					the respective magnitudes of the equivalent geometric imperfection will not be the same. Attractiveness and correct practicability of that clause by regular designers are almost questionable.		

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