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ARTICLE

Structural member stability verification in the new Part 1-1 of the second generation of Eurocode 3

Part 1: Evolution of Eurocodes, background to partial factors, cross-section classification and structural analysis

This two-part article gives an overview of the developments of the structural member verification in prEN 1993-1-1:2020 "Eurocode 3: Design of steel structures – part 1-1: General rules and rules for buildings", one of the second generation of Eurocodes. These developments were undertaken by Working Group 1 (WG1) of Subcommittee CEN/TC250/SC3 and by Project Team 1 (SC3.PT1) responsible for drafting the new version of EN 1993-1-1. In the past, WG1 collected many topics needing improvement, and the systematic review conducted every five years also yielded topics needing further development. Based on this, the current version of EN 1993-1-1 has been developed into a new draft version prEN 1993-1-1:2020 enhancing "ease of use". The technical content of this new draft was laid down at the end of 2019. Many improvements to design rules have been established with respect to structural analysis, resistance of cross-sections and stability of members. This two-part article focuses on member stability design rules and deals with the basis for the calibration of partial factors, the introduction of more economic design rules for semi-compact sections, methods for structural analysis in relation to the appropriate member stability design rules, new design rules for lateral torsional buckling plus other developments and innovations. This first part of the article primarily serves to explain the general background to the European Commission Mandate M/515 that led to the further evolution of the Eurocodes and to illustrate the developments in prEN1993-1-1:2020 that pertain to new material grades, partial factors, cross-sectional classification and structural analysis. These form the necessary background to the changes to member buckling design rules, which are treated more specifically in the second part.

Keywords steel structures; structural stability; Eurocode 3; standardization; flexural buckling; lateral torsional buckling; cross-sectional capacity

1 Structural Eurocodes and EN 1993-1-1 – from past to future

1.1 Development of the second generation of Eurocodes

The Eurocodes were introduced in 2010 in accordance with the directives of the European Union. They comprise 10 standards in 58 parts and replaced the national standards in many countries. This was an important first step towards harmonized European standardization in the field of structural engineering. The Eurocodes have thus been in force now for some time in the different member countries of the Comité Européen de Normalisation (CEN) and they are used in practice by structural engineers to design buildings, bridges, etc. Working with the Eurocodes and gaining experience with them gave rise to proposals for change and possible improvement.

Currently, the further evolution of the Eurocodes is being carried forward within the scope of Mandate M/515 [1], which was agreed between the European Commission and CEN in December 2012. Among other things, the mandate is intended to lead to the extension of the Eurocode rules in terms of new materials, products and construction methods, improve the practical use for day-to-day calculations and achieve better harmonization by reducing the number of Nationally Determined Parameters (NDPs).

The mandate started in 2015 and will last until early 2022. However, publication – including formal procedures, such as a so-called CEN Enquiry – may last up to 2024, perhaps even longer when accounting for the transfer to national rules and legislation. Fig. 1 shows the timetable for the revision and further evolution of the Eurocodes.

There are two main sources for the development of the second generation of the Eurocodes: the general revision and maintenance based on the so-called "Systematic Review" and further developments within the scope of Mandate M/515. As part of the "Systematic Review", which usually takes place every five years according to CEN rules, comments and proposals for improvements to single Eurocodes and their parts were requested and put forward by the National Standardization Bodies (NSBs, e.g. DIN, BSI, AFNOR, NEN, etc.). These have to be dealt with by the subcommittees, i.e. by Subcommittee CEN/TC250/SC3 (TC = Technical Committee, SC = Subcommittee) in the case of Eurocode 3 on steel structures. Apart from these proposals for change and possible improvement, the NSBs were also asked to propose (i) ways of enhancing the ease of use of the standard, (ii) improvements in terms of compactness of the code and (iii) extensions to the desired scope. Moreover, the NSBs have identified rules that are inefficient for the assessment or lead to uneconomic design results.

Furthermore, so-called Project Teams (PT), responsible for the technical work and the development of the drafts, have been given a contract within Mandate M/515 following the procedure adopted for the transfer from the

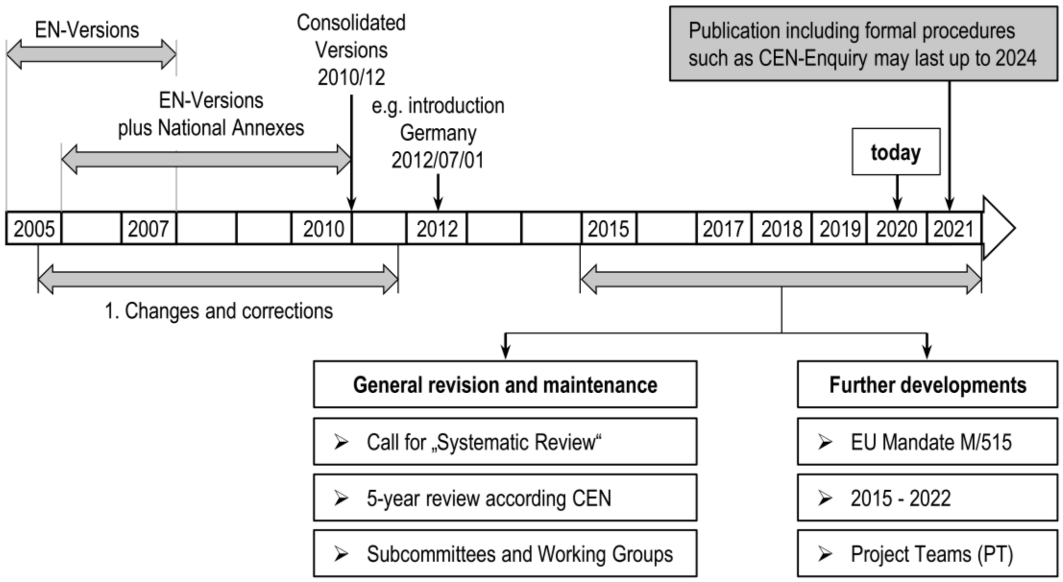

Fig. 1 Planned timetable for the revision of the Eurocodes

ENV versions to the EN versions of the Eurocodes in the late 1990s and early 2000s. Each of these Project Teams consists of five or six members who were chosen in a competitive selection process with open applications, leading to a good representation of different countries and backgrounds (industry and academia).

The CEN/TC 250 work programme has been split into four overlapping phases. These interrelated phases ensure an efficient work programme and enable effective management of the interdependencies of the activities. Phase 1 comprises general parts of the work programme. Other activities in later phases depend on these parts, for reasons of overall coordination or technical scope or because they are essential for achieving the target dates for the delivery of the second generation of Eurocodes.

1.2 Procedure for the development of Eurocode 3

Fig. 2 shows the approach chosen for revising and harmonizing Eurocode 3. The approach was scheduled at a dedicated meeting in Stuttgart in April 2010. Significant contributions to the work were made by the so-called Working Groups (technical working groups, called Evolution Groups until March 2014) which attend to the work of SC3 in scientific and technical terms regarding a specific part of Eurocode 3. In the meantime, SC3 has agreed to apply the same procedure (Fig. 2) for the development of technical changes and their establishment in the new version of Eurocode 3. These agreed “amendments” are put into the “basket” for the time when the Project Team starts working, and are then implemented in the new version. The SC3 Working Groups have the important task of following and advising on the work of the Project Teams.

Among the several Working Groups (WG) of SC3, WG1 is responsible for the basic part: EN 1993-1-1 “Eurocode 3: Design of steel structures – Part 1-1: General rules and

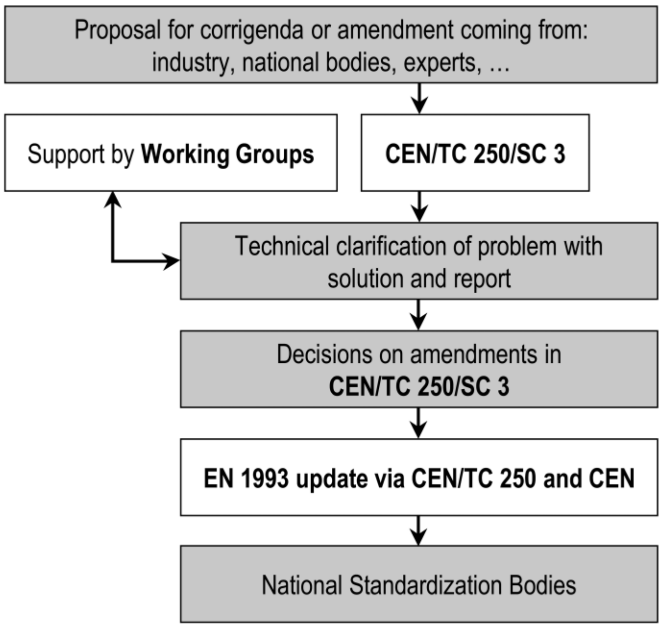

Fig. 2 Procedure for the revision of Eurocode 3

rules for buildings” [2]. The current structure of Eurocode 3 has largely been retained (Tab. 1) with some modifications. Tab. 1 also indicates the Working Groups responsible for each individual standard.

The modifications to the structure of Eurocode 3 are as follows: The current content of EN 1993-1-12 regarding additional rules for extending EN 1993 up to steel grades S700 has been redistributed over the relevant other parts of Eurocode 3 (since the application of these parts has been extended to high-strength steel). Hence, the current version of EN 1993-1-12 could be withdrawn. However, SC3 decided to keep EN 1993-1-12 and adjusted its scope to include high-strength steel grades up to S960. This activity is not within the scope of the mandate given by the EU, but will be finalized later when sufficient knowledge

Tab. 1 Structure of Eurocode 3 on steel structures and SC3 Working Groups responsible

Part of Eurocode 3	Type	Topic	Working Group
EN 1993-1-1	General parts	General rules and rules for buildings	WG1
EN 1993-1-2		Structural fire design	WG2
EN 1993-1-3		Supplementary rules for cold-formed members	WG3
EN 1993-1-4		Stainless steels	WG4
EN 1993-1-5		Plated structural elements	WG5
EN 1993-1-6		Strength and stability of shell structures	WG6
EN 1993-1-7		Plate assemblies with elements under transverse loads	WG7
EN 1993-1-8		Design of joints	WG8
EN 1993-1-9		Fatigue	WG9
EN 1993-1-10		Material toughness and through-thickness properties	WG10
EN 1993-1-11		Design of structures with tension components	WG11
EN 1993-1-12		Additional rules for steel grades up to S960	WG12
EN 1993-1-13		Steel beams with large web openings	WG20
EN 1993-1-14		Design assisted by finite element analysis	WG22*
EN 1993-2	Application parts	Steel bridges	WG13
EN 1993-3		Towers, masts and chimneys	WG14
EN 1993-4-1		Silos	WG15
EN 1993-4-2		Tanks	WG16
EN1993-5		Piling	WG18
EN 1993-6		Crane supporting structures	WG19
EN 1993-7		Design of sandwich panels	WG21

* formerly AHG FE

and experience have been gained. EN 1993-1-13 is a new part on steel beams with large web openings (e.g. cellular and castellated beams). The current draft has mainly been developed within the scope of the mandate as a special task and by a Project Team of CEN/TC250/SC4 responsible for steel-concrete composite construction. Moreover, EN 1993-1-14 is a new part on design assisted by finite elements, which is intended to facilitate the wider use of finite element analysis in the design of steel structures in the future. Initially, an Ad-Hoc-Group Finite Elements (AHG FE), which consisted of members from various SC3 Working Groups, developed a first draft, primarily transferring rules from other parts of Eurocode 3 such as Annex C of EN 1993-1-5 to this general part. In the meantime, a separate, dedicated Working Group WG22 has been set up to deal with the drafting of this standard. Furthermore, the current parts EN 1993-3-1 on masts and towers and EN 1993-3-2 on chimneys have been merged into one part, EN 1993-3, thus avoiding the overlap in the content of the current two parts. In addition, a new part EN 1993-7 on the design of sandwich panels will be integrated into the framework of Eurocode 3. The principal distinction between general parts grouped in the various sub-parts of EN 1993-1 and the application parts found in EN 1993-2, EN 1993-3, etc. has proved useful and will be retained. General parts contain design rules applicable to all types of structure, while application parts contain design rules for a specific type of structure.

Thirteen tasks pertaining to the 20 parts of Eurocode 3 were defined within the mandate (Tab. 2). The technical content of these tasks was developed as so-called Project Proposals. This involved both close collaboration with the convenors of the respective Working Groups and coordination within SC3. Each task is handled by a dedicated Project Team. EN 1993-1-1 and EN 1993-1-8 are the most general and basic parts of Eurocode 3. Various issues must thus be harmonized with other parts of the standard. Parts 1-1 and 1-8 are therefore part of phase 1 of the mandate. Furthermore, four SC3 tasks, most of them dealing with stability, are assigned to the early phase 2 of the mandate. The material-specific parts of Eurocode 3, e.g. EN 1993-1-4 and EN 1993-1-10, are assigned to phase 3 of the mandate, while phase 4 primarily covers the application parts, e.g. EN 1993-2 for steel bridges.

Tab. 2 Tasks of Mandate M/515 concerning Eurocode 3 and relevant Project Teams

Task ref.	Task phase	Corresponding part of EN 1993	Task name
SC3.T1	1	EN 1993-1-1	Design of sections and members according to EN 1993-1-1
SC3.T2	1	EN 1993-1-8	Joints and connections according to EN 1993-1-8
SC3.T3	2	EN 1993-1-3	Cold-formed members and sheeting – revised EN 1993-1-3
SC3.T4	2	EN 1993-1-5	Stability of plated structural elements – revised EN 1993-1-5
SC3.T5	2	EN 1993-1-6, EN 1993-1-7	Harmonization and extension of rules for shells and similar structures – revised EN 1993-1-6 and EN 1993-1-7
SC3.T6	2	EN 1993-1-2	Fire design of steel structures – revised EN 1993-1-2
SC3.T7	3	EN 1993-1-4	Stainless steels – revised EN 1993-1-4
SC3.T8	3	EN 1993-1-9	Steel fatigue – revised EN 1993-1-9
SC3.T9	3	EN 1993-1-10	Material and fracture – revised EN 1993-1-10
SC3.T10	4	EN 1993-2, EN 1993-1-11	Steel bridges and tension components – revised EN 1993-2 and EN 1993-1-11
SC3.T11	4	EN 1993-3	Consolidation and rationalization of EN 1993-3
SC3.T12	4	EN 1993-4	Harmonization and extension of rules for storage structures – revised EN 1993-4-1 and EN 1993-4-2
SC3.T13	4	EN 1993-5, EN 1993-6	Evolution of existing parts of EN 1993 not included in the other parts – revised EN 1993 -5, -6, [-1-12, -4-3]*

* These parts are not being revised.

1.3 Development of Eurocode 3: Design of steel structures – Part 1-1: General rules and rules for buildings

Working Group TC250/SC3/WG1 is responsible for the basic part EN 1993-1-1. This group worked on several technical topics to improve this part of Eurocode 3, resulting in approved and accepted amendments that were later adopted by the Project Team SC3.T1 responsible for drafting the new version of EN 1993-1-1 under Mandate M/515. The work done by SC3.T1 needed the approval of SC3. On behalf of SC3, WG1 monitored the work done by SC3.T1, but final decisions on the content of the next version of EN 1993-1-1 were made by SC3. Project Team SC3.T1 completed its work by the end of June 2018 and delivered a draft for the next version of EN 1993-1-1. Subsequently, SC3 developed and approved some further modifications to the text, resulting in the technical content of prEN 1993-1-1 [3] being fixed in December 2019 in order to be sent to the formal "CEN Enquiry" procedure.

Several basic modifications to EN 1993-1-1 were already decided in SC3 at an early stage. For example, pursuing an improved "ease of use", SC3 decided not to include extensive information on elastic critical buckling forces, buckling lengths and elastic critical lateral torsional buckling moments, which are necessary when applying the buckling design rules. Inclusion of this information would lead to EN 1993-1-1 looking a bit like a textbook on applied mechanics, which is not desirable in a design code. Instead, it was decided to draft and establish a Technical Report on these same subjects. Work is in progress on the Technical Report, CEN/TR 1993-1-103 "Eurocode 3 – Design of steel structures – Part 1-103: Elastic critical buckling of members" [4], which will contain verified and accepted solutions for elastic critical buckling.

In order to avoid informative annexes with an unclear status, some relevant content of the original informative annexes in EN 1993-1-1 has been shifted to the main text or transferred to other places. With respect to the informative annexes in EN 1993-1-1, the following decisions were made within SC3:

- Currently, EN 1993-1-1 contains two methods for beam-column design: "Method 1" defined by the interaction coefficients of Annex A, and "Method 2" defined by the factors in Annex B. Since the rules for the application of Annex B have been developed further over the last decade and now have a wider scope, only "Method 2" has been retained in the revised code, thus avoiding alternative design rules for the same phenomenon. The content of Annex B has thus been integrated into the main text. "Method 1" and Annex A have been transferred to a Technical Specification "CEN/TS 1993-1-101 Eurocode 3 – Design of steel structures – Part 1-101: Alternative interaction method for members in bending and compression" [5] to allow for the continued use of these rules – which is relevant for the countries that have traditionally chosen to use Annex A exclusively.
- Annex AB.1 contains clauses on how to perform a structural analysis when taking into account material

Tab. 3 Content and sections of current EN 1993-1-1 [2]

Foreword	
1	General
2	Basis of Design
3	Materials
4	Durability
7	Structural analysis
8	Ultimate limit states
9	Serviceability limit states
Annex A [informative]	Method 1: Interaction factors kij for interaction formula in 6.3.3(4)
Annex B [informative]	Method 1: Interaction factors kij for interaction formula in 6.3.3(4)
Annex AB [informative]	Additional design provisions
Annex BB [informative]	Buckling of components of building structures
Annex C [normative]	Selection of execution class

Tab. 4 Content and sections of prEN 1993-1-1:2020 [3]

European Foreword	
Introduction	
1	Scope
2	Normative references
3	Terms, definitions and symbols
4	Basis of Design
5	Materials
6	Durability
7	Structural analysis
8	Ultimate limit states
9	Serviceability limit states
10	Fatigue
Annex A [normative]	Selection of execution class
Annex B [normative]	Design of semi-compact sections
Annex C [normative]	Additional rules for uniform members with mono-symmetric cross-sections and for members in bending, axial compression and torsion
Annex D [normative]	Continuous restraint of beams in buildings
Annex E [informative]	Basis for the calibration of partial factors

non-linearities and has been moved to the main text. Furthermore, Annex AB.2 on load arrangements for continuous floor beams has been removed, since it is a clause on loading, belonging to a different part of the Eurocodes. These two actions make Annex AB superfluous.

– Annex BB.1 on elastic flexural buckling in lattice structures has been moved to Technical Report CEN/TR 1993-1-103. Annex BB.2 on restraint stiffness has been made normative. Annex BB.3 on the stable length method for lateral torsional buckling is of limited use in all countries using the Eurocode and has therefore been omitted.

One general change concerns the structure of the different sections (chapters). This change is obligatory for all new Eurocodes. To understand the correspondence between the current EN 1993-1-1 [2] and the future version as given in the draft prEN 1993-1-1:2020 [3], the list of contents is presented for both of them in Tab. 3 and Tab. 4. As an obvious change, it should be noted that section 2 "Normative references" and section 3 "Terms, definitions and symbols" have been inserted, leading to a renumbering of the following sections.

Regarding Part 1-1 of Eurocode 3, the timetable is as follows: As of December 2019, about a year is required for a first round of editing, translation and review, to be followed by another round of about six months ending about mid-2021. In the second half of 2021, EN 1993-1-1 goes to the NSBs for final translation and incorporation into national building legislation and for developing the National Annexes. Publication and implementation are then expected for early 2023. This timetable more or less holds for all Eurocode parts in phase 1.

This article focuses on the member stability design rules in prEN 1993-1-1:2020. Section 2 below explains the basis for the calibration of partial factors for member stability design rules. The need to account for the scatter of the material properties and the dimensions of the cross-sections is also discussed. Section 3 deals with the introduction of new and more economic design rules for semi-compact cross-sections and their implications for the member stability design rules. Section 4 explains how the different methods of global analysis allowed by Eurocode 3 relate to the member stability checks. Furthermore, this section provides information on the use of imperfections in global analysis. New design methods for lateral torsional buckling are dealt with in section 5. Finally, section 6 mentions some further developments and innovations in prEN 1993-1-1:2020.

2 Basis for the calibration of partial factors

2.1 General

Construction is a safety-critical activity [6], hence, requires regulation and strict verification procedures. As the accepted risk is very low (see [7]), the consequences of inappropriate choices of the safety level in design verifications may take some time to result in dramatic consequences that finally lead to the enforcement of corrective measures. Despite the enormous work that went into the development of the first generation of Eurocode 3 (2005) (EN version) [2] and the wealth of proven experience it incorporates, Eurocode 3 combines a very large number of design rules that evolved over many decades of extensive research work, and its safety level is not necessarily uniform across the various design rules and not even within a single design rule. In addition, the continuous push towards more economical solutions coupled with the systematic broadening of the limits of application of the design rules also contribute to the need to reassess the safety level. Typical examples are the new steels that are continuously being introduced for structural applications, with different material properties, residual stress patterns and weldability as well as new, challenging structural systems.

In the past, several reliability assessments were carried out, e.g. in research projects [8] and [9], whose aim was to justify the partial factors used in Eurocode 3. More recently, the intention of the SAFEBRICTILE project [10] was to contribute to harmonizing the reliability level of design rules for steel structures covering modes driven by ductility, stability and fracture. The project provided an objective and consistent safety assessment procedure for the various failure modes that are relevant for steel structures. It also recommended statistical distributions of the relevant basic variables, which were collected continuously during the project in a database of steel properties, further included in Annex E in [3].

Finally, most recently there has been a trend towards the validation of the Eurocode rules for high-strength steel (HSS). Several projects were carried out, HILONG [11], HSS-REF [12], HITUBES [13], RUOSTE [14], STROBE (ongoing), which studied several aspects of HSS. This increased knowledge led to the inclusion of steel grades up to S700 in the main text of EN 1993-1-1.

2.2 New Annex E: Basis of the calibration of partial factors

During the development and validation of new design rules for the second generation of EN 1993-1-1, it became apparent that it is necessary to follow a standardized procedure when assessing the reliability level of the implemented design rules, with a uniform set of agreed values for the statistical distributions of the mechanical and geometric properties of European steel products. In order to make the underlying agreed values available to the various national and European standardization bodies in a direct and public manner, it was thus decided to include a new, informative Annex E in prEN 1993-1-1:2020 [3], which – for the first time – directly states which assumptions were used in the determination of the recommended values of the partial factors γ_M. As stated, the intended users of this annex are mainly the national standardization bodies, which are responsible for structural reliability in general and thus for setting the various NDPs in Eurocode 3, with the values of the partial factors γ_M representing one of the most prominent examples. In addition, this annex should provide background information for both steel producers and the users of steel construction products, allowing for the verification of production values with respect to their alignment with the assumptions mentioned, if this is desired. It should be stressed, however, that this standard is not intended for direct use in design, and no conclusions can be drawn directly from it which concern individual products or structures.

The background to the recommended values of γ_{M0}, γ_{M1} and γ_{M2} in prEN 1993-1-1:2020 [3] was developed primarily over the course of the three-year SAFEBRICTILE project, which was financially supported by the European Commission through the Research Fund for Coal and Steel (RFCS) [10]. This project analysed a large number of existing and new design rules in prEN 1993-1-1:2020 with respect to the values of γ_M required for compliance with the reliability levels prescribed by EN 1990 and for consistency throughout failure modes, from ductile to brittle ones.

Data for material and geometrical properties was systematically collected within the scope of SAFEBRICTILE. The aim of this was to provide reliable data for the application of the reliability assessment performed within the project. The data was gathered in a "European database", one of the published outcomes of the project. Data from previous work was also collected in order to increase the data pool.

The aim of collecting data was to attract contributions coming from different industries. In particular, the focus was on steel sections and plates in different steel grades fabricated in 2013 and 2014. During the project, data was also collected from other sources such as coupon tests performed at universities around Europe. These tests enable independent comparison with the results supplied by the steel producers, as the steels tested in the university laboratories were supplied by random producers. Data was also collected from the literature: (i) a collection in [15] that comprised a large amount of tests (7454 coupon test results) obtained between 1996 and 2007 for steel grades S235, S275, S355, S460 and S690; (ii) a large amount of data collected within the framework of the European project OPUS [16] (25425 coupon test results) tested between 2007 and 2010 for steel grades S235, S275, S355, S460.

Based on the data collected, recommendations for the distributions of the material properties were specified and used in the assessment of design rules carried out in SAFEBRICTILE. Log-normal distributions were assumed for the purpose of assessing the partial factors required. The values were finally also included in Annex E of prEN 1993-1-1:2020. A simplified version of this data is reproduced in Tab. 5 for the yield strength f_y as the most important mechanical property and in Tab. 6 for some selected geometrical properties.

Tab. 5 Assumed variability for the yield strength R_{eH}

Steel	$R_{eH,min} = f_{y,nom}$	$R_{eH,mean}/R_{eH,min}$	CoV	$X_{5\%}$ fractile/$R_{eH,min}$	$X_{0.12\%}$ fractile/$R_{eH,min}$
S235	235 N/mm^2	1.25	5.5 %	1.14	1.06
S355	355 N/mm^2	1.20	5 %	1.11	1.03
S460	460 N/mm^2	1.15	4.5 %	1.07	1.00

Tab. 6 Assumed variability for geometrical properties of I- and H-sections

Dimension	b	h	t_w	t_f
mean/nom	1	1	1	0.98
CoV	0.9 %	0.9 %	2.5 %	2.5 %
$X_{5\%}/X_{nom}$	0.98	0.98	0.96	0.95
$X_{0,12\%}/X_{nom}$	0.97	0.97	0.93	0.91

The tables contain the following statistical parameters for the most relevant mechanical (prEN 1993-1-1, Tab. E.1) and geometric (Tab. E.2) properties: mean values (normalized by the nominal values), coefficients of variation (CoV), the 5 % fractile ($X_{5\%}$ value) and the 0.12 % fractile ($X_{0.12\%}$ value). For future users of prEN 1993-1-1:2020, the following two points are of greatest interest:

1. The statistical parameters for the mechanical properties in Tab. E.1 (Tab. 5 of this paper shows an excerpt), in particular the yield strength and tensile strength, show that a significant "statistical overstrength" is present for most common steel grades, with mean values that are significantly higher than the nominal (i.e. minimum guaranteed) values and low scatter. This explains the comparatively low values of the partial factors γ_{M0} and γ_{M1}. For example, for steel grade S355 it was found (on the basis of the data collected) that the average yield strength R_{eH} is 20 % higher than the minimum guaranteed value $R_{eH,min}$ mentioned in the product standard EN 10025, and that the CoV is 5 %.
2. At the same time, the values in Tab. E.2 (Tab. 6 of this paper shows an excerpt) show that the geometric properties must generally be assumed to scatter fairly narrowly around a mean value that corresponds quite accurately to the nominal geometric values, with some values (such as the flange thickness of I- and H-sections) falling, on average, slightly below the nominal value. This means that the geometric dimensions can regularly be found to be slightly smaller than the nominal values assumed in calculation – a fact that needs to be considered and covered by the specified partial factors.

The $X_{5\%}$ and $X_{0.12\%}$ fractiles contained in Tabs. E.1 and E.2 of prEN 1993-1-1:2020 may mainly be used by producers of steel construction products to verify the compatibility of their production statistics with the basic assumptions underlying the recommended values of γ_M in the standard. These fractile values serve as "anchor points" for lower portions of the scatter band of geometric and mechanical properties, which are of course the most relevant properties for determining design resistances. Although the evaluation procedure in EN 1990 Annex D which was used to determine values of γ_M in the SAFEBRICTILE project makes use of the assumption of log-normal distributions of the input parameters, this must not necessarily be the case in real production data. However, the lower portion of the statistical distribution is not particularly sensitive to the shape of the actual histogram of production values. For this reason, it is more accurate and straightforward to verify a given production dataset for its compatibility with Annex E by checking whether these fractile values are exceeded. This methodology is illustrated schematically in Fig. 3. As can be seen, statistical distributions in which both the $X_{5\%}$ and $X_{0.12\%}$ fractiles are shown to exceed the minimum values specified in Tabs. E.1 and E.2 of Annex E can be directly considered to be compatible with the assumptions of the annex. If this is not the case, additional checks and deliberations might become necessary.

Finally, it should be stressed once more that Annex E is intended to have an informative character and should primarily be considered by code committees and building authorities when determining the nationally prescribed values of the partial factors for the application of Eurocode 3, as well as by steel producers for verifying their production statistics. However, it is intended to strengthen further the direct link between steel production requirements and design rule development over the course of the upcoming, additional standardization work to be undertaken at the level of CEN committees TC250/SC3 (Eurocode 3) and TC459/SC3 (steel products). Annex E of prEN 1993-1-1:2020 provides a suitable basis for this work.

3 Classification and semi-compact sections

3.1 Classification of cross-sections

In Eurocode 3, the class of a cross-section is a critical parameter for verifying the cross-section and the member. It may also affect the type of global analysis of the structure (elastic or plastic). Recent studies [17] have shown that limits to the width-to-thickness ratios for internal compression parts need to be modified in order to keep a partial factor equal to 1.0 and reach the reliability level required by the Eurocodes. These studies were based on the results of experimental tests and numerical simulations.

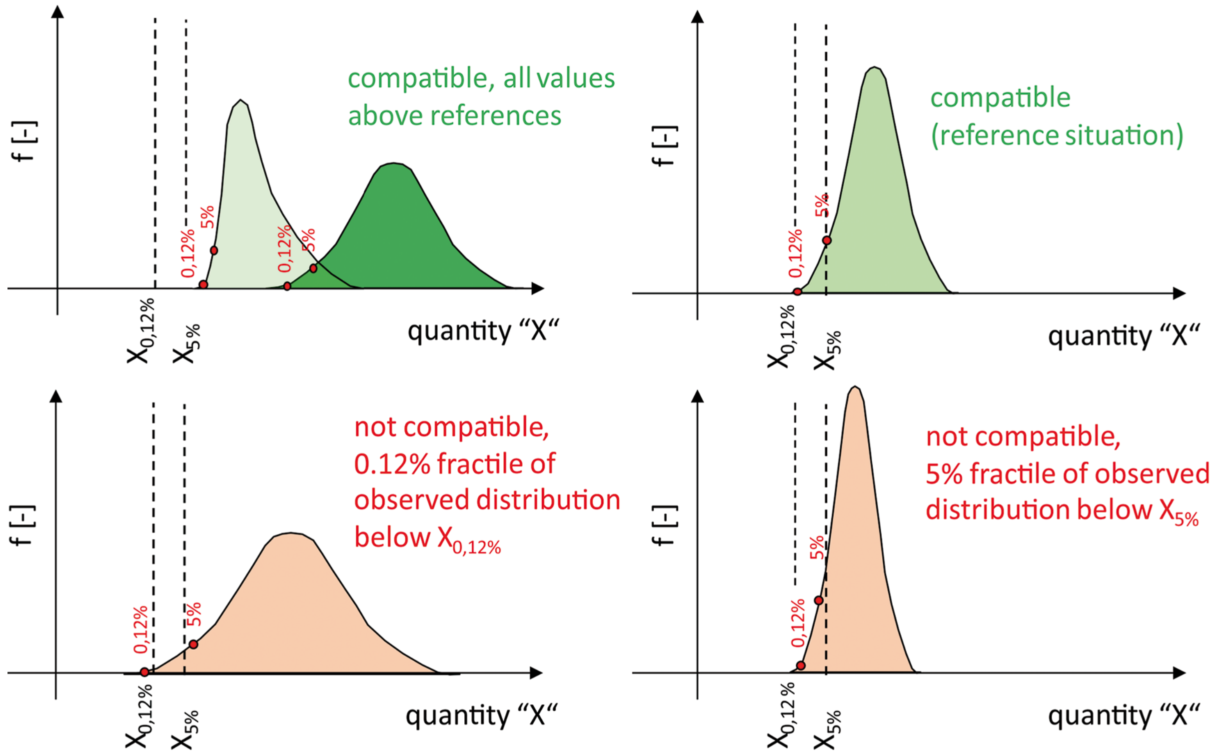

Fig. 3 Schematic representation of the verification procedure for production data for compatibility with Annex E

In EN 1993-1-5 the effective width of the compression part has to be calculated when the relative slenderness of this part is greater than a certain slenderness limit; however, the relative slenderness calculated using the width-to-thickness limits for class 3 given in EN 1993-1-1 does not correspond exactly with this slenderness limit. The new limits for the classification ensure consistency between these two parts of Eurocode 3. For example, for a uniform internal compression part, the limit for the width-to-thickness ratio between classes 3 and 4 is currently 42ε, with material parameter $\varepsilon = \sqrt{235/f_y}$. In EN 1993-1-5 the limit for the relative slenderness is 0.673. This value is obtained for a width-to-thickness ratio equal to about 38ε, which is the new limit for class 3 in this case (see Fig. 4). These changes also provide for better consistency with EN 1993-1-3.

The main changes to the maximum width-to-thickness ratios are given in Tab. 7 for bending and uniform compression. For parts in bending and compression, the expressions have been updated accordingly. The limits for outstand flanges remain unchanged.

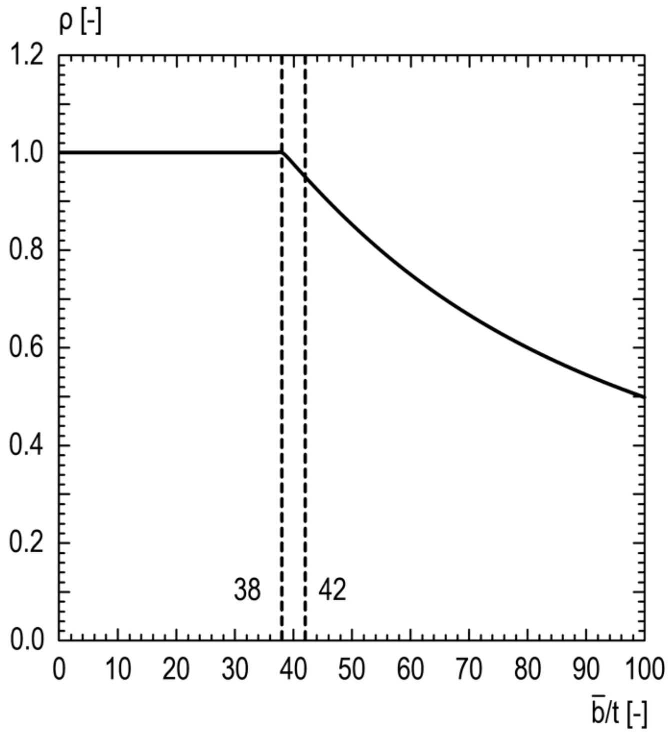

Fig. 4 Reduction curve for an internal compression part under uniform compression for $\varepsilon = 1$ ($f_y = 235$ N/mm^2) according to EN 1993-1-5

3.2 Design of semi-compact sections

The differentiation between cross-sectional classes (1 to 4) is necessary because of the differences in the susceptibility to local buckling of members and sections composed of plated parts with different widths in relation to their thicknesses (c/t ratios) and thus dissimilar abilities to form plastic hinges and reach the full plastic cross-sectional strength. For example, whereas beams with class 1 and 2 cross-sections can reach the full plastic moment capacity in bending, beams with class 4 cross-sections will fail in local buckling before the nominal yield strength is reached. Class 3 sections are defined as the range of sections that falls between these two limit cases. According to the edition of EN 1993-1-1 currently valid, class 3 sections may only be designed using elastic stress distributions, with the yield strength as the highest stress value.

Tab. 7 Evolution of the maximum width-to-thickness ratios for internal compression parts

	Internal part in bending		Internal part in uniform compression	
	EN 1993-1-1	prEN 1993-1-1:2020	EN 1993-1-1	prEN 1993-1-1:2020
Class 1	72 ε	72 ε	33 ε	28 ε
Class 2	83 ε	83 ε	38 ε	34 ε
Class 3	124 ε	121 ε	42 ε	38 ε

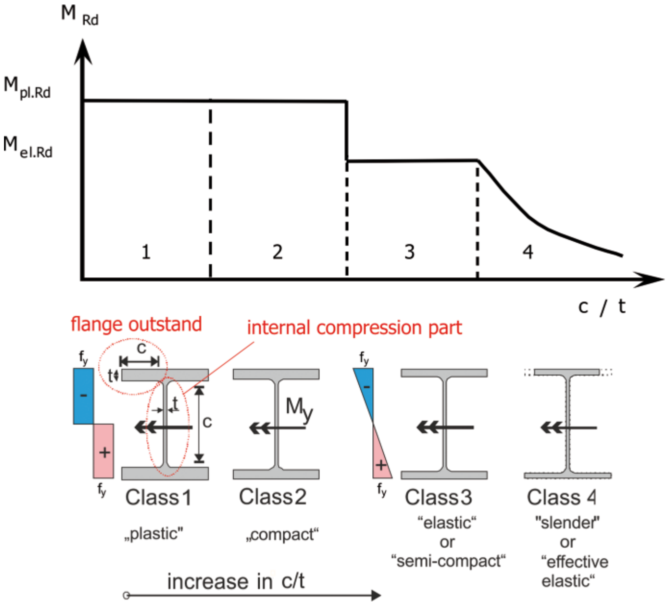

Fig. 5 Schematic representation of the cross-sectional resistance of I-sections in bending as a function of the local, geometric plate slenderness *c/t*; discontinuity at the border between classes 2 and 3

Naturally, this leads to a discontinuity in the definition of strength at the border between classes 2 and 3, see Fig. 5. This discontinuity can be particularly pronounced in the case of I-sections in weak-axis bending.

This discontinuity is not observable in tests and cannot be justified mechanically. The limitation to elastic stress distributions over the cross-section with the yield strength as limit is thus a conservative, quite simplified convention. Furthermore, this is also unable to reflect correctly the behaviour of sections in bending and compression that reach the yield stress on the tensile side before buckling on the compression side.

In order to improve the design outcome for class 3 sections with a standard, doubly symmetric shape (I- and H-sections, square and rectangular hollow sections), a new set of design equations was developed and validated over the course of two EU-RFCS projects: SEMI-COMP [17] and SEMI-COMP+ [18]. The design rules that constitute the outcome of this project were integrated in the new prEN 1993-1-1:2020, in a dedicated Annex B.

The design method of this standard was developed in the SEMI-COMP projects through an extensive physical and numerical test campaign and by calibrating the design proposals to the reliability requirements of EN 1990. In the design rule developed, a simple linear transition of the cross-sectional strength is introduced between the class 2/3 and class 3/4 limits, and then used to determine a linearly interpolated bending strength for either of the two main axes of bending. This simple rule leads to easy application and the possibility of providing dedicated values for the elastic-plastic transition bending resistance $M_{ep,Rd}$ in the form of tables if desired. In the case of combined bending and compression, functions similar to the familiar expressions for the plastic resistance of class 1 and 2 sections are provided in Annex B, thus allowing designers to determine the resistance of the ("semi-compact") class 3 sections for these load cases. Fig. 6 illustrates the various steps needed in the method of Annex B to determine the design resistance of doubly symmetric class 3 sections under compression and bending about both axes.

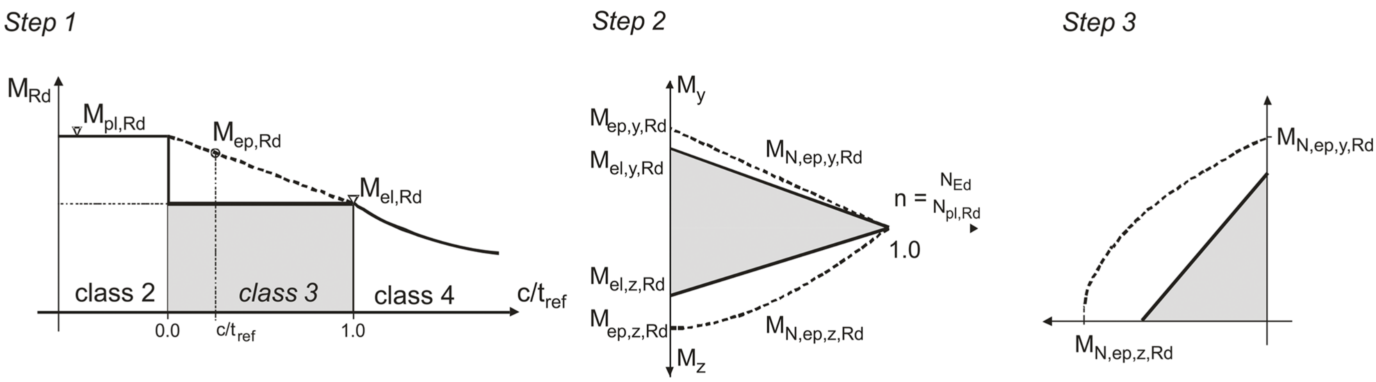

Fig. 6 Schematic representation of the design method for semi-compact (class 3) sections in prEN 1993-1-1:2020, Annex B

At the transition from class 2 to 3, some remaining inconsistencies may be found in cases with relatively small compressive forces, since the method requires an immediate reduction in the bending capacity even for very small compressive forces, while this is not the case in the existing rules for class 2 sections. Furthermore, some small, remaining discontinuities may persist at the border between class 3 and 4. Both residual discrepancies are, however, much smaller than those in the current edition of EN 1993-1-1. Thus, by using the new rules for semi-compact (class 3) doubly symmetric sections in Annex B, significant reserves of strength can be exploited for many load scenarios relevant in practice.

In addition to being used directly in cross-sectional checks, the new rules for semi-compact (class 3) sections may also be used in the member buckling checks as presented in section 8.3.3 of prEN 1993-1-1:2020. In this case the interaction coefficients that apply to plastic (class 1) and compact (class 2) sections may also be applied to semi-compact sections. The only difference in the application of the interaction formulae for the member buckling checks is that the bending resistances $M_{\mathrm{ep,Rk}}$ for the y-y and z-z axes are used instead of the full plastic capacities. As was stated above, these resistance values are calculated very simply through interpolation and may be provided in the form of tables for standardized sections, thus making their use as straightforward as the more familiar elastic and plastic cross-sectional resistances.

In summary, the new cross-sectional resistances for doubly symmetric class 3 sections given in Annex B of prEN 1993-1-1:2020 allow for a more economic and mechanically consistent design of these common sections, both in the cross-sectional verification itself and in the member buckling checks, without unduly increasing the complexity of the design task. Finally, it should be noted that the method is only an alternative to traditional design with elastic cross-sectional capacities, which of course remains valid for all types of cross-section.

4 Structural analysis

4.1 Methods of analysis for ultimate limit state design checks

4.1.1 Structural analysis – changes

The original section 5 "Structural analysis" in EN 1993-1-1 [2] led to a lot of questions and misunderstandings in the past. For the sake of clarity, the new section 7 "Structural analysis" in the draft prEN 1993-1-1:2020 [3] has therefore been completely restructured while largely retaining the same content. Within the framework of the development of the second generation of Part 1-1 of Eurocode 3, it became apparent that engineers in different European countries understand things differently, often due to different traditional approaches. In some cases it was possible to harmonize these different views; in others, more detailed differentiation resulted. The latter ensure that in each country the accustomed way of implementing structural analysis can be continued in the future.

A flow chart has been added to section 7 of prEN 1993-1-1:2020, see Fig. 7. It constitutes an important tool that clearly connects the type of structural analysis and the choice of imperfections with the way of verifying the member resistance at the ultimate limit states. Starting with the original clause 5.2.2(3) in EN 1993-1-1 [2], which describes the three main procedures possible, namely, considering second-order effects and imperfections

a) totally in the global analysis, or
b) partially in the global analysis and partially through individual stability checks of members, or
c) through individual stability checks of equivalent members,

six different methods of analysis have been defined. The choice of method is clearly based on certain decisions concerning the need to consider second-order effects, see section 4.1.2. Further, as a big advantage in view of design practice, the list of methods starts with the simplest case that a first-order analysis suffices (method M0). In addition, the case of lateral torsional buckling is considered more consequently compared with the current edition of EN 1993-1-1.

4.1.2 Criteria for considering second-order effects

The criteria for considering second-order effects (cl. 7.2.1 in prEN 1993-1-1:2020) in the global analysis, the rules for methods of structural analysis as a function of the verification methods at the ultimate limit states (cl. 7.2.2) and rules for imperfections (cl. 7.3) are key components of the revised section on structural analysis.

The draft prEN 1993-1-1:2020 contains two criteria for deciding whether second-order effects should be taken into account. The first criterion (Eq. (1)) looks at non-sway member buckling modes, see Fig. 8. Hence, second-order effects due to in-plane or out-of-plane member buckling may be neglected for the global analysis if the criterion is satisfied. The k_0 value is a Nationally Determined Parameter (NDP) that can be specified by the national standardization bodies of the different countries. The recommended value is 25.

$$\alpha_{\mathrm{cr,ns}} = \frac{F_{\mathrm{cr,ns}}}{F_{\mathrm{Ed}}} \geq k_0 \tag{1}$$

where:

$F_{\mathrm{cr,ns}}$ minimum elastic critical flexural buckling load for either an in-plane or out-of-plane member (non-sway) buckling mode

F_{Ed} design load on structure

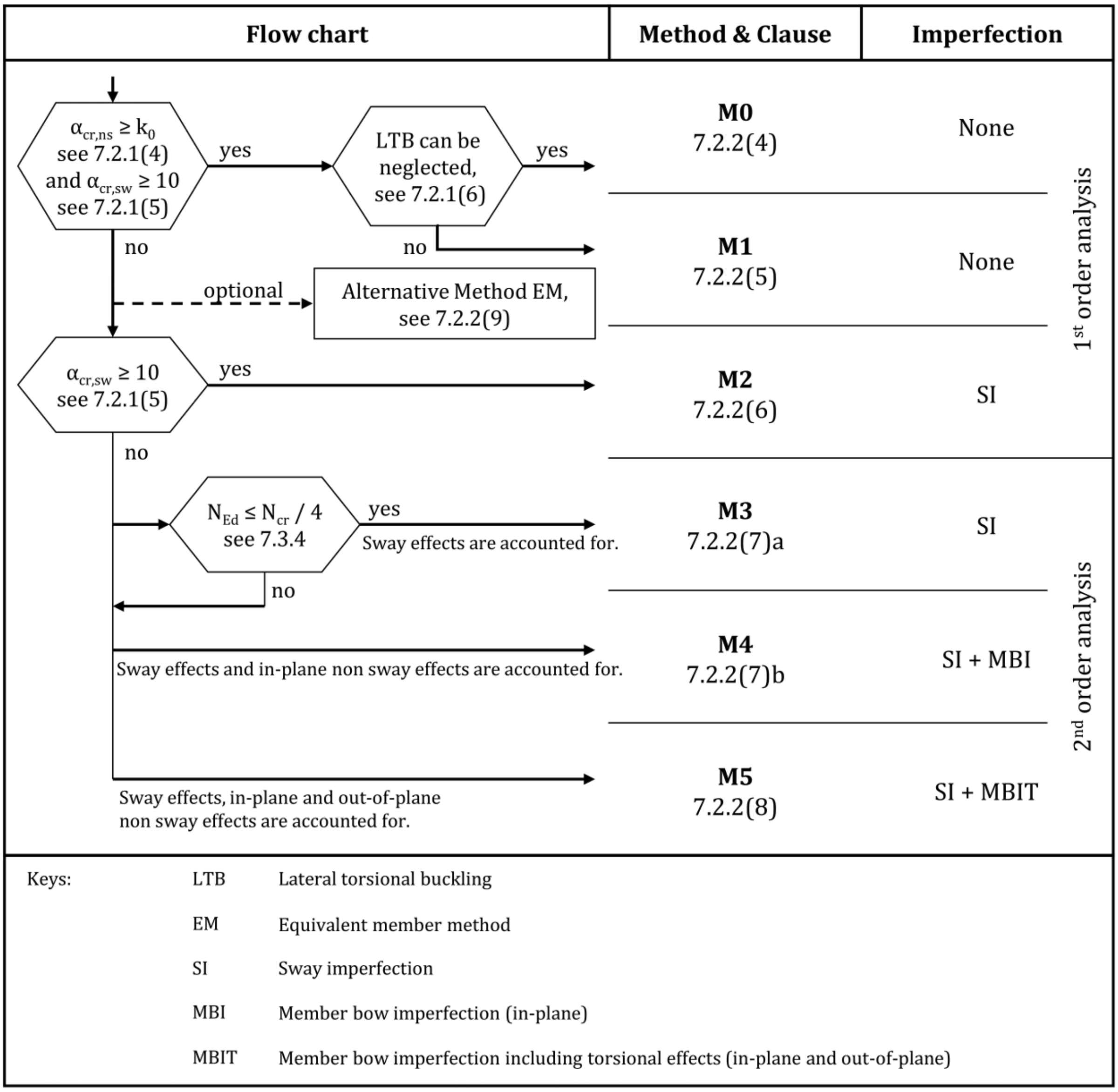

Fig. 7 Methods of structural analysis applicable to ultimate limit state design checks

$\alpha_{cr,ns}$ factor by which the design load would have to be increased to cause elastic instability in an in-plane or out-of-plane member (non-sway) buckling mode

The second criterion (Eq. (2)) was established to check whether first-order analysis may be used to determine the in-plane sway bending moments, see Fig. 8. Compliance with this criterion implicitly assumes that the increase in the internal forces and moments due to sway second-order effects is no more than 10 % of the original internal forces and moments according to first-order theory.

$$\alpha_{cr,sw} = \frac{F_{cr,sw}}{F_{Ed}} \geq 10 \tag{2}$$

where:

$F_{cr,sw}$ elastic critical in-plane flexural buckling load for a global (sway) buckling mode

$\alpha_{cr,sw}$ factor by which the design load would have to be increased to cause elastic instability in a global, in-plane (sway) mode

The distinction between non-sway (ns = non-sway) and sway (sw = sway) buckling modes has become necessary because in many countries different criteria are used for the two cases. Stability failure of the individual members is taken into account from an influence of 1/25 or 4 % (according to the recommended value of 25 of the NDP), the sway buckling mode from 1/10 or 10 % difference between the internal forces and moments according to first- and second-order theory. The first criterion and the value of 25 for the k_0 value result directly from the plateau length of the buckling curves. It is generally assumed that stability influences do not affect the resistance for members with a relative slenderness of up to 0.2 in pure compression. Additional background information on the criteria is provided in [19].

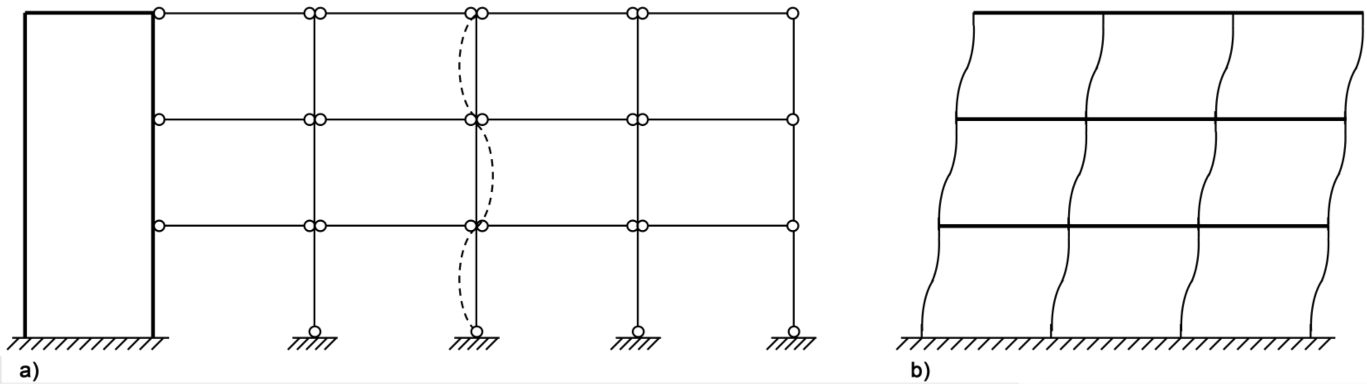

Fig. 8 Frame buckling modes: a) frame with a member buckling mode (non-sway), b) frame with a global buckling mode (sway)

The former criteria refer to in-plane or out-of-plane flexural buckling modes; torsional buckling, torsional-flexural and lateral torsional buckling are not considered. However, in contrast to the current edition of EN 1993-1-1 [2], in cl. 7.2.1(6) of the draft prEN 1993-1-1:2020 [3] criteria are specified for omitting lateral torsional buckling checks. These criteria refer to typical cases in engineering practice:

- Cross-sections with high torsional stiffness, i.e. structural hollow sections and welded box sections.
- Mono-symmetric or doubly symmetric cross-sections that are only subjected to weak-axis bending moments. In this case the destabilizing forces in the flanges of the sections balance each other and no stability effect results from the bending moment.
- Sufficient restraint to the compression flange. In this case no lateral displacement of the compression flange is possible, which typically occurs with lateral torsional buckling. The criteria provided in Annex D of prEN 1993-1-1:2020 [3] can be applied to check whether the restraint is sufficient to prevent lateral torsional buckling. The criteria of the annex not only refer to lateral, but also to sufficient torsional restraints. These stiffness criteria may usually be applied to beams in building structures.
- When the limit slenderness for susceptibility to lateral torsional buckling is not exceeded. Up to a certain limit value for the relative slenderness $\bar{\lambda}_{LT}$ for lateral torsional buckling, the reduction factor χ_{LT} according to cl. 8.3.2.3(1) of prEN 1993-1-1:2020 [3] is 1.0, i.e. the buckling resistance $M_{b,Rd}$ of a member does not have to be reduced and lateral torsional buckling has no effect on the resistance under pure bending moments. According to prEN 1993-1-1:2020, cl. 8.3.2.3, the limit of 0.2 for the relative slenderness $\bar{\lambda}_{LT,0}$, which is the value determined from the buckling curves, may be increased to 0.4 under certain conditions.

In principle, these cases refer to pure bending. For the rare case of torsional or torsional-flexural buckling, no criteria exist and there is also no generally accepted method of second-order analysis. In these cases, verification of the structural member according to cl. 8.3.1.4 of prEN 1993-1-1:2020 [3] is required.

4.1.3 Methods of analysis

The list of methods of analysis, see Fig. 7, starts with the simplest case that a first-order analysis suffices (method M0). In that case both criteria for the member (non-sway) buckling mode and for the global (sway) buckling mode, Eqs. (1) and (2), are fulfilled. The same applies for the case of method M1, with the only difference being that lateral torsional buckling verification is necessary according to cl. 8.3 of prEN 1993-1-1:2020 [3].

Method M2 refers to structural systems as shown in Fig. 8a, where the sway buckling mode is prevented by stability elements such as concrete stairs or bracing. Hence, only member (non-sway) buckling modes have to be considered when verifying the individual members according to cl. 8.3 of prEN 1993-1-1:2020. Nevertheless, a sway imperfection should be considered in the structural analysis, see section 4.2.2. This conceptional approach sometimes differs from the current one in common use today.

Methods M3 and M4 belong to design situations where both criteria fail according to Eqs. (1) and (2). Consequently, a "mixed" procedure according to EN 1993-1-1, 5.2.2(3), case b), is applied: consideration of second-order effects and imperfections partially by global analysis and partially through individual stability checks of members. The two methods differ according to the extent of the second-order analysis. The in-plane analysis of method M3 considers only the global second-order effects and imperfections, but may assume the internal forces and moments between the member ends according to first-order theory. However, method M4 covers both – global and member second-order effects and imperfections for the in-plane analysis. This latter procedure allows the application of the member check to be restricted to Eq. (8.89) in prEN 1993-1-1:2020 [3] (similar to Eq. (6.62) of EN 1993-1-1) representing the out-of-plane member buckling verification, usually on the "weak" axis of the cross-section including lateral torsional buckling.

The most complex method is M5. All in-plane and out-of-plane second-order effects, including torsional effects, as well as global (sway) imperfections and in-plane and out-of-plane local bow imperfections are accounted for in the analysis according to this method. Only a cross-sectional check according to cl. 8.2 of prEN 1993-1-1:2020 [3] with second-order internal forces and moments is necessary for verification.

The complexity increases from method M0 to M5, but the basis of all approaches is identical. This is not the case with the "Equivalent Member" (EM) method, which is an independent and alternative approach. This traditional method also covers the global sway buckling mode by applying an effective buckling length for the stability member check using only first-order internal forces and moments. Imperfections are not explicitly considered in the structural analysis, see also EN 1993-1-1 [2], 5.2.2(3) c) and (8). The limitations of this method are discussed in various textbooks and commentaries, e.g. [20].

The different methods of analysis M0 to M5 and EM were not newly developed. These methods are already common practice within the framework of the current version of EN, but the application of the methods sometimes remains unclear. Therefore, the new systematic approach and the definition of the different methods of analysis markedly enhance clarity and ease of use. Another part of Eurocode 3, prEN 1993-1-3:2019 "Supplementary rules for cold-formed members and sheeting" [21], has also started to define its methods of analysis in the same way so that not only more clarity, but also more harmonization will be achieved.

4.2 Imperfections

4.2.1 General

The draft prEN 1993-1-1:2020, cl. 7. 3. 1(5), considers various types of imperfection:

- Sway imperfections for global analysis of frames (cl. 7.3.2 and 7.3.4 of prEN 1993-1-1:2020)
- Bow imperfections for global and member analysis (cl. 7.3.3 and 7.3.4)
- Imperfections for analysing bracing systems (see cl. 7.3.5)
- Imperfections based on elastic critical buckling modes (see cl. 7.3.6)

In particular, the first two are of great relevance in connection with the structural analysis and the methods M0 to M5 described above. Structural analysis according to the methods M2 to M5 considers sway imperfections. Methods M4 and M5 additionally consider bow imperfections, which is basically congruent with the current procedure in EN 1993-1-1 [2]. However, the calculation of the imperfections has changed in the draft prEN 1993-1-1:2020 [3] and is explained in the following sections.

4.2.2 Sway imperfections for global analysis of frames

There are two main changes compared with the current rules. Firstly, the basic value is now given as a function of the method used for verifying cross-sections and members, i.e. elastic or plastic resistance. Secondly, the limit for the reduction factor α_H for height has changed.

The design of steel structures must consider equivalent geometric imperfections if the plastic capacity of the cross-sections is utilized. In addition to the pure geometric imperfections, the effect of other influences is also taken into account, essentially those due to residual stresses and plastic deformations caused by internal forces and moments. Hence, smaller sway imperfection values are sufficient for a structural analysis and design according to the theory of elasticity, i.e. if the plastic capacity of the cross-sections is not utilized. In addition to the basic value $\Phi_0 = 1/200$ for the equivalent geometric imperfection of the current standard, the draft prEN 1993-1-1:2020 contains an additional basic value for pure geometric imperfection $\Phi_0 = 1/400$ that can be applied if the cross-sectional capacity is computed based on elastic theory. These values are based on measurements [22], [23], [24], [25] and numerical simulation studies [26], [27]. Details of and the background to the values are summarized in [20].

The current EN 1993-1-1 considers a minimum limit value of 2/3 for the height reduction factor. This limit resulted from the original request for a uniform definition of sway imperfections in Eurocode 2 and Eurocode 3. In the draft prEN 1993-1-1:2020, the limit value has now been waived based on the measurement results in [24], [25].

4.2.3 Bow imperfections for global and member analysis

The results of numerical simulations [28] showed the need for adjustments to bow imperfections for global and member analysis. The current values of the bow imperfections in EN 1993-1-1 are based on investigations of centrically loaded members. The effect of combined axial forces and bending moments and the associated pronounced spreading of the yield zones and stiffness degradation have not been taken into account so far and may cause unconservative design results. Moreover, the current values were established for steel grades up to S460. Therefore, new rules for bow imperfections have been implemented in the draft prEN 1993-1-1:2020. These rules consider steel grades up to S700 and differentiate, first, between buckling about the y-y and the z-z axes and secondly, whether the plastic or elastic resistance of the cross-section is utilized for the cross-section verification. Tab. 8 gives the reference relative bow imperfection β used to compute the equivalent bow imperfection e_0 according to Eq. (3).

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Tab. 8 Reference relative bow imperfection β

Buckling about axis	Elastic cross-section verification	Plastic cross-section verification
y-y	1/110	1/75
z-z	1/200	1/68

$$e_0 = \frac{\alpha}{\varepsilon} \cdot \beta \cdot L \tag{3}$$

where:
L member length
α imperfection factor depending on relevant buckling curve
ε material parameter according to Eq. (4):

$$\varepsilon = \sqrt{\frac{235}{f_y}} \tag{4}$$

The type of plastic interaction, i.e. linear or non-linear, used in the recalculation of the representative bow imperfections e_0 also has a marked influence on the numerical values of e_0. The draft prEN 1993-1-1:2020 prescribes the use of the linear plastic interaction according to Eq. (5) for flexural buckling about the y-y axis for all kinds of cross-section. Any plastic interaction may be applied for flexural buckling about the z-z axis. However, the design plastic moment resistance of the cross-section $M_{pl,Rd}$ is limited to 1.25 times the elastic resistance $M_{el,Rd}$. Alternative approaches for bow imperfections which may be used in conjunction with the draft prEN 1993-1-1:2020 are presented in [29] and [30].

$$\frac{N}{N_{pl,Rd}} + \frac{M}{M_{pl,Rd}} \leq 1 \tag{5}$$

4.2.6 Bow imperfections for lateral torsional buckling

For a second-order analysis taking account of lateral torsional buckling of a member, it is still sufficient to apply an equivalent bow imperfection for flexural buckling about the weak axis of the cross-section. There is no need to consider an additional torsional imperfection. However, the presentation of the bow imperfections $e_{0,LT}$ (Eq. (6)) and the values have been amended and adapted to those for flexural buckling. Based on theoretical studies [31] and the German National Annex to EN 1993-1-1, the values of $e_{0,LT}$ have been modified, especially for members with medium slenderness.

Tab. 9 Reference relative bow imperfection β_{LT} for lateral torsional buckling

Cross-section	Condition	Elastic cross-section verification	Plastic cross-section verification
rolled	$h/b \leq 2.0$	1/250	1/200
	$h/b > 2.0$	1/200	1/150
welded	$h/b \leq 2.0$	1/200	1/150
	$h/b > 2.0$	1/150	1/100

$$e_{0,LT} = \beta_{LT} \cdot \frac{L}{\varepsilon} \tag{6}$$

where β_{LT} is the reference relative bow imperfection according to Tab. 9.

5 Conclusions and a look ahead to the second part of the article

This two-part article illustrates the developments in structural member verification that will be implemented in the upcoming revision of Part 1-1 of EN 1993. These developments are currently published in the form of a pre-standard, prEN 1993-1-1:2020 [3], which belongs to the second generation of Eurocodes and will be reviewed by the National Mirror Groups over the course of the next few years. The intention of this article is to familiarize future users of this standard with the main structural and technical changes. These changes aim to improve ease of use, especially with respect to clarity, harmonize the rules both within Eurocode 3 and with related standards, and integrate new findings from research and technical developments. This helps to improve structural designs and strengthens the economic efficiency of steel structures.

The introduction to the first part of the article presented the origins and content of European Commission Mandate M/515, which led to the work programme for the further evolution of the Eurocodes, of which prEN 1993-1-1:2020 is a key outcome. The other sections in this part of the article illustrated the most relevant changes pertaining to material grades, partial safety factors and their background, cross-section classification and design rules for semi-compact cross-sections as well as the various amendments to the section dedicated to structural analysis.

Part 2 of this article will be published in the next issue of *Steel Construction* and will be dedicated primarily to amendments to the design rules for member buckling introduced in prEN 1993-1-1:2020.

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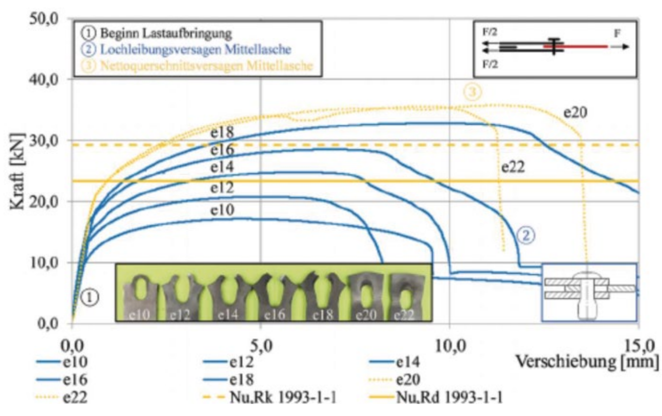
EDITOR'S RECOMMENDATIONS

Design and execution of bearing type shear connections with blind rivets in steel construction

This paper has been published in Stahlbau 4/2020 in German.

Abstract: With the development of blind rivets with a nominal diameter of d > 6.4 mm accompanies with an increasing tendency for use in steel construction. Reasons for the extended application are mainly the advantages over the classic joining technologies. Blind rivets are used wherever classic joining technologies, such as welding or screwing are not applicable or uneconomical for technological or structural reasons. Unfortunately, there are no adequate normative guidelines for the design and execution of blind rivet connections in accordance with DIN EN 1993-1-8, DIN EN 1993-1-9 and DIN EN 1090-2. Because of these approvals (ETA, abZ/aBG, ZiE) must be used for applications regulated by the building authorities.

In this article, the results of systematic investigations on the shear bearing resistance and the fatigue strength of blind rivets and blind rivet connections are presented. The aim of the investigations was to extend the existing design and execution rules in steel construction according to the DIN EN 1993 and DIN EN 1090 to blind rivet connections in order to allow a purely analytical proof of bearing type connections in future. Furthermore, it was a matter to define detail categories in accordance with DIN EN 1993-1-9 for the different types of blind rivets and connection with blind rivets, in order to enable the fatigue strength verification based on the model of Eurocode 3.



Force-displacement-diagram and failure pictures for the tests of VK Ie

Kalkowsky, F.; Glienke, R.; Blunk, C.; Dörre, M.; Henkel, K.-M. (2020) *Zur Bemessung von Scher-/Lochleibungsverbindungen mit Blindnieten im Stahlbau* in: Stahlbau 89, No. 4, pp. 304-325. https://doi.org/10.1002/stab.202000002