EC3: The New Eurocode for Steel Structures

EC3: Nouvel Eurocode pour les structures en acier

EC3: Der neue Eurocode für Stahlbauten

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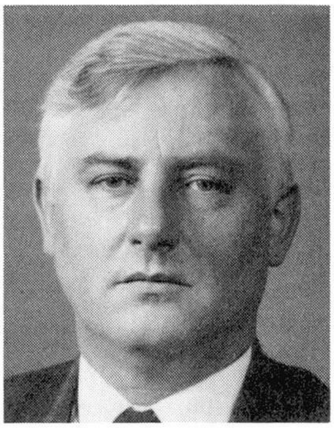

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SUMMARY

The background, scope and contents of Eurocode 3 are reviewed in this paper. Some special features of the code are highlighted. Aspects of the safety bases of the code, its use for innovative and economical design of steel structures, and its user-friendliness are outlined as an introduction to the four more detailed papers accompanying this overview. The paper concludes with a brief description of the developments planned for Eurocode 3.

RESUME

L'article présente les bases, l'objectif et le contenu de l'Eurocode 3. Il souligne quelques caractéristiques particulières de la norme. En guise d'introduction aux quatre contributions suivantes, cet article explique les aspects fondamentaux de sécurité dans l'Eurocode 3, son utilisation pour un projet innovateur et économique de structures en acier, et son emploi facile pour l'utilisateur. En conclusion, une brève description est donnée des développements prévus pour l'Eurocode 3.

ZUSAMMENFASSUNG

Hintergrund, Umfang und Inhalt von Eurocode 3 werden bezüglich einigen Eigenheiten hervorgehoben. Sein Sicherheitskonzept, seine Verwendung im innovativen und wirtschaftlichen Tragwerksentwurf und seine Benutzerfreundlichkeit werden profiliert als Einleitung zu den nachfolgenden vier speziellen Beiträgen. Anschliessend wird kurz die weitere Entwicklungsplanung für Eurocode 3 angesprochen.

BACKGROUND TO EUROCODE 3

The new European Prestandard for the design of steel structures, ENV 1993-1-1[1], which is generally referred to as Eurocode 3, or EC3, is the result of some twelve years of collaborative effort between engineers drawn from all the member states of the EEC and more recently, from the EFTA countries. The principal source document, which formed an appropriate starting point for the drafting, was produced under the aegis of the European Convention for Constructional Steelwork and issued by them in 1978 as the "ECCS Recommendations for Steel Construction"[2]. The ad-hoc drafting panel nominated by the ECCS to the CEC comprised six experts chaired by the author, most of whom had been previously involved in the preparation of the ECCS Recommendations. This panel produced several initial drafts between 1979 and 1984 when the CEC published a draft for public comment (in English, French and German), Eurocode No 3: Common Unified Rules for Steel Structures[3].

During the consultation period which followed there was considerable interaction between the mainly newly constituted Editorial Group (comprised of members drawn from the Drafting Panel and some new experts) and the liaison engineers chosen to represent the various member states. There was also close collaboration with the various technical committees of the ECCS which proved to be invaluable in formulating amended proposals to satisfy the various comments raised on the CEC draft.

The process of redrafting EC3 was undertaken by the Editorial Group in cooperation with the Eurocode Coordinating Group which was responsible for the harmonised presentation and editing of those parts of the code which are material independent; for example safety principles, terminology, accidental damage. The various liaison engineers worked with the Editorial Group to produce the final draft of "Eurocode 3: Design of steel structures; Part 1.1 General rules and rules for buildings" which was completed in 1989 and passed to CEN for publication as an ENV. The transfer of responsibility for publication from the CEC to CEN was somewhat protracted. However, various editorial matters, some connected with translating the English version into French and German, were attended to during the two years leading up to its recent issue.

Scope of EC3

Part 1.1 of EC3 contains principles which are valid for all steel structures as well as detailed application rules for ordinary land-based buildings. Many of the application rules will also be cross-referenced for bridges, towers and other structures which will be dealt with in subsequent parts of EC3 but the rules in Part 1.1 are only considered complete in respect of buildings.

CONTENTS AND LAYOUT

There are 9 chapters and 9 annexes in the new code. Annexes are classified as either normative, and have the same statistics as the material in the main body of the text, or informative, providing additional information. The chapter titles are given below and the annex titles and status thereunder.

Chapter 1 : Introduction
Chapter 2 : Basis of design
Chapter 3 : Materials
Chapter 4 : Serviceability limit states
Chapter 5 : Ultimate limit states
Chapter 6 : Connections subjected to static loading
Chapter 7 : Fabrication and erection
Chapter 8 : Design assisted by testing
Chapter 9 : Fatigue

The Annexes and their status are listed hereunder

Annex B	Reference standards	Normative
Annex C	Design against brittle fracture	Informative
Annex E	Buckling length of a compression member	Informative
Annex F	Lateral-torsional buckling	Informative
Annex J	Beam-to-column connections	Normative
Annex K	Hollow section lattice girder connections	Normative
Annex L	Column bases	Normative
Annex M	Alternative method for fillet welds	Normative
Annex Y	Guidelines for loading tests	Informative

The design procedures in EC3 are only valid if the workmanship criteria during fabrication and erection given in Chapter 7 are satisfied. For example, the levels of initial geometric imperfections assumed in many of the strength rules are directly related to these criteria and are therefore invalid if they are exceeded. A separate CEN committee, TC 135 "Execution of Steel Structures" has drafted the fabrication and erection rules using annexes prepared by the original EC3 Drafting Panel as source documents.

Ten Reference Standards are mentioned in Annex B, each of which defines a product or process and makes reference to a number of CEN and/or ISO Standards, only some of which are already drafted. Where no such standard is available each member state's National Application Document (NAD) defines the appropriate national standard which should be used with the code.

BASIS OF DESIGN

Chapter 2 of EC3 follows to a large extent the harmonised version of the General Principles and Basis of Design prepared by the Coordinating Group which adopts a limit state approach with the use of partial safety factors. The code gives indicative values for the various safety factors in boxes, the so-called boxed-values. The background to the adoption of these values is given in the accompanying paper by Brozzetti and Janss(4).

Material Properties

The code currently covers only three nominal grades of structural steel with yield stresses of 235, 275 and 355 N/mm^2 which are modified at thicknesses of 40 and 100 mm. Annex D which is currently in draft form will cover steels of higher grade with yield stresses of the order of 460 N/mm^2.

Nominal values of yield and ultimate tensile strength for various bolt grades are also given. Stark(5) in another paper accompanying this one deals with the background to the design of bolted connections.

Design against brittle fracture is treated in some detail in Annex C which gives formulae covering the various design criteria including service conditions, loading rate and consequences of failure. The table in Chapter 3 limits itself to the cases of welded tension members, to members either non-welded or in compression subjected to static loading and to normal consequences of failure. The table gives the maximum thickness for various grades and qualities of steel for temperatures down to -20°C.

The modulus of elasticity of steel is taken to be $E = 210{,}000$ N/mm^2 and the coefficient of linear thermal expansion to be $\alpha = 12 \times 10^{-6}$ per °C.

Serviceability and Ultimate Limit States

For verifying the serviceability limit state EC3 gives various deflection limits for beams under unfactored variable loads and incudes values for total (permanent and variable) deflection limits. Horizontal deflection limits for each storey of a multi-storey building and for the structures as a whole are also given. Other rules are designed to limit the effects of rainwater ponding on roofs and the effects of vibration on floors.

Methods of Analysis

Elastic or plastic global analysis may be used to calculate the internal forces and moment in a statically indeterminate structure. Distinction is made between first-order theory using the initial geometry of the structure and second-order theory which accounts for the change in shape of the structure under load. The former may be used for braced frames, non-sway frames and with design methods which make indirect allowances for second-order effects. The latter may be used in all cases but, of course, is not normally required except for sway frames.

Plastic analyses range from the commonly adopted rigid-plastic method to advanced computer-based elastic-plastic methods. Two forms of elastic-plastic analysis are distinguished. In the elastic-perfectly plastic method the members remain elastic until a plastic hinge has fully formed, whereas in the elasto-plastic method the spread of plasticity through the depth and along the length of a member is followed in an incremental computer-based analysis.

A special feature of the code is the treatment of semi-continuous frames as well as simple and continuous framing. Plastic analyses may be used for such frames. It involves the consideration of partial-strength joints which develop plastic hinges with a smaller plastic-moment resistance than the members they connect, but with sufficient rotation capacity to justify plastic analysis. The use of such joints can lead to worthwhile economies in construction, compared to simple connections.

Structural Stability

The effects of imperfections are to be taken into account in frame analysis, analysis of bracing systems and member design.

The effects of frame imperfections are dealt with by means of an initial sway imperfection which is a function of the number of columns and storeys. These sway deflections can be represented by equivalent horizontal forces. Allowance for these notional forces must be made in all load combinations including ones involving wind forces.

In the case of bracing systems allowance is made for imperfection, or bow, in the members to be restrained. Normally the effects of imperfections on member design are accounted for within the buckling strength formulae given in the code.

Clear definitions of sway or non-sway frames are given in the code. A non-sway frame is one which is sufficiently stiff for it to be acceptably accurate to neglect any additional internal forces or moments arising from horizontal displacements of its nodes. A simple criterion is given to help distinguish the two.

Distinction is also made between braced and unbraced frames. A frame is said to be braced if the bracing system reduces its horizontal displacements by at least 80%. All braced frames are treated as non-sway frames but some unbraced frames may also be non-sway frames using this definition.

The code allows the use of simple rigid-plastic analysis for braced frames and for unbraced frames up to two storeys high, with amplified sway moments if they are sway frames. Sway frames may be designed using rigid plastic analyses provided the simplified method given in the code is used and all the accompanying conditions are met. Otherwise a second-order elastic-plastic sway analysis must be used.

DESIGN OF MEMBERS

In designing structural members consideration needs to be given to cross-section resistance, buckling resistance of the member and where appropriate, shear buckling resistance and web crippling including stiffener design.

Cross sections are divided into four classes. Class 1 sections are ones for which it is possible to take advantage of the full plastic cross-sectional resistance and use rigid-plastic analysis with full moment redistribution. Class 2 sections are ones which can develop their plastic resistance but have limited rotation capacity. Class 3 sections are ones which can reach yield in their extreme compression fibre, but in which local buckling prevents the attainment of the full plastic moment capacity. Class 4 sections are ones where appropriate allowances must be made for the effects of local buckling when determining their resistance.

The limitations on slendernesses of the various elements of the cross-section have been selected from the best available data and the tables presented allow a separate classification under axial load for the compression term and under bending for the moment term when combined bending and axial loading is being considered. The concept of effective width is used to reduce the section properties of class 4 to sections whilst retaining the full yield strength in calculating the resistance.

Fastener holes in compression zones of the cross-section are neglected whereas the design tension resistance is taken as the smaller of the design plastic resistance of the gross area based on the yield stress and the design ultimate resistance of the net area using 90 per cent of yield and a larger safety factor. The shear resistance of a web need not be reduced due to bolt holes unless the ratio of net to gross area is less then the ratio of yield to ultimate strength. It is, however, necessary to check block shear strength at the ends of a member.

The effects of shear on the plastic moment of resistance must be accounted for when shear exceeds 50 per cent of the plastic shear resistance and an appropriate expression is given for the reduced plastic moment of resistance. Interaction expressions for axial force and bending moments are also given.

Buckling Resistance of Members

The bases for the checks on buckling resistance of columns are the European Column Buckling Curves contained in the ECCS Recommendations[2] which have been derived from the statistical evaluation of test results of a large number of experiments on columns with different sections, production methods and steel grades. Four column curves are given, the selection of which is based on the type of cross-section and axis of bending. A non-dimensional slenderness is calculated based on buckling length. The latter can be obtained from Annex E. When used with the appropriate column curve this slenderness leads to a reduction factor which is applied to the compression resistance of the cross-section to give the buckling resistance.

Lateral-torsional buckling strength is calculated by a method which also refers back to the column buckling curves with an appropriate slenderness to determine the reduced design buckling resistance moment. Annex F may be used to arrive at this slenderness which is a function of the elastic critical moment for lateral-torsional buckling of the beam, the type of loading and the degree of warping restraint. If the non-dimensional slenderness is less than 0.4 no reduction in strength need be made.

Shear Buckling Resistance

Unstiffened webs with depth to thickness slenderness ratios less than given values (e.g 69 for Fe 360) need not be checked for shear buckling. Where these limits are exceeded two methods may be used to check the shear buckling resistance. The first is a simple postcritical buckling approach and the second allows for tension field action. Both may be used for transversely stiffened webs and rules are also given for the design of the stiffeners which includes checks on their compressive buckling strength and their bending stiffness.

Plate girders with more complex stiffening arrangements rarely occur in buildings so the rules are felt to be sufficient for the scope of Part 1.1. It is intended to produce more comprehensive design rules for plated structures which will have more general application to other forms of construction such as bridges and offshore structures.

Design rules for flange buckling in the plane of the web, as well as guidance for members curved in elevation are included in the code. The resistance of webs to inplane transverse forces such as occurs at supports and in some beam-column connections is treated by considering three modes of failure. These are web crushing, local buckling or crippling of the web, and overall buckling of the web. Limited expressions are given to cover each mode.

Triangulated Structures

Built up compression members such as laced or battened columns are treated in some detail. Rules are given for the buckling resistance of the chords, lacing members and battens based on an analogous model of a member subjected to finite shear deformations and including the effects of initial imperfections.

DESIGN OF CONNECTIONS

The code contains an unusually large section on connection design because of the importance of this topic in relation to the economical design of steel structures. The accompanying paper by Stark[5] is dedicated to a discussion of this part of the code.

Bolted connections are divided into five categories which distinguish between connections loaded in shear and tension, and connections with preloaded bolts which are designed to resist slip. Advantage is taken of the larger deformations which are allowed to occur in the design of connections where rotation is required at the end of beams. In the case of welded connections advantage has been taken of the best information available for the design of fillet welds, both side and end, long lap joints and intermittent welds.

Beam to column connections, both welded ones and ones with bolted end-plate connections, are treated in Annex J. It also contains data on the calculation of prying forces. A special feature of the code is the treatment given to semi-rigid and partial strength connections[5] which should lead to more economical design of frames by avoiding the use of expensive stiffening of joints in many cases.

Design Against Fatigue

A chapter covering fatigue is included in Part 1.1 as part of the general rules for design of steel structures which can be referred to from other future parts,but, of course, can also be used for building design where fatigue is an issue. The paper by Brozzetti and Janss[4] gives a more detailed description of the background to these rules.

THE RELIABILITY AND POTENTIAL ECONOMY OF EUROCODE 3

Eurocode 3 can be claimed to be amongst the most extensively calibrated and cross-checked code ever written for steel structures. The numerous background studies carried out to calibrate element design rules as described by Brozzetti and Janss[4], together with the trial calculations carried out at national and international level, as described by Finzi and Taylor[6], together with the enormous contribution made by member states and channelled through their liaison engineers, have combined to ensure that the code has been subjected to extensive checking. Potential economies can only be judged at a national level by comparing the outcome of design to EC3 with designs done using national codes and loading. Calibrations against designs done using EC3 and the proposed loadings in EC1 have yet to be carried out and are planned for the future.

However, several aspects of EC3 have been introduced specifically to encourage efficient and economical design of steel structures. These include the previously mentioned semi-rigid design methods and design for partial-strength connections. A feature of these developments and others relating to web design and joint design for hollow sections is that fabrication is reduced to a minimum by avoiding, where possible, the use of stiffeners, welding etc. whilst compensating for any possible apparent reduction in strength by exploiting postbuckling or plastic reserves which are and often ignored in more traditional codified design procedures. An efficient application of the code should lead, therefore, to more cost-effective structural detailing.

Yet another feature of the code has been the attempt to encourage innovation. Recognising the increasing use of computers in design, methods of global analysis, frame design, etc. are permitted which, potentially, should lead to more innovation in design. This matter is treated in greater detail in the accompanying paper by Sedlacek[7].

Design assisted by testing, which may often be resorted to by innovative engineers introducing new systems is encouraged by the code which includes a chapter on the subject outlining the principles which should be followed. Annex Y develop principles in greater detail and covers test conditions and procedures including acceptance tests, strength tests, tests to failure, check tests and other test procedures.

EASE OF USE OF EUROCODE 3

The drafters of the code have been conscious of he need for the code to be user friendly from the outset of their work[6].

However, the problem is not an easy one to solve solely within the code itself. This is because potential users vary from engineers in large consultancy offices with full computer aided engineering facilities available, to designers in offices of small steel fabricators with few such facilities available and with an interest confined to a very restricted range of steel structures. Other potential users include engineers within offices of regulatory authorities, proof engineers, engineers and students based in educational establishments and construction engineers on site. It is only with a hierarchy of supporting material ranging from concise versions of the code for a restricted range of applications, through design aids, dedicated software and text books that it will be possible to satisfy all the demands for user-friendliness. Happily steps are being taken at national and international level to provide this essential backup to the code[6].

FUTURE DEVELOPMENTS OF EUROCODE 3

Further parts of EC3 which are being prepared at present include the following annexes to Part 1.1 and other Parts.

Annex D	The use of steel grade Fe E 460
Annex K	Hollow section lattice girder connections
Annex G	Design for torsion resistance
Annex H	Modelling of building structures for analysis
Annex N	Openings in webs
Annex S	The use of stainless steel
Part 1.2	Fire resistance
Part 1.3	Cold formed thin gauge members and sheeting
Part 2	Bridges and plated structures

Work has now yet commenced on the following parts which are planned for the future.

Part 3	Towers, masts and chimneys
Part 4	Tanks, silos and pipelines
Part 5	Piling

Part 6	Crane structures
Part 7	Marine and maritime structures
Part 8	Agricultural structures

The part dealing with cold formed members and sheeting, Part 1.3, should be available shortly, together with Annex D on high yield steel and a revised Annex K which includes multiplanar as well as single plane connections of hollow section members. The remainder of the work in progress should follow a year later. Of the planned new work, Part 3 on towers, masts and chimneys and Part 5 on piling will receive priority.

CONCLUDING REMARKS

Eurocode 3 has been produced by the combined efforts of a large number of experts throughout the EEC and EFTA. It has also had a not inconsiderable input from colleagues in central and eastern Europe as well as experts from the United States and Japan and elsewhere. Although no code can be perfect and satisfy all its potential users the Editorial Group has gone to great lengths to ensure maximum exposure for the code so as to attract as much useful feedback as possible, which has been incorporated into the ENV 1993-1-1.

It is to be hoped that it will not just be studied but used extensively during the ENV period so that when it is issued in modified form as an EN in a few years time it will prove to be a powerful tool in uniting Europe in the field of steel construction and will help fuel further growth in the proper and effective use of steel in construction.

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EC3: A Eurocode for Reliable Steel Structures

EC3: Un Eurocode fiable pour les structures en acier

EC3: Ein Eurocode für sichere Stahlbauten

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SUMMARY

After having recalled in which context the background information particular to Eurocode 3 were made, this paper presents the basic studies which have been carried out to justify the choice of strength formulae, and in certain cases, when experimental data were sufficient enough, to determine the values of the partial safety factors assigned to the limite state functions. The evaluation of the partial safety factors depends upon some assumptions concerning the choice of the reliability level adopted which are given within the framework of this paper. At last the paper describes, in general terms, the calibration procedure which was set out within the study framework of the Editorial Group of the Eurocode 3.

RESUME

Après avoir rappelé dans quel contexte se situe le développement des études particulières à l'Eurocode 3, cet article présente les études de bases qui ont servi à justifier du choix des formules de résistance, et dans certains cas, lorsque les données expérimentales étaient en nombre suffisant, à déterminer les valeurs des coefficients partiels de sécurité affectés aux modèles de fonctions d'états limites. La détermination des valeurs des coefficients partiels de sécurité relève d'hypothèses et de certains choix sur les niveaux de sécurité qui sont précisés dans le cadre de cet article. Enfin on décrit, dans ses grandes lignes, la procédure de calibration qui a été mise au point dans le cadre des études du Groupe de rédaction de l'Eurocode 3.

ZUSAMMENFASSUNG

Nach Hinweis auf die Gründe und Zielsetzungen für die Ausarbeitung von Hintergrundberichten zu den Eurocode 3-Regeln wird auf die Grundlagenuntersuchungen eingegangen, die zu der Wahl der Bemessungsformeln für die verschiedenen Grenzzustände und soweit genügend Versuchsdaten vorhanden, auch zur Festlegung der Sicherheitsbeiwerte geführt haben. Die Teilsicherheitsbeiwerte hängen vom angestrebten Zuverlässigkeitsniveau ab; die Annahmen dazu werden in diesem Bericht angegeben. Schliesslich wird ein Überblick über das verwendete Versuchsauswerteverfahren gegeben, das von der Redaktionsgruppe für den Eurocode 3 für die Hintergrundberichte benutzt wurde.

1. INTRODUCTION

Eurocode 3 claims to be based on the best scientific and professional information available today. EC3 adopts modern principles in matter of structural safety based on probabilistic concepts of safety within the framework of a level 1 reliability code format through the use of partial safety factors applied to the load effects derived from a proper structural analysis and to the design resistance. The method of checking structural safety envisaged in EC3 refers to limit states and does not anymore refers to the traditional allowable stresses concept.

Adopting these main principles it results a major change for European countries, where codes are still based on the method of allowable stress design, and it requires a substantiated assessment of the safety.

National codes of the European Economical Communities member countries reflect various level of experiences or knowledges and various design practices which make difficult to reach a final consensus on the best safe and economical design formulae to be adopted for Eurocode 3.

Although recognizing the intrinsic value of the ECCS Recommendations, which represents the principal source document, they were incomplete on certain items and did not present a sound consistency between the single chapters.

From the Eurocode 3 first draft revision period it was clear that the conflicting ideas on particular design requirements or strength design model could only be solved by background appraisal studies.

At last, few knowledge and experience existed on newly developed high strength steel material elaborated by the steel industry such as the FeE 460 TM. The applicability of the design rules derived for normal steel grades needed to be proved for this new material and partial safety factors had to be adequately determined.

These were the main reasons which led the Editorial Group to undertake such detailed studies. These background information studies were carried out to fulfil various objectives :

- to assess the background information on the choice which led the Editorial Group to adopt particular design requirements when experimental data were insufficient to perform a sound statistical analysis, in such case, the objective was purely informative ;

- to choose the best "qualified" strength function, suitable model factors, and related partial safety coefficients. The statistical procedure required for such development could only be performed if a sound experimental basis already existed ;

- to achieve a coherent and an uniform safety level through the entire Eurocode 3 design code.

This paper presents an overview to the scientific background studies which have been carried out under the supervision of the Editorial Group to support basic provisions and reliability levels of design formulae of Eurocode 3. A special attention is brought to the definition of the partial safety factors which were considered in the studies of various strength formulae and particularly for the fatigue design strength.

2. STRENGTH FUNCTIONS

When designing a structure and its components parts an appropriate structural model has to be chosen and this choice concerns the analysis of the structure and the design check. A model implies the use of a method of global analysis (elastic or plastic) combined to a method of cross section or member resistance design check. Eurocode 3 provides the necessary requirements for such a choice as the best "qualified" strength functions (members in tension, bending or compression, connections design, fatigue assessment of structural details ...).

Using the agreed evaluations procedure described hereafter a uniform safety level was sought for all the proposed strength formulae and design rules throughout the Code.

3. GENERAL PRINCIPLES ADOPTED ON THE RELIABILITY LEVEL 1 CODE FORMAT

3.1 Design and safety checking format

In many background studies (see the list given in reference [3]) proposed strength formulae (strength functions) and design rules were compared with available tests results. The adoption of the limit state methods and the use of partial safety factors was adopted as the general checking and reliability design format through the entire Eurocode 3.

With this concept in mind, safety analysis is carried out comparing the effects of the actions (s) with the material or structural element strengths (r). Both variables are random and belong to a limit state function of the type :

$$z = r - s \tag{1}$$

where the random variables R and S denote, on the one hand, the effects of the material strength and geometrical uncertainties of the structural element and, on the other hand, the action effect variations.

In the context of a reliability analysis it is necessary to assess the probability of the failure event, i. e., the risk appreciation that the limit state expressed by the equation (1) will be reached, and to check that this probability is lower than a predefined value p_f :

$$\text{Prob}\{ Z < 0 \} < p_f \tag{2}$$

Under particular assumptions, knowing the mean value $(\bar{z})$ and the standard deviation σZ of the random variable Z, this probabilistic failure criterion may be replaced by the following condition :

$$\bar{z} - \beta . \sigma_Z > 0 \tag{3}$$

where β is denoted as the "design safety index".

For the case where R and S (the respective random variables of r and s are both normal, it can be demonstrated that :

$$\beta = -\Phi^{-1}(p_f) \tag{4}$$

where $\Phi^{-1}(.)$ stands for the inverse function of the standardized cumulative normal distribution.

With reference to a fixed limit state function as expressed by equ. (1), the reliability of the partial safety concept level 1 method as shown in equ. (5) can be made strictly identical to level 2 safety concept method as represented by the equ. (2) if the partial safety factors γ_f and γ_m are expressed by means of a proper functional relationship in terms of β.

$$\gamma_f \cdot s - r/\gamma_m > 0 \tag{5}$$

3.2 Choice of the reliability level

The reliability level is expressed by the safety index β. For the design rules of Eurocodes 3 the following failure probability were proposed [2] :

Safety classes	ULS		SLS	
	p_f	β	p_f	β
reduced	- 5. 10^{-4}	3.3	- 16. 10^{-2}	1.0
normal	- 7. 10^{-5}	3.8	- 7. 10^{-2}	1.5
high	- 8. 10^{-6}	4.3	- 2.3 10^{-2}	2.0

The safety requirements are defined by three safety classes. The various levels of target values of reliability indices take into account the possible consequences of failure in terms of risk to human life or injury and economic losses resulting from failure. The above table was established under particular assumptions with a reference to a 50 years design life of the construction. The conversion of β from the reference period of T= 50 years to an another reference period T' can by obtained from the following transformation :

$$\beta' = \Phi^{-1}\{[\Phi(\beta)]^{T'/T}\} \tag{6}$$

The characteristic and design values of strengths (r_k, r_d) in Eurocode 3 have been evaluated on the basis of the normal safety class with β =3.8 as target value for ultimate limit state (ULS) and β = 1.5 for serviceability limit states (SLS).

4. GENERAL PROCEDURE FOR THE DETERMINATION OF CHARACTERISTIC STRENGTHS FROM TESTS AND FROM GIVEN STRENGTH FUNCTIONS

Many background studies deal with the comparison of strength formulae with available experimental tests results. The resistance of structures or part of structures in Eurocode 3 is check against a number of ultimate limit states which can be reached. These limited number of ultimate limit states are taken into account by suitable strength functions, specific for each limit state considered, which allow the definition of a set of safety domains.

These strength functions were checked against tests results in order to obtain the characteristic strength functions and their corresponding partial safety factors complying with the target safety value given in § 3. The main aspects of this statistical calibration procedure, developed specially to have a common reference for the background studies of Eurocode 3, are presented in the following [see ref. 3, Doc. 7.01].

4.1 Main steps of the calibration procedure

Lets have a strength function denoted by $g_R(X)$, where X are the basic random variables (geometrical, resistance) assumed to follow a lognormal a probability density function, the calibration procedure proceeds from the following main steps :

- STEP 1 : Check the correlation between experimental and calculated values :
The strength function $g_R(X)$ compared with the tests results will be corrected if necessary by an additional factor $\overline{b}$ (mean value corrective factor) ; an another factor δ (an error term) gives an information on the scatter of the results from the mean value of the strength function. If the coefficient of correlation is greater than 0.9 then no correction is brought to the model function.
The strength function must be established on a sound mechanical or physical interpretative model of the mode of failure considered or observed during the experiment. It should also include all relevant basic variables which affect the resistance.

- STEP 2 : Evaluation of the statistical characteristics of the basic variables and of the error term.
In this step the determination of the mean value and the standard deviation of the basic variables may be obtained from the statistical analysis of the test data, if their representative values have been measured, if not they may be assumed from preknowledge or from an initial guess of the coefficient of variation.

- STEP 3 : Determination of the characteristic strength function and of the design strength function (or the partial safety factor) :

The characteristic strength function is evaluated from the basic stastistical information on all variables as established in step 2. Two assumptions are made concerning the calculation of the value of the characteristic strength function :

- The number of test specimens is such that it can be considered as infinite. In such case, their is no statistical uncertainty, and the characteristic value of the strength function can be determined in a straightforward manner.

- The number of test specimens is limited, therefore a statistical uncertainty is taken into consideration.

The fractile factor k_s is determined according to the relevant number of test results. k_s is established for a estimating 5% fractile and a level of confidence of 75%.

The same derivation applies for the design strength function which must satisfy the requirement of a given value (target value) of the design safety index β. Depending upon also the number of test results, a table gives the corresponding design fractile factor k_d.

Then the partial safety factor applied to the characteristic strength function can be calculated from the following relation :

$$\gamma_M = r_k / r_d \qquad (7)$$

4.1 Remarks concerning the calibration procedure

The calibration procedure for the determination of the characteristic strength function and for the associated partial safety factor can be adapted to take into account for particular cases.

The incompleteness of available statistical data does not always allow the rigorous characterization of all the basic random variables. Particular considerations have been developed when some of the variables are defined by their nominal values (which are related in most cases to their mean values) instead of their characteristic values. Then the calibration procedure introduces a distinction between variables which have been measured within the course of the experimental investigation and variables for which preknowledge exists on their coefficient of variation.

The calibration procedure has been fully described when the strength functions are of particular formats such as the followings :

$$g_R(x) = X_1 * X_2 * .....* X_n$$

$$g_R(x) = X_1^{\alpha} * X_2^{\beta} * .....* X_n^{\nu}$$

$$g_R(x) = g_R(x_i) + g_R(x_j) + ...$$

For more complicated strength functions a more general iterative treatment is needed to determine the minimum distance of the limit state surface boundary from the origin of the standardized variables.

5. FATIGUE STRENGTH RELIABILITY IN EUROCODE 3

Several uncertainties affect a structural element subjected to fatigue loading. The variability of the parameters governing the fatigue strength life (i. e. fatigue loading and fatigue resistance) needs to be studied with careful attention.

A level 2 reliability model has been implemented (see ref. 3, Doc. 9.02) for the derivation of recommended partial safety factors in relation with the following fatigue strength assessment equation :

$$\gamma_F \cdot \Delta\sigma_{equ} = \Delta\sigma_R / \gamma_R$$

Where :

$\Delta\sigma_{equ}$ is the equivalent constant applied stress range which, for the given number of cycles, leads to the same cumulative damage as the design spectrum.

$\Delta\sigma_R$ is the fatigue strength as defined by the S-N curve of the relevant detail category.

γ_F and γ_R are the partial safety factors applied respectively to the spectrum loading and the resistance.

The partial safety factors depend upon the required safety index for which recommended values have been proposed on the basis of an appreciation of a risk appraisal which may be expressed in terms of two main parameters :

- The notional concept of "non fail-safe" and "fail-safe" of a structural element whose fracture may potentially gives rise (or not) to a catastrophic failure of the whole structure.

- The periodic inspection and maintenance of the construction in conjunction with the more or less accessibility of the structural detail for inspection and repair. Difficulties to access may be such as to make the detection or the repair unpractical, and in such a case particular measures to perform inspection should be taken.

It must be understood that the safety indices which were proposed (see following Table) are mainly based on an engineering judgement of what may be called a potential risk of acceptance of losses or damages. It belongs to each concerned authorities to make the decision on the proper choice of these values on the basis of a realistic risk assessment.

	"fail-safe" structural detail	"non fail-safe" structural detail
Periodic inspection and maintenance. Accessible joint detail.	$\beta = 2$	$\beta = 3$
Periodic inspection and maintenance. Poor accessibility.	$\beta = 2.5$	$\beta = 3.5$

The fatigue strength curve for each appropriate detail category has been determined on the basis of a statistical analysis of fatigue test data. The value of the "statistical" stress range $\Delta\sigma_{R,stat}$ correponding to a value of N of two million cycles, has been calculated for a 75% confidence interval of a 95% probability of survival for log N, taking into account the standard deviation and the sample size. Then the detail has been tabled to the closest appropriate conventional safe fatigue S-N curve.

Discontinuities play a major role in the fatigue strength, particularly for welded detail, and a careful consideration must be given to the weld quality which affects deeply the fatigue strength variation. About 6000 experimental fatigue test results were analysed altogether, and standard deviations of fatigue strength varied from $S_{\log\Delta\sigma R} = 0.1$ to $S_{\log\Delta\sigma R} = 0.2$.

In Chapter 9 of Eurocode 3 values of the product $\gamma_F.\gamma_R$ have been proposed on the assumption that $\gamma_F = 1.0$. There is few information concerning the fatigue loadings, and their characteristic strength and associated partial safety factors have to be evaluated from special statistical studies of recorded fatigue loading spectra, and the value of γ_F may thus be adjusted. Eurocode 3 does not give information on the fatigue loading ; this will be given in Eurocode 1.

6. CONCLUSIONS

The statistical evaluation procedure proposed in the former Annex Z of Eurocode 3 (which will be implemented in Eurocode 1) is fully independent of the variation of the load effects, the primary objective of the procedure is to calibrate selected strength functions versus a set of reference tests in order to obtain consistent values of partial safety factors.

Normally each strength function should have its own specific value of γ_M, however in order to avoid a large variety of partial safety factors on the resistance two reference values of γ_M were selected :

- γ_{M1} = 1.1 to be applied to all resistance formulae related to the yield strength f_y.

- γ_{M2} = 1.25 to be applied to all resistance formulae related to the tensile strength f_u (generally for bolt and weld resistances or net section and bearing strength).

However, for the particular case of hot-rolled sections of classe 1 that are bent about the strong axis bending and not subjected to any instability phenomena (except local buckling in the plastic domain) it has been found, from calibration studies using data (geometrical dimensions and yield strengths) from some modern European mill plant, that it would be justified to reduce γ_{M1} factor to the value of γ_{M0} = 1.0. However it is thought that this rule needs to comply with production control and quality assurance system requirements.

The calibration procedure has been developed mainly to propose a consistent methodology to evaluate the partial safety factors on strength. The γ_{M1} values which have been proposed by the Editorial Group are indicative and are identified through the Eurocode 3 document by a border frame ("boxed values"). The national Authorities in each member country are free to assign alternative values to these partial safety factors on due account of their own experience.

The main objective of a code is presumed to the achiement of structures which are optimal with regards to the state of economy and development and general values and experiences of the nation. Moreover, the measures that can be taken to achieve the required degree of structural reliability include not only the justification of relevant design rules and choice of associated partial safety factors, but also it requires an appropriate level of execution quality and proper standards for workmanship. Execution of steel constructions is covered partly in chapter 7 of Eurocode 3, which gives generally the minimum requirements. Rules related to execution and workmanship are further developed under the auspices of CEN TC/135.

7 . BACKGROUND DOCUMENTS REFERENCES FOR EUROCODE 3

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Recommendations for steel structures.
European Convention for Constructional Steelwork, Bruxelles, 1978.

[2] CEB Bulletin n° 116-E
Volume 1, Paris, 1976

[3] List of the Background Documents :

References to volume 1 - Chapter 1 to 9

Doc. 2.01 Background document for chapter 2 of Eurocode 3.

Doc. 3.01 Design against brittle fracture.

Doc. 3.02 The relation between the nominal value of the yield strength in EC3 and the specifications in material standards

Doc. 4.01 Background document for chapter 4 of Eurocode 3

Doc. 5.01 Background document for the justification of safety factor $\gamma_{M0} = 1{,}0$ for rolled beams in bending about the strong axis.

Doc. 5.02 The b/t ratios controlling the applicability of analysis models in Eurocode 3

Doc. 5.03(1) Evaluation of test results on columns, beams and beam-columns with cross-sectional classes 1 to 3 in order to obtain strength functions and suitable model factors.

Doc. 5.03(2) Evaluation of test results on columns, beams and beam-columns with cross-sectional classe 4 in order to obtain strength functions and suitable model factors.

Doc. 5.04 Evaluation of test results on columnsand beam-columns with cross-sectional class IV in order to obtain strength functions and suitable model factors

Doc. 5.05 Evaluation of tests results on shear buckling in order to obtain suitable model factors.

Doc. 5.06 Evaluation of test results on web crippling in order to obtain suitable model factors.

Doc. 5.07 Evaluation of test results on hollow section lattice girder connections.

Doc. 5.08 Imperfections for compressed members.

Doc. 6.01 Evaluation of tests results on bolted connections in order to obtain strength functions and suitable model factors - PART A : Results.

Doc. 6.02 Evaluation of tests results on bolted connections in order to obtain strength functions and suitable model factors - PART B : Evaluation.

Doc. 6.03 Evaluation of tests results on bolted connections in order to obtain strength functions and suitable model factors - PART C : Test Data.

Doc. 6.05 Evaluation of tests results on welded connections in order to obtain strength functions and suitable model factors - PART A : Results.

Doc. 6.06 Evaluation of tests results on welded connections in order to obtain strength functions and suitable model factors - PART B : Evaluation.

Doc. 6.07 Evaluation of tests results on welded connections in order to obtain strength functions and suitable model factors - PART C : Test Data.

Doc. 6.08 Comparison of weld strength according to Eurocode 3 with weld strength according to national standards.

Doc. 6.09 Beam to column connection.

Doc. 6.10 Evaluation of test results on beam to column connections in order to obtain strength functions and suitable model factors.

Doc. 7.01 Procedure for the determination of design resistance from tests.

Doc. 9.01 Background document for chapter 9 of Eurocode 3.

Doc. 9.02 Report on the comparison of classification tables in existing national codes for fatigue in Europe and statistical evaluation of test data for large and small scale specimen.

Doc. 9.03 Background information on fatigue design rules for hollow sections ; statistical evaluation - Part A : Classification method

References to Volume 2 : Annexes

Doc. A.01 Evaluation of test results on connections in thin walled sheetings and members in order to obtain strength functions and suitable model factors
Part A : Evaluation and Results.

Doc. A.02 Part B : Test Data

Doc. D.01 Bakground document for design rules for high strength steels according to EN 10113.

Doc. D.02 Statistical evaluations of the results of bolted connections.

Doc. D.03 Evaluations of test results on welded connections made from FeE 460 in order to obtain strength functions and suitable model factors.

Doc. D.04 Statistical analysis of strength functions for welded H section joints with respect to available experimental data.

EC3: A Steel Eurocode for Innovative Structural Engineers

EC3: Un Eurocode en vue du projet de structures en acier innovatrices

EC3: Ein Eurocode für innovative Entwürfe in Stahl

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SUMMARY

After pointing out that the design rules in Eurocode 3 have been developed on the basis of test results some examples for such rules are given which demonstrate the compromise between sophisticated design models with close approximation to the actual physical behaviour of the structure and the user-friendly simplified design models that have been proposed. Some new design rules for frame structures, hollow section lattice structures and plate girders are presented.

RESUME

Après avoir rappelé que les règles de dimensionnement de l'Eurocode 3 ont été établies sur la base de résultats d'essais, l'article présente quelques exemples illustrant ces règles basées sur un compromis entre des modèles de calculs très élaborés, cherchant à représenter exactement le comportement physique de la structure d'une part, et des modèles de calculs simplifiés, d'utilisation pratique, d'autre part. De nouvelles règles de calcul sont présentées pour les structures en cadres, à profil creux et les poutres composées.

ZUSAMMENFASSUNG

Nach Hinweis auf die Entwicklung der Bemessungsregeln im Eurocode 3 anhand von Versuchsresultaten wird an einigen Beispielen gezeigt, wie ein Kompromiss zwischen möglichst grosser Nähe der Bemessungsmodelle zum wirklichen Verhalten der Konstruktion und einer nutzerfreundlichen Vereinfachung gesucht wurde. Dabei wird auf die neuartige Bemessung von Rahmentragwerken, von Hohlprofilkonstruktionen und von Blechträgern eingegangen.

1. INTRODUCTION

Eurocode 3 is a limit state design code in which principles and rules for serviceability limit state and ultimate limit state verifications are given.

These limit states are referred to physical phenomena as e.g.

- the exceedance of values for deflections or vibrations that may limit the serviceability or
- the collapse or other form of structural failure that may endanger the safety of people and thus be regarded as ultimate limit.

Thus the attainment of limit states can be proved by tests. On the other hand available test results can be used to develop strength functions and functions for the resistance of members and systems, and it is evident that these functions are the better the more accurately they can predict the ultimate strengths of members in tests.

This however produces a dilemma: In general a mechanical model with a good prediction (expressed in terms of a small mean value deviation and a small scatter, see fig. 1a) is more complex and sophisticated (e.g. a finite element model). The advantage of such a model is that from the test evaluation (see the paper "Eurocode 3: A Eurocode for Reliable Steel Structures) only small γ_M-values are derived that lead to economical design values with sufficient reliability. A simplier model however, in which certain unlinearities and parameters in view of better usability are omitted, may produce larger deviations between predictions and test results. It then will be punished by larger γ_M-factors and hence will be less economical, fig. 1b.

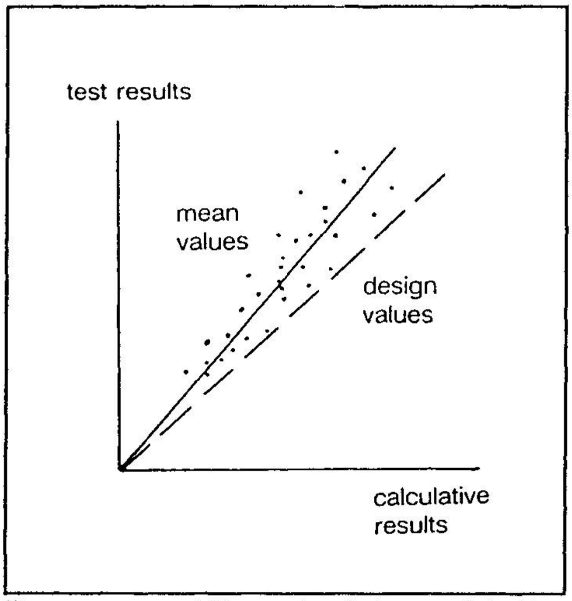


Figure 1a: Effects of a design model with a good test prediction.

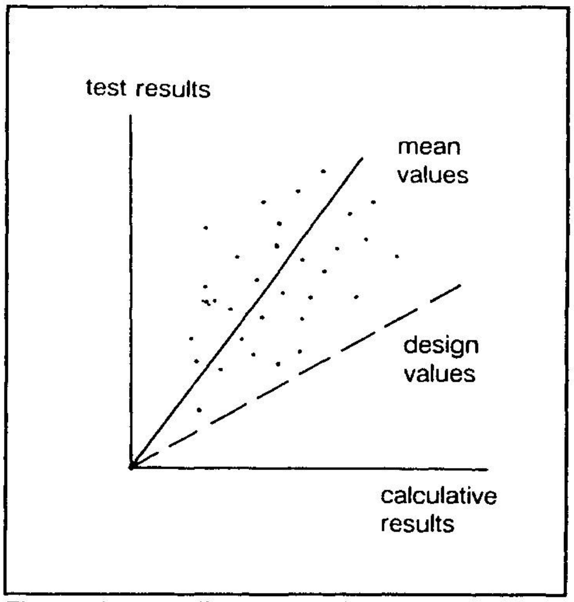


Figure 1b: Effects of a design model with a moderate test prediction.

The rules in Eurocode 3 present a compromise; the intension was to keep them accurate enough to make them economical and clear and simple enough to ensure their userfriendliness. In many cases both more detailed and more simple conservative approaches have been presented to allow the user to choose.

In any case the development of strength and resistance models on the basis of test-evaluations allowed to include the most recent world wide test results in the works and to reach the justification of new rules and rules for new types of structures and structural detailing for Eurocode 3.

In the following some examples for such rules are given. Their justification is given in the relevant background documents to Eurocode 3 [1].

2. ANALYSIS MODELS FOR FRAMES

2.1 General procedure

In general spacial frame structures may be separated into several plane frames, fig. 2, that may be considered as laterally supported at the spacial nodes.

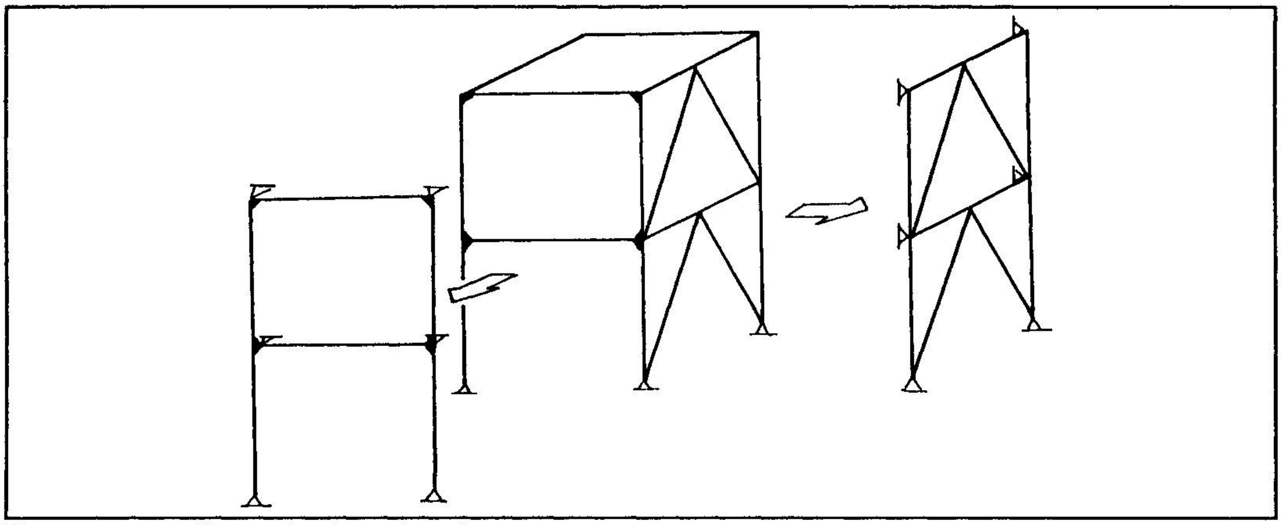


Figure 2: Separation into plane frames.

For the inplane loading of these plane frames in the first step out of plane deflections between the lateral supports are neglected and only the inplane monoaxial action effects are determined.

In the second step the individual members of the plane frame between the lateral supports, i. e. the beams and the columns, are separated from the plane frame, to consider lateral buckling and lateral torsional buckling, under monoaxial bending and compression, fig. 3. Members which are common to two different frames, e. g. columns, may be verified for biaxial bending and compression.

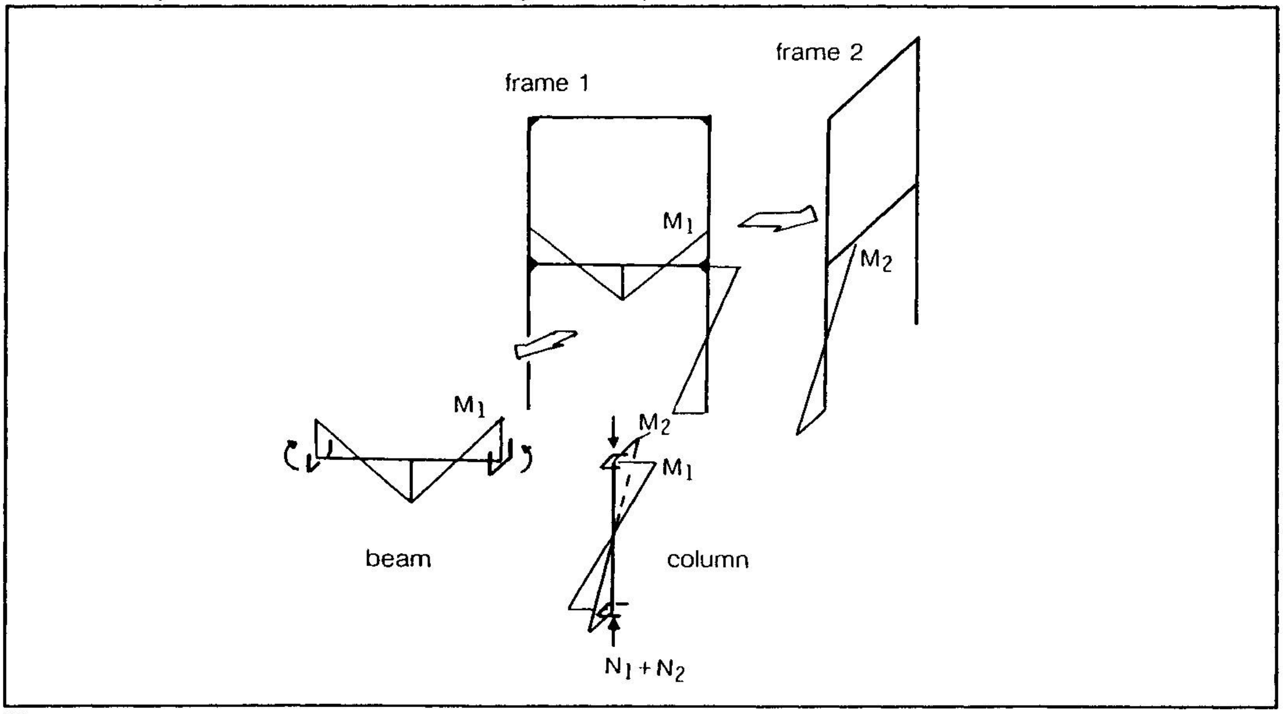


Figure 3

2.2 Plastic or elastic models

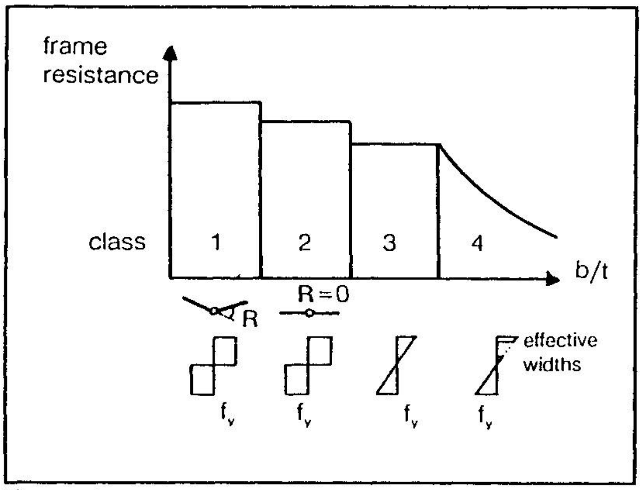


Figure 4

The model for the first step inplane analysis of the frame depends on the type of failure of the most stressed zones of the members.
The type of failure is classified by the slenderness b/t of the flanges of the cross section in compression, fig. 4. Class 1 sections allow for the use of the plastic hinge method with moment redistributions, class 2, 3 and 4 for elastic analysis methods with different levels of resistance of the cross sections.

Eurocode 3 allows to bypass the limiting b/t ratios for class 1 sections when the plastic hinge analysis is justified by an additional check of the rotation capacity R of the cross sections in plastic hinges, fig. 5. Guidance for this check is given in [2].

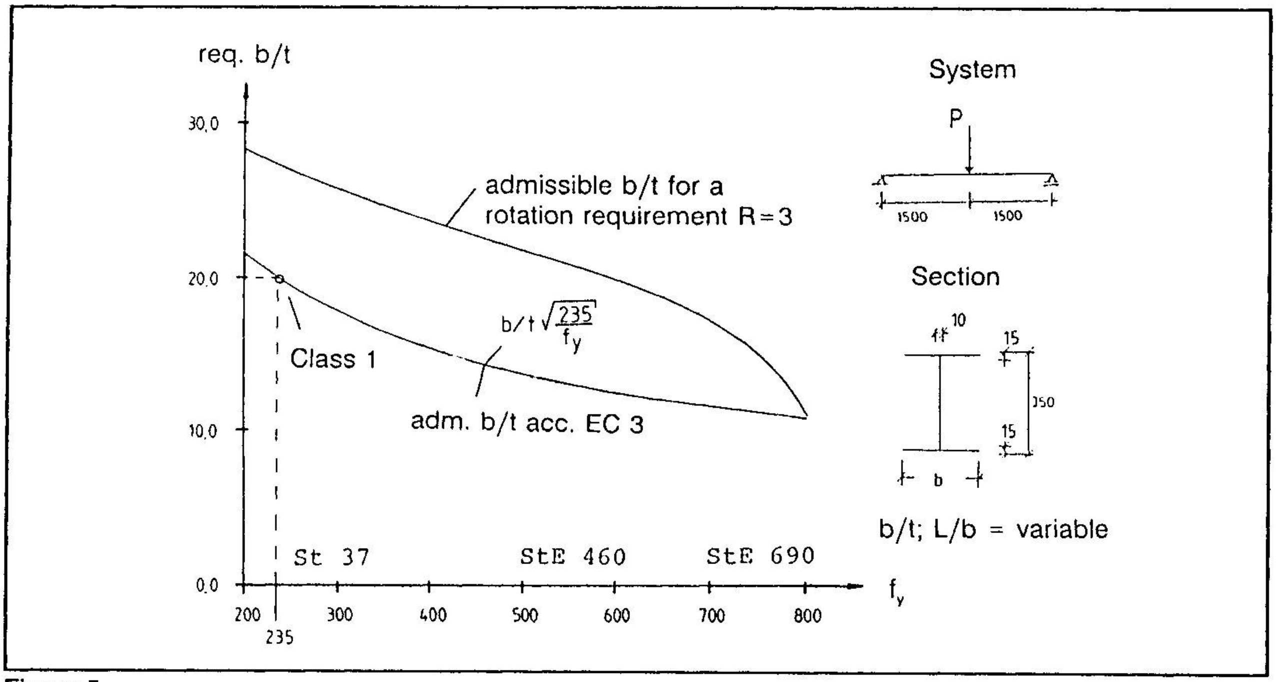


Figure 5

2.3 1st or 2nd order theory

The question whether in the first step elastic or plastic analysis 1st order or 2nd order theory has to be used can be easily answered in Eurocode 3 by considering the '10%-criterion': When from the action effects calculated with the 1st order theory a sway deformation ψ_1 is obtained that fulfills

$$\psi_1 \leq 10\,\%\,\frac{H_1}{V}$$

where H_1 = shear force in a frame storey
V = vertical load in a frame storey

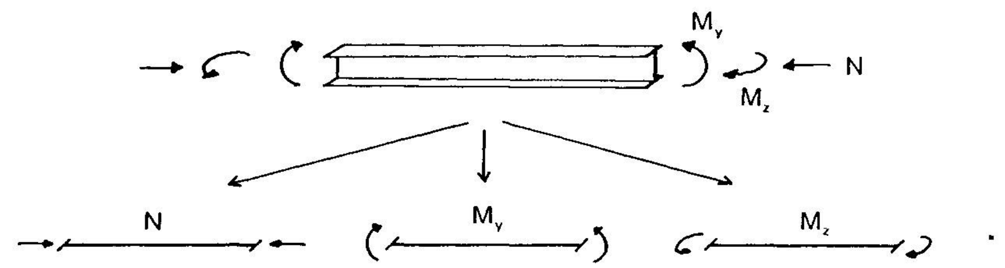
the results of the 1st order theory may be used.

This criteron also works for inhouse-structures, where no wind action has to be applied, because the shear force H_1 also contains the effects of initial sway imperfections $\psi_o \cdot V$ that have to be considered for all frames.

The 10%-criterion allows the majority of frames to be calculated with 1st order theory.

2.4 Buckling and torsional buckling verifications of members

For the buckling and torsional buckling verifications of members according to step 2 of the general procedure interaction formulae have been derived, fig. 6, that represent the best fit approximation to the design values from test results.



$$\frac{N_{Sd}}{\chi\, N_{pl,Rd}} + k_y \frac{M_{y,Sd}}{\chi_{LT}\, M_{pl\,y,Rd}} + k_z \frac{M_{z,Sd}}{M_{pl\,z,Rd}} < 1$$

column | beam 'M$_y$' | beam 'M$_z$'

Figure 6: Interaction formulae for lateral-torsional buckling.

In case the boundary conditions of members are such, that individual members cannot be separated from the frames, e. g. for portal frames where unsufficient support is given at the knee-points, see fig. 7, Eurocode 3 offers the following alternative procedure to the interaction formulae in fig. 6:

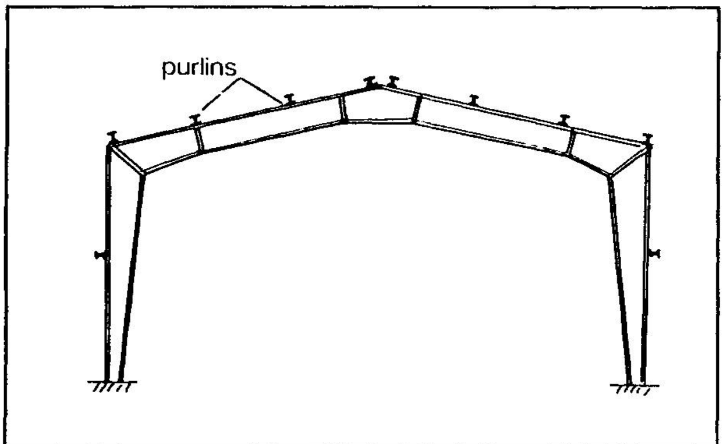


Figure 7

In the first step the inplane safety factor γ_{pl} of the frame with inplane response only (lateral deflections neglected) is calculated using the relevant elastic or plastic method (see clause 2.2). In the second step the out of plane safety factor γ_{crit} for lateral torsional buckling using a hyperelastic frame model is determined. From the overall slenderness

$\bar{\lambda} = \sqrt{\frac{\gamma_{pl}}{\gamma_{crit}}}$ of the frame, which includes the interaction effects of compression and bending moments, the reduction factor χ_c is obtained from the European buckling curve c that gives the overall safety factor $\gamma_u = \chi_c \cdot \gamma_{pl}$.

For the 2nd step of this procedure in general a computer program is needed, that takes the local restraints due to the connections with purlins and the distorsions of the cross section into account. Fig. 8 gives an example for a thin walled cross section that may be used for such frames. Detailed information on the strength verifications of such frames are given in Annex A of Eurocode 3.

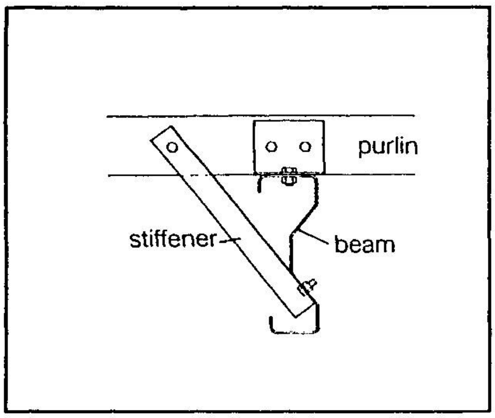


Figure 8

3. SPECIAL RULES FOR HOLLOW SECTION LATTICE STRUCTURES

Hollow section lattice structures are more and more used for roof structures, columns etc. They offer a good architectural appearance and request due to their reduced surface low maintenance.

The key for economical design of hollow section lattice structures is even more than in other areas the appropriate design of connections. Fig. 9 gives a survey on the possibilities specified in Eurocode 3 Annex K, where resistance rules are available.

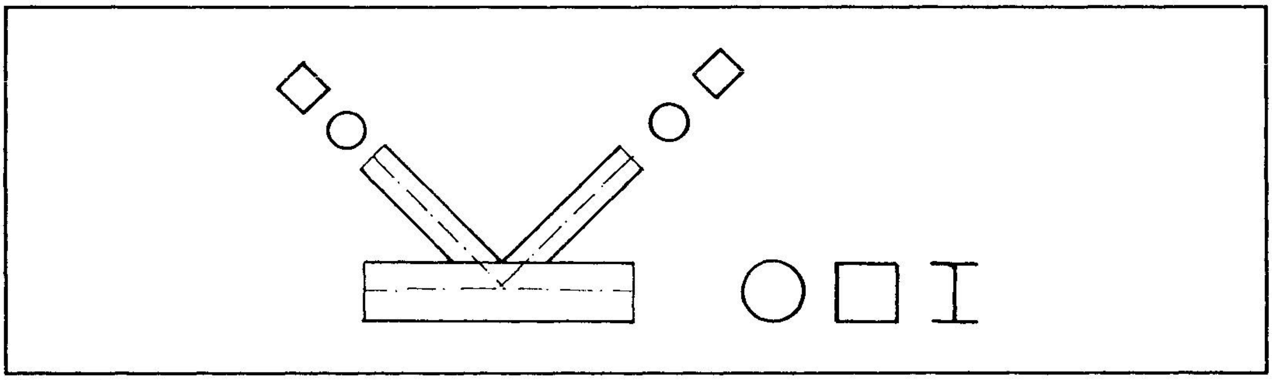


Figure 9

4. PLATED GIRDERS

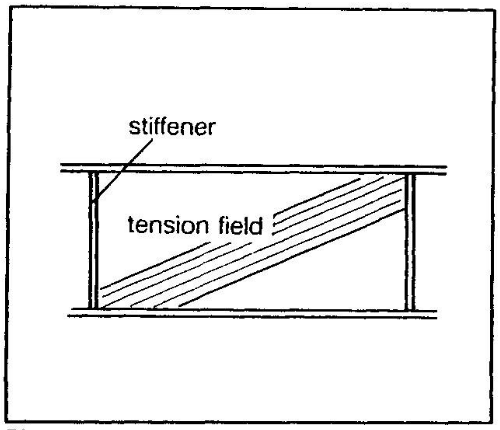


Figure 10

For plated girders two cases are dealt with in Eurocode 3:

- webs without transverse stiffeners,
- webs with transverse stiffeners.

The shear buckling resistance of webs without transverse stiffeners may be obtained by a simple postcritical method using a buckling curve $\tau = f(\bar{\lambda}_w)$

For webs with transverse stiffeners a tension field method may be used, where the total strength has two components :

- the residual shear buckling strength which is independent on any flanges
- the strength of the tension field which allows for the strengths of the flanges, fig. 10.

In the course of the development of design rules for bridges these rules are being further developed and extended.

REFERENCES

[1] CEN European Committee for Standardisation:
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[2] Spangemacher, R.: "Zum Rotationsverhalten von Stahlkonstruktionen, die nach dem Traglastverfahren berechnet werden", Diss. RWTH Aachen, 1992

EC 3: A Eurocode for Economical Steel Structures

EC 3: Un Eurocode pour des structures en acier économiques

EC 3: Ein Eurocode für wirtschaftliche Stahlbauten

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SUMMARY

In this contribution an outline is given of the design methods for connections in Eurocode 3. The design approach forming the basis for the presentation of the rules in EC 3 is discussed. Indications are given how to take advantage of the possible application of partial strength connections and semi-rigid frame design. Finally the evaluation procedure used to determine the resistances of bolts and welds is presented.

RESUME

La présentation traite des méthodes de calcul des assemblages dans l'Eurocode 3. Cette approche du projet est à base de la présentation des règles de calcul dans l'Eurocode 3. Des indications sont données sur les avantages possibles de l'application du projet d'assemblages à résistance partielle et des cadres semi-rigides. Les procédures d'évaluation en vue de déterminer la résistance des boulons et des soudures sont enfin présentées.

ZUSAMMENFASSUNG

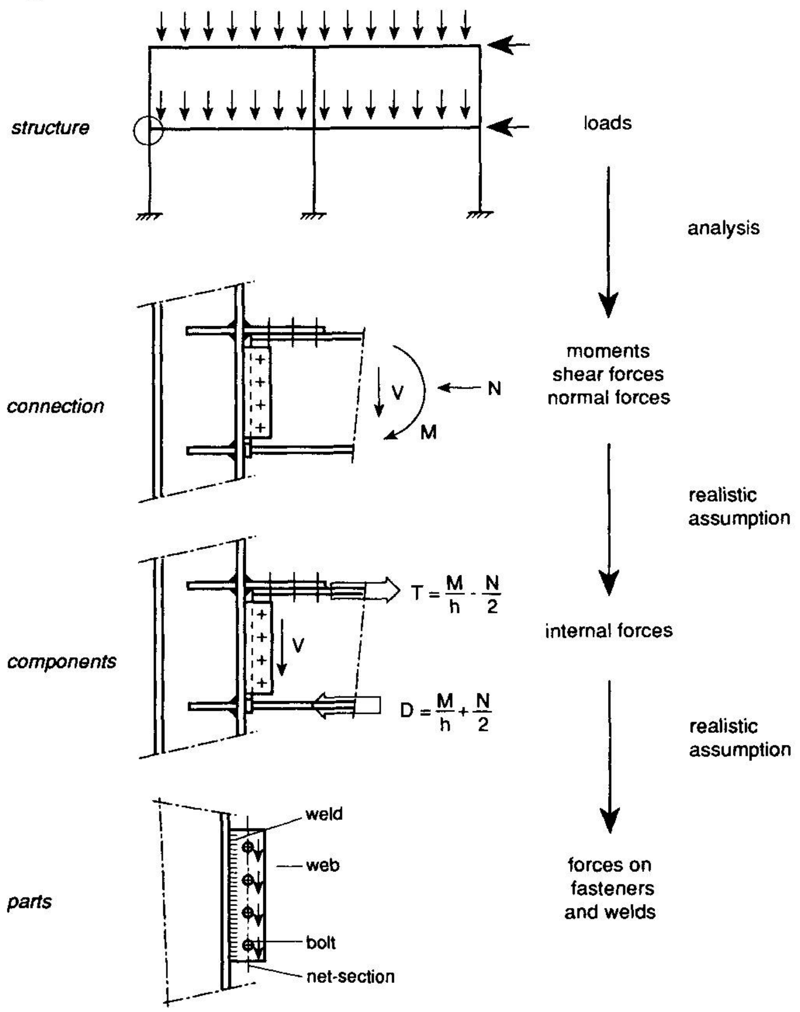
Im vorliegenden Beitrag wird ein Abriss der Bemessungsverfahren für Verbindungen in EC 3 gegeben. Das seinen Regeln zugrundeliegende Bemessungskonzept wird erörtert, und es werden Anhaltspunkte gegeben, wie mit Vorteil Verbindungen auf anteilige Widerstände ausgelegt und halbsteife Rahmen eingesetzt werden können. Schliesslich wird das Nachweisverfahren für Schrauben- und Schweissverbindungen vorgestellt.

1. INTRODUCTION

In structural steelwork the joints between the members play an important role. Evidently the properties of the joints influence the response of the structure to actions. The relevant structural properties are strength, stiffness and deformation capacity. But also from economical points of view the joints are very important. The number and simplicity of the joints influence greatly the time, required for designing and drawing.
Production of connections, cutting, drilling and welding of main members, plates, cleats and stiffeners. consumes much of the work in the fabrication shop. The ease with which the site connections can be made is a key factor for erection. So the selection, design and detailing of the connections significantly influences the total costs of a steel structure.
It is for that reason that in Eurocode 3 relatively much attention is spent to the design of connections. In this paper an outline is given of the design methods in Eurocode 3 for connections. Emphasis will be given to those rules which are expected to be of prime importance for the economy of steel structures.

2. DESIGN APPROACH

There are so many different structural solutions, even for the same type of connection that it would be impracticable to cover each separately in detail in the Code. Therefore in 6.1 of EC3 a procedure is given that essentially can be applied to all type of connections and leads to a check of individual fasteners and other parts of a connection. This procedure is illustrated in figure 1.

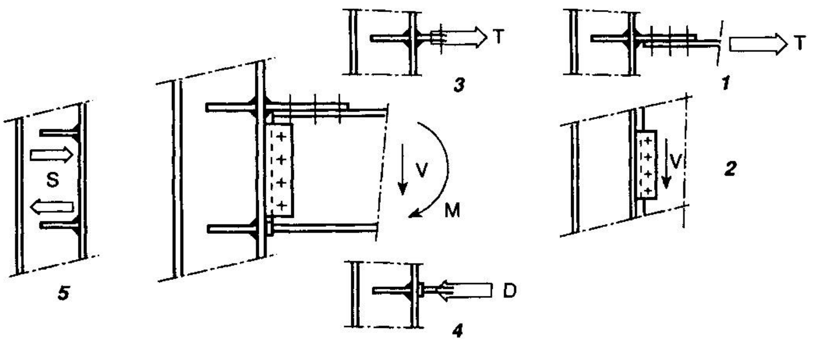


The first step is to schematise the structure. In this phase the connections must be classified as covered in 6.4 of EC3. This subject is discussed in more detail in section 3 of this paper. Then the forces and moments applied to the connections shall be determined by elastic or plastic global analysis. The next step is to determine the distribution of forces within the connection. It is not necessary, and often not feasible to determine the real internal distribution of forces. It is sufficient to assume a realistic distribution, provided that (clause 6.1.4.):

(a) the internal forces are in equilibrium with the applied loading,
(b) each element is capable of resisting the forces
(c) the deformations implied by this distribution are within the deformation capacity of the fasteners or welds and of the connected parts.

This is the most difficult part of the procedure, because this will, of necessity, entail the making of certain simplifying assumptions about the way in which the connection "works".
Account should be taken of the distribution of forces in the elements that are connected.
In addition, the assumed distribution shall be realistic with regard to relative stiffenesses within the joint. The internal forces will seek to follow the path with the greatest rigidity.
It is most important to ensure that the analysis is consisted throughout the connection.

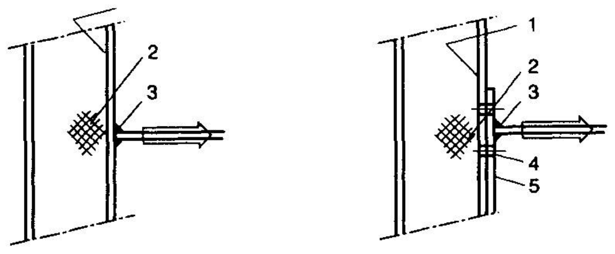
To cover the large variety of different forms of connections it is useful to use the concept of a set of basic force transfers of the type found in the component parts of many forms of connection. The basic forms are shown in figure 2.



1 Axial force
2 Shear force
3 Introduction tensile force
4 Introduction compression force
5 Shear panel

Figure 2
Basic force transfers in connections

For each of these basic force transfers a number of failure modes are possible. Each of these shall be checked and the weakest link shall be able to resist the applied load.
This is illustrated in figure 3 for the introduction of a tension force in an unreinforced web. The rules for checking the welds and the bolts (criteria 3 and 4) are covered in chapter 6.6 and 6.5 of EC3. The other criteria are covered in Annex J for beam to column connections, but as explained above parts of the method can also be applied to other forms of connections. The procedure can in principle also be used to check the other two structural properties of a connection being stiffness and deformation capacity.

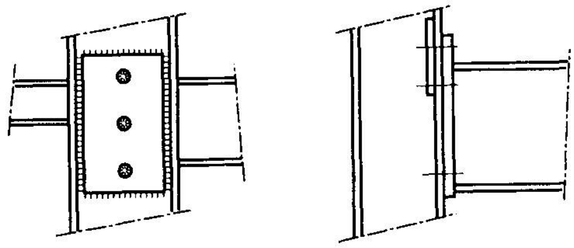


welded *bolted*

1. Excessive deformation column flange
2. Yielding/tearing column web
3. Failure weld
4. Failure bolts
5. Yielding end plate (c.q. L or T)

Figure 3
Possible failure modes for introduction of tension force (case 3 in figure 2)

In Annex J, besides traditional welded stiffeners, other often more economical means to reinforce or stiffen connections are treated. These are supplementary web plates and backing plates as shown in figure 4.



supplementary web plate *backing plates*

Figure 4
Alternative solutions to reinforce connections

3. CLASSIFICATION OF CONNECTIONS

The section 6.4 on classification of connections in Eurocode 3 has given the most controversial reactions. This is probably caused by the first impression that existing standard practice for modeling of connection behaviour would not be allowed anymore. This is certainly not the case. In the past, when "working stress" design was normally used, the connection design was based on rather simple though not necessarily economical assumptions.
The connections were assumed to behave either as hinges (simple construction) or as infinitely stiff (rigid construction). The forces on the connections then followed from an elastic analysis of the structure. The parts of the connections such as end plates and angles, welds and bolts, could subsequently be dimensioned. Even now this design procedure seems to be used in the majority of cases. It was not the intention to abandon this procedure in EC3 but to give the designer, as a matter of choice, an alternative to use connections having properties in between the two extreme cases.

This is important because the introduction of limit state design, including practical rules for plastic design, requires a more realistic treatment of the connections. When using these methods, the designer is confronted directly with the fact that for a better insight into topics such as the stability of columns and frames and for a minimum cost design of members and connections, understanding of the behaviour of connections is essential.
Another factor is that modern computer programs, now available to the majority of designers, allow a more sophisticated treatment of connections without an appreciable increase in calculation costs. Finally, also the use of automatic NC drilling and sawing equipment in the fabricators shop influences the cost relationship between various forms of connections, leading to a need to minimise the number of welded stiffeners and to use other means to reinforce or stiffen connections.
Now two situations will be discussed where a designer may decide to alter the traditional way of modeling beam-to-column connections.
An important factor for this choice is whether the frame is braced or unbraced. Let's consider now both cases.

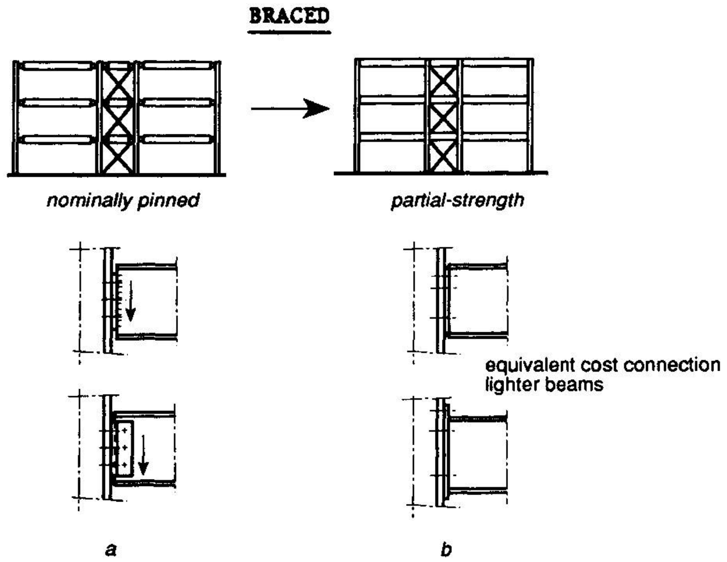


Figure 5
Connections in braced frames

In a braced frame it is possible to design the connections as hinged. Stiffness is not required for stability. Two possible forms of so-called nominally pinned connections are shown in figure 5 (a). The connections with a short-end plate or web cleats are designed only for transfer of shear force. By using detailing rules based on experience and test evidence the designer takes care that the flexibility and deformation capacity is sufficient. In 6.9.6.2(2) of EC3 this procedure is explicitely allowed for. In figure 5 (b) two forms of connections are shown which are not much more expensive to fabricate. Although these connections are normally not full-strength they offer some end restraint to the beam, so giving more load carrying capacity and less deflection. Therefore often a smaller beam section will do. Apart from the material savings this also will reduce construction depth. An additional advantage may be that the frame has allready some stiffness during erection before the bracing system has become effective. By using plastic design and assuming partial-strength connections the design is even simpler than the traditional elastic design.

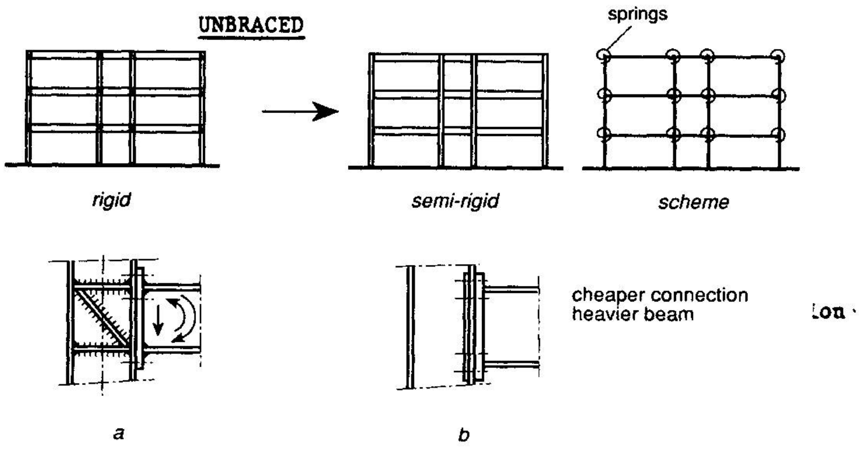


Figure 6
Connections in unbraced frames

If the frame is unbraced stiffness of the connections is essential for stability. Unreinforced connections are normally more flexible than the connected elements. However this is not unacceptable provided that the reduced stiffness is compensated by the choice of heavier members. The optimum is dependent of the relation between material cost and fabrication cost. For the analysis of the structure the connections are represented by fictitious structural elements at the ends of the members or by springs. In Annex J of EC3 rules are given to determine the stiffness of joints.

4. RESISTANCES OF BOLTS AND WELDS

In the drafting process it became clear from a comparison of rules in national standards that the various national rules for bolts and welds showed considerable differences. As an example the extreme values of the tensile strength of a simple element as a bolt differed by more than a factor 2. It was found that many of the rules in the national standards were based on engineering judgement more than a consistent evaluation of experimental evidence. This of course can not form the right basis for the determination of design rules in a harmonisation process. For that purpose an objective procedure was needed. Fortunately the Eurocode Coordination Group had developed a semi-probabilistic limit state verification to be used in level-I codes. In a level-I code the verification of the ultimate limit state is expressed by the condition that the design effect of loads and other actions on the structures will not exceed the design resistance.

$$\text{Effect of actions} \longrightarrow \boxed{S_d \leq R_d} \longleftarrow \text{Design resistance}$$

$$\gamma_f \, \gamma_{S_d} \, S_k \leq \frac{R_k}{\gamma_m \, \gamma_{R_d}}$$

$$\gamma_F \, S_k \leq \frac{R_k}{\gamma_M} \longleftarrow \text{evaluation of tests}$$

Basis for each side of the expression are the characteristic values for action effects and resistances S_k and R_k respectively.

Also on both sides partial safety elements, so called γ factors are introduced to arrive at the required safety level. Based on the proposed verification procedure of the Coordination Group the Eurocode 3 Drafting Panel developed a procedure for the determination of characteristic values, design values and γ_M values for resistances from test results [1]. This method is successfully used for the determination of design rules for bolts and welds and beam to column connections.
The evaluation procedure goes along the following lines.
Based on observation of actual behaviour in tests and on theoretical considerations, a "design model" is selected, leading to a strength function. The efficiency of the model is checked by comparing the theoretical results from the strength function with available results of tests.
The design model has to be adapted until the correlation of the theoretical values and the test data is sufficient. The accepted strength function can then be used to derive an expression for the <u>characteristic resistance</u> R_k.
The characteristic resistance is defined as having a 5% probability of not being exceeded for a level of confidence of the prediction of 75%. The procedure also includes a method to derive <u>design values</u> from the given data and hence to deduct γ_M-factors, to be applied to the relevant characteristic strength functions. The value of γ_M is dependent on the required failure probability determined by the safety index β (for the ultimate limit state normally $\beta = 3.8$). For practical reasons the coefficients in the design functions derived from the statistical evaluation were modified slightly, to enable a single value of $\gamma_M = 1.25$ to be recommended for all cases.
The total collection of the results used for the evaluation amounted about 2000 for bolts and about 500 for welds. More details are given in [2], [3], [4]. An overview of the design capacity is given in Table 1.

Table 1 Design resistance for bolts $\gamma_{Mb} = 1.25$

<u>Shear resistance per shear plane:</u>

if the shear plane passes through the threaded portion of the bolt:

- for strength grades 4.6, 5.6 and 8.8:

$$F_{v.Rd} = \frac{0{,}6\, f_{ub}\, A_s}{\gamma_{Mb}}$$

- for strength grades 4.8, 5.8, and 10.9:

$$F_{v.Rd} = \frac{0{,}5\, f_{ub}\, A_s}{\gamma_{Mb}}$$

if the shear plane passes through the unthreaded portion of the bolt:

$$F_{v.Rd} = \frac{0{,}6\, f_{ub}\, A}{\gamma_{Mb}}$$

<u>Bearing resistance:</u> *)

$$F_{b.Rd} = \frac{2{,}5\, \alpha\, f_u\, d\, t}{\gamma_{Mb}}$$

where α is the smallest of:

$$\frac{e_1}{3d_o} \; ; \; \frac{p_1}{3d_o} - \frac{1}{4} \; ; \; \frac{f_{ub}}{f_u} \text{ or } 1.0.$$

<u>Tension resistance:</u>

$$F_{t.Rd} = \frac{0{,}9\, f_{ub}\, A_s}{\gamma_{Mb}}$$

A is the gross cross-section area of bolt
A_s is the tensile stress area of bolt
d is the bolt diameter
d_o is the hole diameter

After presentation of the evaluation results to the Liaison Engineers and to members of ECCS-TC10 the question was raised whether the new formulae could adversely affect the economy of steel structures. Therefore it was decided to carry out calibration studies. The results according the proposed EC3 strength functions were calibrated against results according to national standards [5], [6].
It was found that EC3 was more liberal than most of the existing codes.
Just as an example two points are raised to show that the EC3 rules for bolts and welds allow for more economic steel structures.

- The end distances and the pitch may be chosen freely within certain limits. Of course the choice of small distances must result in lower bearing resistances. But still this possibility can be of great interest to avoid gusset plates and allow for compact design of joints.
- It is allowed to use bolts with the threaded portion in the shear plane. This allows the use of fully threaded bolts, leading to considerable reduction of bolt types to be kept in stock, improving efficiency and reducing the potential for error.

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EC3: A Steel Eurocode for Practical Structural Engineers

EC3: Un Eurocode d'utilisation pratique pour les structures en acier

EC3: Ein Eurocode als eine praxisorientierte Stahlbaunorm

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SUMMARY

The Eurocode 3 has been drafted, within the agreed parameters for Eurocodes, to be an user-friendly aid for practical structural engineers. The success of this is confirmed by a report on comparative design studies. Some of the editorial features adopted for useability are described. Mention is also made of various design aids, including two simplified versions of the code.

RESUME

Dans les limites des paramètres des Eurocodes, l'Eurocode 3 a été préparé afin d'en permettre une utilisation simple par l'ingénieur civil de la pratique. Le succès de cette réalisation est confirmé par des études comparatives de projets. Quelques caractéristiques de présentation pratique et simplifiée sont décrites. L'article mentionne quelques aides de calcul, y compris deux versions simplifiées de la norme.

ZUSAMMENFASSUNG

Innerhalb der Vorgaben für Eurocodes wurde der EC 3 als anwenderfreundliches Instrument für praktisch tätige Ingenieure entworfen. Den Erfolg dieser Bemühungen belegt ein Bericht über vergleichbare Bemessungsstudien. Einige wichtige Merkmale für den leichteren Gebrauch werden beschrieben. Ferner werden verschiedene Bemessungsmittel erwähnt, darunter zwei vereinfachte Versionen der Norm.

1. INTRODUCTION

The ultimate purpose of any design standard must be to serve as a tool for the practical structural engineer. Today a modern Euro-Engineer will recognise the need for:

- Limit States philosophy, practically applied
- Computer Aided Design, in appropriate cases
- a personal Data Bank to access relevant criteria, from the wealth of information available
- procedures for design and for execution, strictly governed by Quality Assurance criteria

It is against this background the new Eurocodes should be judged.

2. CONTENT AND SEQUENCE

The design sequence adopted in EC3 seems to be both sensible and realistic for practical application : first the basic assumptions and general structural analysis, then verification of sections and of member stability, finally connection design. This suits both the organization of work in the design office and the presentation of formal calculations for approval.

All generally recognised methods of analysis are permitted, under adequate safeguards, including plastic and elastic-plastic methods, thus setting designers free to choose what is most appropriate to a given situation. Similarly all types of joint, including semi-rigid, are permitted, provided that the details are consistent with the design assumptions. Practical and robust structures are encouraged by the explicit recognition of the need to consider practical tolerances in construction. Straightforward rules include such considerations in the structural analysis, and these rules are linked to the specification of reasonable but strict tolerances. These tolerances are outlined for designers in the EC3 Chapter on Fabrication and Erection and then expanded in detail in the forthcoming companion CEN Standard "Execution of Steel Structures".

Recognition is given to the practical role that may be played by testing of actual full-scale components in appropriate cases. However the need for testing of prototypes is an exception and for general routine work the traditional practice of design by calculation alone is retained.

The inclusion of a Chapter on Fatigue in Part 1 of EC3 arises from its dual role, comprising General Rules as well as Rules for Buildings. EC3: Part 1 is intended to be complete only for Buildings, but its contents are also intended to be valid, whilst not necessarily complete, for any other structures such as bridges, towers or silos. It is not intended to imply that buildings generally need a fatigue check, but rather it is provided for use in those limited specialised cases where fatigue may be a relevant design criterion. The Fatigue rules are in Part 1 because they apply to all types of structure. The subsequent parts of Eurocode 3 will cover in detail any additional rules that are specific to particular types of structures.

3. PRECEDENTS

Whilst EC3 itself has only recently become available, (though the work started 15 years ago and various drafts have circulated), a major source for its technical contents has been the pre-normative work represented by the long-range activities of the ECCS Technical Committees, as well as the pioneering pre-harmonisation of the ECCS Recommendations, originally addressed to national code-writing bodies but in the event the fore-runner of this European Standard.

The ECCS Recommendations were in fact published in 1978 and were discussed both before and after that date in various international meetings. The involvement of Engineers from the industry in all ECCS activities, along with researchers and academics, has served to ensure that the resulting design rules are useable and lead to practical solutions.

Similarly the influence on a worldwide scale of the Structural Stability Research Council (SSRC) can be detected within the contents of Chapter 5 Ultimate Limit States, albeit with a European rather than an American flavour.

The only real deficiency is the temporary absence of many of the CEN Reference Standards needed for the application of EC3. As an interim measure this has been covered by the National Application Documents (NAD) issued to permit the trial use of the ENV version of EC3 : Part 1. These give alternative equivalent national standards and also specify the required national values for the partial safety factors.

4. CALIBRATION AND TRIAL CALCULATIONS

The calibrations to determine recommended "boxed" values for the partial safety factors are a topic for another contribution. However at least in some cases comparisons with existing practice in various countries were also made, in collaboration with ECCS Technical Committees.

Useability aspects were also checked by trial calculations. Some were ad-hoc, but at the public enquiry stage some countries carried out complete design examples which were also compared to the use of their national codes.

Considerations of completeness and useability were the principal topics for complete design studies carried out as the final editing of EC3 progressed, by CEDIC, the then existing organisation of Consulting Engineers for the EEC countries, now superseded by EFCA which also covers the EFTA countries. A number of recently completed buildings in a variety of countries were chosen and the design was re-checked according to EC3. These buildings were selected so as to be reasonably challenging in terms of testing the availability of appropriate code provisions, whilst also being reasonably representative of best modern practice.

The development of CEDIC's reports on these studies interacted with the development of the final editing of EC3, so that comments made in their initial report were found to have already been acted upon, in terms of code improvement, by the time the final report [1] was prepared. Thus the final conclusion could be reached that EC3 is "sufficiently clear, transparent and comprehensible for practising engineers".

One result of this has been that the National Application Documents for the trial use of the ENV version of EC3 Part 1 mostly call for only marginal variations to the recommended "boxed" values of the partial safety factors. There is thus real hope that harmonised values can eventually be achieved by voluntary agreement, after the EC1 harmonised loadings are available.

A further testimony to the practical nature of EC3 is the adoption of large portions of its contents into proposed new versions of national standards, both in Switzerland and in the Netherlands.

It would of course be foolish to suggest that any new code is perfect or not capable of further improvement. The drafters of EC3 are at least as conscious as anyone else of the need for review. The 3 year ENV period offers the possibility of extensive trial use in practical design situations. It is greatly to be hoped that practising designers will take advantage of this opportunity to try out EC3 in current design work and to feed back their experience (whether good or bad) with any suggestions for improvements.

5. USEABILITY ASPECTS

The CEDIC report also expressed satisfaction with "the division....(of the text)....with some of the extensive calculation procedures presented as annexes". This has been adopted for certain aspects which are not necessarily applicable to every design, in order to aid the clarity and ease of locating the relevant basic requirements. This has proved to be such a practical feature that it has been suggested it could be taken even farther, for example by moving "Built-up members" into an annex, and perhaps this will be considered at the next stage of development.

A similar motive of improving useability, by keeping lengthy detailed procedures out of the main text, has led to the use of "Figures" containing text and formulae as well as diagrams (or even without any diagrams) to describe the necessary provisions, such as those for "average yield strength" of cold-formed hollow sections or for "stabilizing forces" in bracing systems. This too is an editorial device which could perhaps be used even more extensively in the future development of this — and maybe other — Eurocodes.

The detailed Application Rules necessary to develop the Principles of economic joint design for semi-rigid beam-to-column connections and for the connections of tubular ("hollow section" members are particularly extensive. In the case of beam-to-column connections, tabulated "Procedures" have been introduced to clarify exactly how the various checks are intended to fit together into a logical and efficient sequence of design steps.

In the case of tubular connections, tables illustrated by sketches have been used to list the design expressions relevant to various cases. This recognises the ability of designers to identify the item they require more readily by reference to sketches. A comparable approach has also been adopted for the determination of "detail categories" for Fatigue, where the appropriate table is selected by reference to the title, but the numerical values are selected by reference to sketches, amplified by descriptive text.

An aspect which should add to the useability of this code compared to others, once designers gain familiarity with its novel features, is the inclusion of a specific method for examining the adequacy of a bracing system in terms of both its resistance to stabilizing forces and its stiffness, including the influence of this stiffness on the magnitudes of the stabilizing forces to be resisted and the effects of external loads on the efficiency of the bracing system.

Other features that may be considered as useful, depending on which other code the Eurocode is compared with, include the following provisions:

- simple alternative method for load combinations
- alternative method for resistance of a fillet weld
- economic alternative method for shear buckling resistance
- guidance on choice of steel quality to avoid brittle fracture
- specific fabrication and erection tolerances
- guidance on slip resistant connections with high strength bolts

All these aspects, plus several of the features of EC3 already mentioned, including systematic treatment of frame imperfections, choice of analysis methods, procedures for connection design etc, are listed as potential advantages for the designer in the CEDIC report. Of course there is more still that could be done, as in any code, but the needs of the user have been a major consideration throughout the preparation of Eurocode 3, within the constraints of the agreed style and format, the need to adopt common Chapters in all Eurocodes and the editorial rules of CEN.

6. SIMPLIFIED VERSIONS

6.1 Introduction

Several people, including the writers of the CEDIC report, have identified the need for measures to assist the introduction and acceptance of the Eurocodes into everyday design practice. Whilst the comprehensiveness and freedom of choice for the designer are welcomed, the need is seen to give simple guidance on the appropriate methods to adopt in common cases, at least until designers become more familiar with those options and features that are novel for them.

6.2 Essentials of Eurocode 3

The ECCS have published a shortened version of EC3, together with additional tables and other practical information and design aids, in their publication No 65, "Essentials of Eurocode 3 : Design Manual for Steel Structures in Building" (E-EC3) [2]. It is intended as a design aid to facilitate the use of EC3 : Part 1 during the ENV period, and contains only those rules "that are likely to be needed for daily practical design work". This has led to the omission of plastic analysis, second-order analysis and semi-rigid joints. As an ECCS document, although produced initially in English, it is written on a pan-European basis; remaining as general as possible, but giving lists of "boxed" values extracted from available NAD's.

E-EC3 is intended to be used by designers who have studied at least the relevant portions of EC3. It is intended to serve as an aide-memoire both for the essentials of the Eurocode provisions themselves and for other necessary design information, including tables and figures which can be treated as "deemed to satisfy" the rules of EC3. In all cases of doubt, or for items not covered, EC3 and the relevant NAD must be consulted. E-EC3 is not intended to be used independently of the Eurocode itself.

6.3 Concise Eurocode 3

"The Concise EC3" (C-EC3) [3] is a different type of simplified version published by The Steel Construction Institute (SCI). It has a number of features in common with E-EC3 and the two drafting groups worked in close collaboration. The emphasis is however different, in that C-EC3 is a shortened version of EC3, limited in its scope to cover only those types of building structures that can currently be designed using a modern national code. It excludes frames where second-order analysis is necessary and does not cover elastic-plastic analysis or semi-rigid joints.

The C-EC3 is a self-contained, stand-alone design code. Its purpose is to introduce designers to the provisions of EC3 by building on familiar ground. Within its own more limited scope it can be used independently of EC3, yet it will produce designs that also comply fully with the Eurocode itself. However it is not intended as a complete substitute for EC3 and direct reference to the Eurocode will be more appropriate in cases where the need for maximum economy warrants the use of the most refined available approach.

The initial version of the Concise version, like the Essentials, has been produced in English; unlike the ECCS document the initial version of the SCI document is tailored specifically for designers in the UK. It incorporates all the requirements of the UK NAD as well as the values for all the boxed values applicable to buildings to be constructed in the UK. It is intended to also produce versions for use in other countries, depending on demand and other logistic considerations.

The Concise version contains only design rules, presented in easy-to-use form, but not design guidance or design aids as such, as these will be available separately. To maximise the ease of transition for designers currently accustomed to British Standards, the Eurocode wording has been modified. However the Eurocode symbols and axis conventions have been adopted.

Some of the clauses have been re-sequenced; in particular in Chapter 5, the main design chapter, the clauses have been grouped according to type of member (beams, columns etc) as in a British Standard, rather than according to phenomenon (cross-section yielding, member buckling etc) as in Eurocode 3. Generally however the C-EC3 follows the sequence in which topics appear in EC3 in order to help users to become more familiar with the Eurocode. It is hoped that as familiarity and confidence increase, users of the C-EC3 will soon wish to progress to the full EC3 in order to take advantage of its more advanced and economic methods in appropriate cases.

A number of user-friendly figures and tables have been included giving ready-reference values, as in a British Standard, such as strut buckling stresses corresponding to the normal slenderness ratio ℓ/i. Such values do of course need to be specific to each strength grade of steel and even though only two grades are included, the resulting tables are fairly voluminous (which is why a more compact treatment is used in EC3), but are liked by designers, at least in the UK.

7. DESIGN AIDS

For the cost-effective application of any set of design rules, various tables and other types of design aids are generally agreed to be important for efficiency. Traditions vary even within one country, and particularly between countries. However the following hierarchical list of possible design aids is probably fairly representative of an acceptable set of design documents:

- Simple code for everyday design [E-EC3 Essentials or C-EC3 Concise EC3]
- Design Guides for Buildings [Various Design Guides]
- Design Aids for Beams, Columns etc. [Design tables and charts]
- Section Dimensions and Properties [Tables of dimensions and properties]
- Design Examples [Simple, complete and practical examples]
- Explanatory Commentary [Background Documents etc]
- User's Handbook [Design manual]
- Design Textbooks [Educational material based on the Code]

Most, if not all, of these are already available or soon will be, at least in some countries. Design examples have been prepared by ECCS, CIDECT and SCI and the other usual national sources. In addition international teams have been collaborating on the production of user-friendly design software. The fact that EC3 has been found amenable to the production of all these documents and computer programs is a further indication that even at the present ENV stage it is a well-ordered document, well suited for use by practical structural engineers.

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