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EC 3 : THE NEW EUROCODE FOR STEEL STRUCTURES ; Review of Design Philosophy and Limit State Principles

Patrick J DOWLING*
Professor of Steel Structures
Marios K CHRYSSANTHOPOULOS*
Lecturer
* Department of Civil Engineering
Imperial College of Science, Technology and Medicine
University of London

ABSTRACT

The paper reviews the background, scope and design philosophy of Eurocode 3. Emphasis is given to the safety checking format and the method for deriving design values and partial factors. The principles adopted in selecting appropriate limit states are outlined, together with some special features of the code. Complementary documents are identified and the way in which they help improve the user-friendliness of the code for a wide range of potential applications is highlighted.

1. BACKGROUND

By their very nature, structural codes are subject to periodic revision and amendment. From time to time, existing design rules are modified to incorporate significant research findings or new rules are introduced to allow for more advanced or novel methods of analysis and construction. In the decade 1970～79, there was a move towards limit state codes and the replacement of single safety factors by a set of partial factors, often quantified through the use of structural reliability and stochastic load combination. In addition to these important factors, in the last fifteen years there has been a flurry of activity in Europe dedicated to the development of Structural Eurocodes.

A coherent system of Eurocodes is of vital importance to the construction industry. Harmonisation will help remove artificial barriers to trade between member states and will go some way towards reducing the effects of geography, local materials and skills, and legislative/procedural matters and customs. Perhaps more importantly, it will increase the opportunities for work in the rest of the world, whilst, at the same time, allow easier access to the European market for competitive overseas constructors.

The new European Prestandard for the design of steel structures, ENV 1993-1-1[1], generally referred to as Eurocode 3 or EC 3, 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. Reference[2] was the principal source document, which formed an appropriate starting point for the ad-hoc drafting panel, chaired by the first author, nominated by ECCS to the European Commission in 1981. Since that time, various stages of development have taken place, see[3)～5)], culminating in the publication by CEN of the ENV 1993-1-1[1]. The document is now being used on a "trial basis" so that experience is gained with its use in real situations. Although there has already been a good deal of public exposure in the eight years preceding the publication of the ENV, it is vital that in the current phase all sectors of the construction industry continue to play a part so that a properly balanced code can be implemented with mimimum disruption.

2. SCOPE

Part 1.1 of EC 3 contains principles which are valid for all steel structures as well as detailed application rules for ordinary land-based buildings. Appendix A of this paper presents a list of chapter and annex titles. Many of the application rules will also be cross-referenced for other structures which will be dealt with in subsequent parts of EC 3 but the rules in Part 1.1 are only considered complete in respect of buildings. The remaining parts of EC 3 will cover bridges and plated structures, towers, masts and chimneys, tanks, silos and pipelines, piling, crane structures, marine and maritime structures, agricultural structures and fire resistance.

Part 1.1 currently covers only three grades of structural steel with nominal yield stresses of 235, 275 and 355 N/mm^2 which are modified at thicknesses of 40 and 100 mm. Annex D, to be issued in 1993, extends the application to 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.

The design procedures 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 implicit in strength rules are directly related to these criteria and, therefore, the rules are invalid if these tolerances are exceeded. It is worth noting that EC 3 specifically requires the provision of a "Project Specification" containing details of any special requirements for materials, fabrication and erection, along with the usual design drawings.

3. BASIS OF DESIGN

EC 3 can claim to be one of the most extensively calibrated and cross-checked standards ever written for the design of steel structures. Almost all of the 18 European Economic Area countries (EEC and EFTA) have contributed to ensuring that the code has been subjected to extensive testing. Furthermore, EC 3 adopts modern principles in matters of structural safety based on limit state design (as opposed to the traditional allowable stress design) and probabilistic concepts within the framework of a Level 1 reliability code format.

At the very heart of the code lies the safety checking format, which controls the way in which the various clauses of the code lead to the desirable level of safety of structures designed to the code. In this area, EC 3, like all the other Eurocodes, draws from EC 1 Part 1 : Basis of Design[6], which, in turn, is compatible with ISO 2394[7]. EC 1 sets out a common basis for defining design rules relevant to the construction and use of all buildings and civil engineering works. It is based on the limit state concept used in conjunction with a partial factor method.

The code format relates to the number of design checks required, the rules for load combinations, the number of partial factors and their position in design equations, as well as whether they are single or multiple valued, and the definition of characteristic or representative values for all basic design variables. In principle, there is a partial factor associated with each design variable to cater for physical, statistical or model uncertainties. Furthermore, the number of load combinations can become exceedingly large for structures subjected to a variety of permanent and variable actions. Thus, in practice, it is of paramount importance to reduce the number of partial factors and load combinations while, at the same time, ensuring an acceptable range of safety level and an acceptable economy of construction.

3.1 Structural Safety Concepts

Verification of a structure with respect to a particular limit state is carried out via a mathematical model describing the limit state in terms of a function, called the limit state function or failure function, whose value depends on all relevant design parameters (load*, strength, geometry). In general terms, attainment of the limit state can be expressed as

$$g(\boldsymbol{S}, \boldsymbol{R}, \boldsymbol{L}) = 0 \quad \cdots\cdots (1)$$

where $\boldsymbol{S}$, $\boldsymbol{R}$ and $\boldsymbol{L}$ represent sets (vectors) of load, strength and geometry variables. Conventionally, $g(\boldsymbol{S}, \boldsymbol{R}, \boldsymbol{L}) \leq 0$ represents failure. Since $\boldsymbol{S}$, $\boldsymbol{R}$ and $\boldsymbol{L}$ are subject to physical, statistical and model uncertainties, they can be described as random variables (this being the simplest possible probabilistic representation, whereas, to represent reality, more advanced models might be appropriate, such as random fields). In this context, failure is a probabilistic event and it is important to be able to calculate its associated probability of occurrence, P_f

$$P_f = 1 - \mathcal{R} = \mathrm{Prob}\{g(\boldsymbol{S}, \boldsymbol{R}, \boldsymbol{L}) \leq 0\} \quad \cdots\cdots (2)$$

where, $\mathcal{R}$ is the complementary event, i.e. the reliability of the structure in respect of the particular limit state considered.

The limit state surface, $g(\boldsymbol{S}, \boldsymbol{R}, \boldsymbol{L}) = 0$, can often be separated into one resistance function, $r(.)$, and one loading function, $s(.)$, in which case the design condition can be expressed by

$$r(\boldsymbol{R}, \boldsymbol{L}) - s(\boldsymbol{S}, \boldsymbol{L}) = 0 \quad \cdots\cdots (3)$$

Within a Level 1 code, such as EC 3, $\boldsymbol{S}$, $\boldsymbol{R}$ and $\boldsymbol{L}$ are treated as deterministic quantities but their particular values have been obtained using past experience and probabilistic verification. This process takes into account various uncertainties and ensures that target reliabilities with respect to various limit states are satisfied. The actual values used for $\boldsymbol{S}$, $\boldsymbol{R}$ and $\boldsymbol{L}$ in eq. (3) are called "design values". The term "design values" originates from a particular type of analysis used for probabilistic verification, as will be explained below. In other limit state codes, the term "factored value" has

The term "load" is being used here with essentially the same meaning as the term "action"; the latter has been introduced to cover also the effects due to imposed deformation and being used in both EC 1 and ISO 2394.

been used with, broadly, the same meaning. Typically, the design value of a particular variable Z_i is obtained from the following expression

$$z_{di}=\gamma_i z_{ki} \quad \cdots\cdots (4.a)$$

$$z_{di}=z_{ki}/\gamma_i \quad \cdots\cdots (4.b)$$

where z_{ki} is a characteristic value and γ_i is a partial factor. Eq. (4 a) is appropriate if Z_i is a loading variable, whereas eq. (4 b) applies to resistance variables, so that in both cases γ_i has a value greater than unity. A characteristic value is defined in terms of a prescribed probability of not being exceeded for loads, or attained for resistances. As will be seen below in the treatment of load combinations, a value other than the characteristic may be introduced for loading variables. Furthermore, for material properties a specified or nominal value is often used as a specified characteristic value.

Partial factors account for the possibility of unfavourable deviations from the characteristic value, inaccuracies and simplifications in the assessment of the resistance or the load effect, uncertainties introduced due to the measurement of actual properties by limited testing, etc. It is clear from eq. (4 a) and (4 b) that a particular design value z_{di} may be obtained by different sets of z_{ki} and γ_i. The precise relationship between these interrelated parameters is discussed in more detail below, in the context of a particular probabilistic method which can be used to obtain suitable values for partial factors.

The process of selecting the set of partial factors to be used in a particular code could be seen as a process of optimization such that the outcome of all designs undertaken to the code is in some sense optimal. However, such a formal optimization process is not always carried out in practice ; even in cases where it has been undertaken, the values of the partial factors adopted may be modified in the interests of simplicity and ease of use. In general terms, the aim of the optimization is to minimise the deviations of failure probabilities of a range of similar components from a target probability of failure, whilst maintaining the average failure probability at the target level. This procedure, described in[8], has been used in the calibration of partial factors in the UK bridge code[9].

In addition to the formal probabilistic optimisation approach, calibration of partial factors to reflect a long and successful history of building tradition is always carried out. In the present set of Eurocodes, the two methods have been used in combination, with the probabilistic method giving added value to the traditional approach.

Returning to eq. (3), ISO 2394[7] suggests the following partial factor safety checking format

$$\gamma_n \gamma_{Sd} s(F_d,\ldots) \leq \frac{1}{\gamma_{Rd}} r(f_d,\ldots) \quad \cdots\cdots (5)$$

where F_d, f_d are design values, which can be obtained from characteristic values and associated partial factors (see eq.4 a and 4 b), and γ_{Sd}, γ_{Rd}, γ_n are partial factors related to modelling uncertainties (loading/resistance) and failure consequences.

3.2 Methods for probabilistic verification

Different methods can be used for probabilistic verification but in most code related work that has taken place in the last fifteen years or so, the so-called Advanced Level 2 method has been employed[10]. There are several reasons for this, which are covered in detail in standard texts[8),11]. Apart from computational efficiency and the ability to handle complex and/or implicit limit state functions, this method can accommodate both second moment description of random variables (i.e. making use of mean and variance only) and full distribution information and it can equally be applied to time-independent or time-dependent limit state functions. Furthermore, with the experience acquired over many years, initial problems that were encountered due to the discrete nature of safety checking (i.e. due to the evaluation of P_f on the basis of information relevant to certain points on the limit state surface) have been largely overcome. Perhaps more importantly, the results from this method can be readily used to evaluate and to optimize partial factors in codes.

The results obtained from this method are the reliability index β(sometimes called the Hasofer-Lind reliability index to distinguish it from similar, but less general, definitions that exist in the literature) and the sensitivity factors α_i, associated with any random variable ($i=1, \ldots n$). These parameters have a special geometric meaning in standard normal space, i.e. the space where all the basic random variables have been transformed to standard normal variables[8),11], However, from an application point of view, the important relation is the following

$$P_f=\Phi(-\beta) \quad \cdots\cdots (6)$$

where Φ is the standard normal distribution function. This particular result is obtained from a linearisation of the limit state surface at the so-called "design point" and is, hence, a first-order approximation.

Under certain conditions, the design point is the most likely failure point ; in other words, the coordinates of this point give the combination of basic random variables that are most likely to occur at failure. As might be expected, this normally implies values of resistance variables below their mean value, whilst the opposite is true for loading variables. Since the objective of a Level 1 code is to ascertain attainment of a limit state, it is clear that any check should be performed at a critical combination of loading and resistance variables and, in this respect, the design point values are a good choice. Hence, the term "design values" mentioned above.

More elaborate calculations are needed if a quadratic (or second-order) approximation of the limit state surface is used. In recent years, the terminology FORM and SORM has been widely accepted for first-order and second-order reliability methods. It is important to note that these methods differ only in the approximation used for estimating P_f and both have their basis in the Advanced Level 2 method. FORM, in particular, should not be confused with some early methods[12] which are valid for linear limit state functions but can produce erroneous results for more general cases.

The sensitivity factors α_i are quantities of considerable use, as they provide an indication of the relative importance of basic random variables on the reliability. Their absolute value ranges between 0 and 1 and the closer this is to the upper limit, the more significant the influence of the respective random variable is to the reliability. It is also worth noting that, unless there is stochastic dependence between the random variables, the following expression is valid

$$\sum_{i=1}^{n}\alpha_i^2=1 \quad\cdots\cdots (7)$$

Both β and α_i can be directly related to the value of the partial factor assigned to the characteristic or representative value of any variable in a Level 1 code. It can be shown that the partial factor associated with a basic random variable Z_i, is given by the following

$$\gamma_{Zi}=\frac{z_{di}}{z_{ki}} \quad\cdots\cdots (8\text{ a})$$

where z_{di} is the design point value of Z_i in any particular limit state and z_{ki} is the characteristic value of Z_i.

If Z_i is normally distributed with a coefficient of variation (i.e. the standard deviation divided by the mean value) v_{Zi}, and by non-dimensionalising both z_{di} and z_{ki} with respect to the mean value, eq. (8 a) can be written as

$$\gamma_{Zi}=\frac{1-\alpha_{Zi}\beta_t v_{Zi}}{1+kv_{Zi}} \quad\cdots\cdots (8\text{ b})$$

where β_t is the target reliability and α_{Zi} is the sensitivity factor obtained from a FORM/SORM analysis for any particular limit state, and k is a constant related to the fractile of the distribution selected to represent the characteristic value of the random variable Z_i. As shown, eq. (8 a) and (8 b) are used for determining partial factors for loading variables, whereas their inverse is used for determining partial factors for resistance variables.

For example, a typical 95% characteristic value (i.e. with a probability of being exceeded equal to 0.95) for a normally distributed material property variable would give $k=-1.645$. In general, characteristic values for material properties, and other resistance variables, are chosen so that a large fraction would, in reality, exceed this value. Assuming, for illustration purposes, a coefficient of variation $v_{Zi}=0.15$, a target reliability index $\beta_t=3.8$ and a sensitivity factor $\alpha_{Zi}=0.5$, one would obtain $\gamma_{Zi}=1.05$ from the inverse of eq. (8 b). This is the partial safety factor with which the characteristic value is divided to arrive at the design value used in eq. (3) or (5), i.e.

$$z_{di}=z_{ki}/1.05$$

For completeness, assuming that Z_i were a loading variable with the same sensitivity, then, from FORM analysis, one would obtain $\alpha_{Zi}=-0.5$ (notice difference in sign) and the characteristic value would, in general, be specified so that there is only a small probability of being exceeded (say 10%), hence $k=1.28$. Thus, from eq. (8 b), $\gamma_{Zi}=1.08$ and

$$z_{di}=1.08z_{ki}$$

Expressions similar to eq. (8 b) can be derived for variables described by distributions other than normal (e.g. log-normal, Gumbel type I). It is clear from the above how partial factor values could be obtained provided that FORM/SORM analysis is carried out for a whole range of limit states and structural elements in order to estimate the sensitivity factors. However, it is worth noting that a simplified method for determining partial factors exists, provided the designer has some prior knowledge of the relative importance of the variables on the reliability. For limit states that can be expressed in the form of eq. (3), it is suggested[7] that the sensitivity factors are written as

$$\alpha_{ir}=\alpha_R\alpha_{R,i} \quad\cdots\cdots (9)$$

where α_{ir} is the sensitivity factor associated with the ith resistance variable (i.e. it has a rank i in terms of its sensitivity factor), α_R is the overall sensitivity factor for a combined variable modelling the resistance and $\alpha_{R,i}$ is a weighting factor which can be determined as follows

$$\alpha_{R,i}=\sqrt{i}-\sqrt{i-1} \quad\cdots\cdots (10)$$

Identical expressions can be written for the sensitivity factors of loading variables. For a wide range of structural members and limit states, overall sensitivity factors $\alpha_R=0.8$ and $\alpha_S=-0.7$

have been found satisfactory. Hence, if proper ranking of variables can be achieved, e.g. by experience and by selective FORM analyses, the required sensitivity factors may be estimated approximately. By comparing with eq. (7), it is clear that this procedure tends to overestimate sensitivities and, hence, it would lead to conservative partial factor values, provided that the correct ranking has been assumed. Implementation of this approach does not require repeated FORM analyses for a range of structural members and limit states but is based on approximations that have been validated through years of experience in a variety of applications.

3.3 Load combination principles

In accordance with ISO 2394[7], actions, which can be direct, such as forces (loads) applied to the structure, or indirect, such as imposed deformations, are primarily classified with regard to

- their variation in time
- their spatial variation
- the nature of the induced structural response.

In order to account for time variations, individual actions, which are essentially random processes, $p(t)$, are modelled by the distribution of the maximum value within a given reference period T. i.e. $Z=\max_T\{p(t)\}$ rather than the point-in-time distribution. For continuous processes, the probability distribution of the maximum value (i.e. the largest extreme) is likely to be very closely approximated by one of the asymptotic extreme value distributions. In this way, for structures subjected to a single time varying action, a random process model is replaced by a random variable model and the principles given in the previous section apply, in terms of defining characteristic and design values.

The theory of stochastic load combination is used in situations where a structure is subjected to two or more time varying actions acting simultaneously. It is worth noting that due to time and, possibly, space related stochastic dependence, modelling of actions in such cases might require a more detailed characterisation of the stochastic process than that mentioned in the preceding paragraph.

From the designer's point of view, perhaps the most important factor that needs to be addressed in a code is that, in such cases, it is unlikely that each load will reach its peak lifetime value at the same moment in time. Thus, considering two time varying load processes $p_1(t)$, $p_2(t)$, $0\leq t\leq T$, acting simultaneously, for which their combined effect may be expressed as linear a combination $p_1(t)+p_2(t)$, the random variable of interest is

$$Z=\max_T\{p_1(t)+p_2(t)\} \qquad \text{(11 a)}$$

If the loads are independent, replacing Z by $\max_T\{p_1(t)\}+\max_T\{p_2(t)\}$ leads, in many cases, to very conservative designs. On the other hand, the distribution of Z can be derived in only few cases. One possible way of dealing with this problem, which also leads to a relatively simple code format, is to replace Z with the following

$$Z'=\max_T\begin{cases}\max_T\{p_1(t)\}+p_2(t)\\ p_1(t)+\max_T\{p_2(t)\}\end{cases} \qquad \text{(11 b)}$$

This is the so-called Turkstra's rule which, in effect, suggests that the maximum value of the sum of two independent load processes occurs when one of the processes attains its maximum value. The conditions which render this approximation adequate are explained in[11].

A code format for design values, compatible with the above rationale, may be written as

$$\max\begin{cases}\gamma_1F_{k1}+\gamma_2\psi_{02}F_{k2}\\ \gamma_1\psi_{01}F_{k1}+\gamma_2F_{k2}\end{cases} \qquad \text{(11 c)}$$

where, F_{ki} and γ_i are characteristic values and partial factors discussed in the preceding section and ψ_{0i} are the combination factors ($i=1, 2$). The above can readily be generalised for more than two loads acting in combination.

As stated in ISO 2394[7], two types of combinations should be considered, the first involving permanent and variable actions only (called fundamental combinations), while the second comprises of permanent, variable and accidental actions (called accidental combinations). Different combination factors should be derived to cover the various cases. The need for a number of design checks is thus apparent although, in practice, it is often possible to reduce this number through experience and/or knowledge of a structure's characteristics.

In principle, the ψ_0 factors express ratios between fractiles in the extreme value and point-in-time distributions so that the probability of exceeding the design value arising from a combination of loads is of the same order as the probability of exceeding the design value caused by one load. For time varying loads, they would depend on distribution parameters, target reliability and FORM sensitivity factors and on the frequency characteristics (i.e. the base period assumed for stationary events) of loads considered within any particular combination.

Finally, it is worth noting that a factor similar to a ψ_0 factor may be applied even in a case where only two loads are applied on a structure (i.e. one permanent, G, and one variable, Q, load) to account for the fact that only one of the two loads can be dominant with respect to the reliability sensitivities. Assuming that FORM analysis indicates that the dominant load in terms of sensitivity is Q, the design value of G may be multiplied by the following factor

$$\psi_{0G}=\frac{1-(-0.7)(0.4)\beta_t v_G}{1-(-0.7)\beta_t v_G} \quad \cdots\cdots (12)$$

where v_G is the coefficient of variation of G and it is assumed that G is normally distributed. The factor 0.4 in the numerator arises from application of equation (10), since Q is the dominant and G is the second most dominant loading variable. However, as stated previously, the ψ_0 factors are primarily intended for combinations of several time varying loads and, in this case, different expressions are applicable.

3.4 EC 3 code format and partial factor evaluation

In accordance with the various principles and methods outlined in the preceding sections, EC 3 adopts the following safety checking format for ultimate limit states

$$\sum_j \gamma_{Gj} G_{kj} + \gamma_{Q1} Q_{k1} \text{“}+\text{”} \sum_{i>1} \gamma_{Qi} \psi_{0i} Q_{ki} \leq R(f_k, ...)/\gamma_M \quad \cdots\cdots (13)$$

where G_{kj} and Q_{ki} are characteristic values of permanent and variable loads, γ_{Gj} and γ_{Qi} are the associated partial factors, Ψ_{0i} are combination factors, f_k is the characteristic material strength value used within a resistance function $R(.)$ and γ_M is the associated partial factor of the resistance. The symbol “+” implies “to be combined with” and is used in order to emphasize that a combined loading effect might not be always definable as a sum of individual loads. Broadly similar formats are adopted for serviceability and accidental limit states but, of course, the partial factor and combination factor values are different.

In comparison with eq. (5), it is clear that in the EC 3 format the number of partial factors has been reduced for ease of application. Nevertheless, it is sufficiently transparent for all the important principles mentioned above to be incorporated. As explained in background documents and other publications[13)~15)], the loading uncertainty partial factor γ_{Sd} has been set equal to 1.05 and incorporated in the evaluation of the load factors, γ_{Gj} and γ_{Qi}. Furthermore, instead of using partial factors on individual resistance variables (of which, probably the most important is the material strength relevant to any particular limit state), it has been decided to use nominal or specified characteristic values for resistance variables together with a single resistance partial factor, γ_M. Thus, the right hand side of eq. (13) represents the design value of the resistance R_d (which is equal to R_k/γ_M, where R_k is the specified characteristic resistance value). The procedure for the evaluation of R_k and γ_M or, equivalently, of R_d is summarized below but more details can be found in[13)~15)].

The resistance function can be expressed in general terms as

$$R=bR_t\delta \quad \cdots\cdots (14)$$

where, $R_t=g_R(X)$ is a derived random variable and $g_R(X)$ is the design model, i.e. a function of the basic random variables X based on structural behaviour, b is a mean value correction factor and δ is a random error term with mean value equal to unity and a coefficient of variation v_δ. The function $g_R(X)$ can take different forms depending on the models used for calculating the resistance. In many cases, the design model function can be expressed as a product, i.e.

$$g_R(X)=X_1X_2X_3...X_N \quad \cdots\cdots (15\text{ a})$$

and by defining an auxiliary variable $R'=\ln R$, eq (14) can be transformed into

$$R'=\ln R=\ln b+\ln X_1+\ln X_2+\cdots+\ln X_N+\ln\delta \quad \cdots\cdots (15\text{ b})$$

If variables $X_j (j=1,...N)$ are described by log-normal distributions then variables $\ln X_j$ are normally distributed and the standard deviation of $\ln R$ is obtained from

$$\sigma^2_{\ln R}=\sigma^2_{\ln X_1}+\sigma^2_{\ln X_2}+...+\sigma^2_{\ln X_N}+\sigma^2_{\ln\delta} \quad \cdots\cdots (15\text{ c})$$

The standard deviation of basic variables is usually estimated by recourse to published data (so-called pre-knowledge), whereas the standard deviation of $\ln\delta$ is based on an assessment of the design model using test results.

As pointed out in background studies, comparison of different design models for particular limit states is undertaken with reference to mechanical behaviour and correlation to any available test results. After selecting a particular design model, a set of correction factors can be obtained from test results ($i=1$, ... N_t) by considering the ratio

$$b_i=r_{ei}/r_{ti}$$

where r_{ei} is the experimentally recorded value and r_{ti} is the design model prediction for test i. In calcuating r_{ti}, it is important to use measured values for the basic variables. These will, in general, be different from one test to another, even when considering nominally identical specimens. Thus, an estimate for the mean value of the correction factor is given by

$$\bar{b}=\frac{1}{N_t}\sum_{i=1}^{N_t} b_i \quad \cdots\cdots (16)$$

The error term is defined by the ratio

$$\delta_i=r_{ei}/br_{ti}$$

and by considering the auxiliary variable $\delta'=\ln\delta$, an estimate of the standard deviation of δ' can be obtained from

$$s_{\delta'}=\sqrt{\frac{1}{N_t-1}\sum_{i=1}^{N_t}(\delta'_i-\bar{\delta}')^2} \cdots\cdots\cdots\cdots\cdots\cdots\cdots (17)$$

Unless a large sample of randomly generated test data exists, which is not usually the case, the above estimate must be corrected for statistical uncertainty through a factor that depends on the number of test results and the required degree of confidence in the estimate. Furthermore, corrections need to be introduced if the values of some of the basic variables influencing the design model prediction have not been measured during tests and are, hence, defined by nominal or specified characteristic values or if the resistance function has a form different to that given by eq. (15 a). Through all these considerations, it has been possible to validate the strength prediction models to a considerable degree and, moreover, to account for the quality and quantity of existing test data.

Once the standard deviation of the resistance has been determined, both design and characteristic values and, hence, partial factors can be obtained through the use of expressions similar to eq. (8). As can be seen from eq. (8 b), in addition to the variability of the resistance, its associated FORM sensitivity factor, α_R, and the target reliability index, β_t, need to be defined.

Since only a single partial factor for the resistance is introduced in the EC3 code format, the associated sensitivity factor is taken as $\alpha_R=0.8$, following the procedure of ISO 2394[7], see eq. (9) and (10). Thus, effectively the target reliability β_t is achieved through a resistance requirement $\alpha_R\beta_t$ and an associated loading requirement $\alpha_S\beta_t$, where $\alpha_S=-0.7$. This has been found acceptable for a wide range of member checks, provided that the ratio of standard deviations of the two composite random variables representing loading and resistance lies within a wide range, which covers normal design[13].

The target level of reliability will, in general, depend on the cause and mode of failure, the possible failure consequences and socio-economic factors. **Table 1** gives indicative values for the target reliability index proposed in EC 1[6] assuming a reference period of 50 years. These numbers represent notional values, intended primarily as a tool for developing consistent design rules, and should not be interpreted as failure frequencies. In exceptional cases, where over a range of design parameters it is not possible to maintain the target value of $\beta_t=3.8$, a local reduction of 0.5 has been allowed. System effects have not so far been considered in EC 3, although it has been proposed to account for these in an approximate way by varying β_t to reflect redundancy and ductility.

Table 1 Indicative values for the target reliability index[6]

Limit state	Target β (lifetime)	Target β (one year)
Ultimate	3.8	4.7
Fatigue	1.5~3.8*	–
Serviceability (irreversible)	1.5	3.0

**Depends on degree of inspectability, repairability and damage tolerance*

Thus, using the above semi-probabilistic procedure, together with target reliability and sensitivity values, specific γ_M values have been derived. In principle, each resistance function would produce a different γ_M value. However, based on the results obtained and in order to avoid a large set of partial factors, it was decided to group γ_M values into different classes, depending on whether the resistance is associated with yielding, buckling or fracture. The latter is normally a function of the ultimate tensile strength of the material (e.g. bolt and weld resistance, bearing resistance), whereas the former two phenomena are related to the yield strength of the material.

The evaluation of partial factors and combination factors for load variables, i.e. the left hand side of eq. (13), is undertaken using approximate sensitivity factors based on an overall $\alpha_S=-0.7$ and the relative importance factors determined from eq. (10). Load combinations are treated using the principles outlined above in section 3.3 but reduction factors similar to eq. (12) are not so far used for combinations of permanent and variable loads, in order to reduce the number of load cases for the designer. Simplifications in the treatment of load combinations are also allowed for both ultimate and serviceability limit states in building structures.

Finally, it is worth noting that in the ENV period, EC 3 is intended for use in association with National Application Documents (NAD), issued under the authority of the national governments of each member state. To facilitate use during this period, partial factor values are given in boxes, the so-called boxed values, whereas the NAD gives the actual national values, selected to reflect national loading codes and local requirements concerning practice and public safety.

From the presentation made above, it is perhaps clear that safety requirements in a code can be achieved in different ways, through the specification of characteristic and design values and partial and combination factors. For this reason, attempts to explain and quantify the safety aspects and the design philosophy of any particular code should deal with a variety of items and procedures. Otherwise, it is quite easy to misinterpret safety provisions and associated values of factors and variables. In the main part of EC 3, considerable effort has been made to retain transparency, which is further assured through the availability of background documents and Annexes. If this were not the case, not only would it not have been possible to produce the NADs and to use EC 3 in conjunction with national loading codes but it would have also restricted the applicability of the code. As will be seen below, the code encourages innovation by including a chapter on design assisted by testing, where the methods outlined above for determining resistance partial factors and design values are also employed.

4. METHODS OF ANALYSIS AND DESIGN ASSUMPTIONS

Elastic or plastic global analysis may be used to calculate internal forces and moments in statically indeterminate structures. Plastic analyses range from the commonly adopted rigid-plastic method to advanced computer-based elastic-plastic methods. Within elastic-plastic analysis, two forms are distinguished. In the elastic-perfectly plastic method, the cross-section remains fully elastic until the plastic resistance moment is reached and then becomes fully-plastic, whereas in the elasto-plastic method the spread of plasticity through the depth and along the length of a member is followed incrementally. The former is used in most of the currently available computer software for plastic design, while the latter is employed in some more advanced software.

There is also a distinction between simple first-order theory, which uses the initial geometry of the structure, and second-order theory, which takes into account the influence of deformations due to load on stability. 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.

In common with other building codes, EC 3 distinguishes between simple, continuous and semi-continuous construction. However, a special feature is that plastic analysis of semi-continuous frames may be used. It involves the consideration of partial strength joints which develop plastic hinges with a smaller plastic 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.

Table 2 Use of simple rigid-plastic analysis in EC 3[17)]

No. of storeys	Braced	Unbraced	
		Non-sway	Sway
1, 2	Yes	Yes	Yes*
>2	Yes	Modified method	Modified method

**with amplified sway moments*

Although not required in the course of routine design, EC 3 includes a chapter on design assisted by testing (Chapter 8), which may be resorted to by innovative engineers introducing new systems. In addition to general principles and guidance on planning of tests, assessment of test results is covered. This includes a method for evaluating resistance design values and partial factors following the principles used by the code drafters in arriving at the design values given in the code.

5. 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 column and storeys. These can be represented by equivalent horizontal forces, which should be allowed for in all load combinations including those involving wind loads. In the case of bracing systems, allowance is made for imperfection, or initial bow, in the members to be restrained. The resulting forces from both frame and bracing imperfections are used for member design.

Normally, the effects of imperfections on member design are accounted for within the buckling strength formulae given in the code. The exception is highly stressed and/or slender axially compressed members in sway frames with moment-resisting connections.

The code provides clear definition of sway and non-sway frames in terms of a simple application rule involving the elastic critical load ratio (design value of total vertical load divided by the elastic critical sway mode value). For regular plane frames, the distinction can be based on the results of first-order theory, i.e. if

$$\frac{\delta}{h} \leq 0.1\frac{H}{V} \qquad (18)$$

then the frame is classified as non-sway. In eq. (18) δ is the horizontal deformation of a storey, h is the storey height, H is the total horizontal reaction (including equivalent loads arising from frame imperfections) and V is the total vertical reaction at the bottom of the storey. This criterion allows the majority of regular plane frames in building structures to be designed with first-order theory[16].

Distinction is also made between braced and undraced 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 classified as non-sway frames using eq.(18). The bracing system should be designed to resist any horizontal loads applied to the frame, including the effects of initial sway imperfection, as well as loads applied directly to the bracing system.

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 analysis provided the simplified method given in the code is used and all accompanying conditions are met. Otherwise a second-order elastic-plastic sway analysis is required. **Table 2**[17] summarizes the various cases.

6. LIMIT STATE DESIGN OF MEMBERS AND CONNECTIONS

In line with most modern codes, EC3 examines both serviceability and ultimate limit states. The former comprise various deflection limits, including ponding and dynamic effects. For the latter, rules are assembled under four headings : cross-section resistance, member buckling resistance, shear buckling resistance and web crippling. Moreover, there is a separate chapter on connections (Chapter 6), which treats in depth a large number of bolted and welded connections due to their importance in economical design of steel structures. Finally, fatigue rules are also given (Chapter 9), which can be referred to from other future parts but, of course, can also be used for building design if fatigue is an issue.

Cross sections are divided into four classes as follows :

Class 1 cross-sections are those which can form a plastic hinge with the rotation capacity required for plastic analysis.

Class 2 cross-sections are those which can develop their plastic moment resistance but have limited rotation capacity.

Class 3 cross-sections are those in which the calculated stress in the extreme compression fibre of the steel member can reach its yield strength but local buckling is liable to prevent development of the plastic moment resistance.

Class 4 cross-sections are those in which it is necessary to make explicit allowances for the effects of local buckling when determining their moment resistance or compression resistance.

The limiting slenderness values used for this classification are obtained through simple expressions involving the non-dimensional parameter $\varepsilon=(235/f_y)^{1/2}$, where f_y is the yield strength. A comprehensive set of diagrams of cross-sections and possible stress distributions (bending, compression or combined loading) is provided. The slenderness limitations of the various elements of the cross section have been selected from available data. The concept of effective width is used to reduce section properties of class 4 cross-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 two values : the plastic resistance of the gross area based on the yield strength and the ultimate resistance of the net area using 90% of the ultimate strength. As indicated in section 3.4, different γ_M factors are used since the former depends on yield strength while the latter depends on ultimate strength. The effect of bolt holes on shear resistance is only taken into account if the ratio of net to gross area is less than the ratio of yield to ultimate strength. It is, however, necessary to check block shear strength at the ends of a member.

The plastic moment resistance is reduced to account for co-existent shear when the shear force exceeds 50% of the plastic shear resistance. Interaction expressions for axial force and bending moments are also given.

6.1 Buckling resistance of members

The basis for buckling resistance checks is provided by the European column buckling curves[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. In general terms, the design buckling resistance of compression members can be written as

$$N_{b.Rd}=\frac{\chi(\lambda,\alpha,\varepsilon,P_{cr},\beta_A)Af_y}{\gamma_M} \quad\cdots\cdots\cdots\cdots\cdots\cdots(19)$$

where, β_A is a parameter related to the class of the cross section, A is the cross-sectional area, f_y is the yield strength, γ_M is the appropriate partial factor for buckling resistance and $\chi(.)$ is the reduction factor obtained from tables or simple expressions. As might be expected, $\chi(.)$ is a function of a slenderness parameter λ calculated using the effective (buckling) length of the member (for which guidance is provided in Annex E), an imperfection factor α, a material parameter ε, the elastic critical load P_{cr} and the cross-section class factor β_A. Values of this function, which describes the four column curves through different values of α, can easily be calculated using simple expressions or directly obtained from tables.

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. The latter can either be omitted or taken advantage of, provided it can be achieved in practice.

Combined bending and axial load is treated through interaction diagrams, whereas the case of non-uniform bending is dealt with by defining equivalent uniform moment factors.

6.2 Shear buckling resistance

Unstiffened webs with depth to thickness slenderness ratios greater than 69 ε need to be checked for shear buckling. Two methods may be used, the first being a simple post-critical buckling approach, while the second is a tension field method. Either method may be used for webs with or without transverse stiffeners, although application of the tension field method is restricted to spacing to depth ratios greater than unity. Moreover, it is indicated that it becomes over-conservative if this ratio is greater than three. Design expressions for end panels, as well as stiffness and strength criteria for intermediate transverse stiffeners and end posts are given.

It is intended to produce comprehensive design rules for plate girders with more complex stiffening arrangements (e.g. with both transverse and longitudinal stiffeners) in Part 2 of EC 3 dealing with bridges and plated structures.

Design rules for flange buckling in the plane of the web, as well as guidance for members curved in elevation are included. The resistance of webs to in-plane transverse forces such as those that occur at supports and in some beam-column connections is treated by considering three modes of failure: web crushing, local buckling or crippling of the web and overall web buckling.

6.3 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.

6.4 Connections

Extensive design criteria for all relevant connection properties, i.e. strength, stiffness and deformation capacity, are presented. Despite the plethora of possible structural solutions, a common design approach has been adopted for all types of connections[18]. Guidance on the appropriate assumptions for determining a realistic distribution of forces within the connection is given in terms of satisfying equilibrium, deformation and load path characteristics. Classification of connections is undertaken in terms of rigidity (nominally pinned, rigid or semi-rigid) and strength (nominally pinned, full-strength or partial-strength). This is important in modern limit state design, which requires a more realistic and detailed treatment of connections, in order to properly take advantage of plastic analysis methods. For the determination of appropriate resistance functions and partial factors about 2 000 test results on bolted connections and about 500 test results on welded connections were used.

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. In addition to checking the tension resistance of bolts in tension, the code includes a check on punching shear resistance by the bolt head or nut. 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 the 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 with bolted end-plate connections are treated in Annex J, which includes detailed guidance on the design of semi-rigid and partial-strength connections. It also contains data on the calculation of prying forces.

Stark[18] gives two examples where the rules have been found to allow for more economical construction in comparison with other codes :

(i) By allowing more freedom in the selection of end distances and pitch in welded connections, which is accounted for in the determination of bearing resistance, it is possible to avoid gusset plates and, hence, produce compact design of joints.

(ii) By allowing the use of bolts with the threaded portion in the shear plane, it is possible to reduce the number of bolt types to be kept in stock and, hence, improve efficiency and reduce the potential for errors in construction.

7. USEABILITY AND SIMPLIFIED VERSIONS

The drafters of the code have been conscious of the need for the code to be user friendly from the outset of their work[19]. 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 facilties available, to designers in offices of small steel fabricators with very 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 in educational establishments and site engineers. Furthermore, as with every new code, there is a need for measures to assist the introduction and acceptance of the Eurocodes into everyday design practice. For these reasons, a hierarchy of supporting material has already been produced, and will be further expanded, both at national and international level. A hierarchical list of possible design aids linked to EC 3 is given in[19], whereas a brief summary of some of the already existing documents is given below.

The ECCS have published a shortened version of EC 3, together with additional tables and other supporting information entitled 'Essentials of Eurocode 3'(E-EC 3)[20]. It is intended as a design aid to facilitate the use of the code 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. In all cases of doubt, or for items not covered, EC 3 and the relevant NAD must be consulted, since E-EC 3 is not intended to be used independently of the code itself.

This document has now been complemented by the publication of a set of 'Examples to EC 3'[21] with the same scope as E-EC 3. Some forty eight examples are presented-with calculations, sketches, cross-reference to code clauses and commentary-arranged in three parts :
Part 1 covers load combinations, methods of analysis, frame analysis and bracing systems analysis.
Part 2 covers member design in compression, bending and combined compression and bending.
Part 3 covers connection design, i.e. bolted, welded and pinned.
The Steel Construction Institute in the UK has produced another document, 'The Concise EC 3' (C-EC 3)[22]. Although in some respects similar to E-EC 3, the objective is to present a shortened version of EC 3 to cover only those types of building structures that can currently be designed with 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-EC 3 is a self-contained, stand-alone design code. Its purpose is to introduce designers to the provisions of EC 3 by building on familiar ground. It can be used independently of EC 3, yet produce designs fully compatible with EC 3 for the range of strctures covered, although not necessarily taking advantage of some of the more refined approaches given in EC 3. A number of features have been introduced to facilitate its use by designers currently accustomed to British Standards.

8. 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, Japan and elsewhere. It is based on limit state philosophy and probabilistic safety concepts and has been produced in a format which should be sufficiently clear, transparent and comprehensible for practising engineers.

The design philosophy and assumptions, the limit state functions, the evaluation of available test results and the derivation of partial factors are fully described in background documents, which should prove particularly useful to future reviews of the code. In this paper, an attempt has been made to outline the fundamental principles and to highlight their application in EC 3. Moreover, a qualitative description of the code clauses has been presented and supporting documents and their scope have been identified.

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

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APPENDIX A

There are 9 Chapters and 9 Annexes in ENV 1993-1-1. Annexes are classified as either normative of informative. Normative Annexes have the same status as the main Chapters. Informative Annexes provide additional information. The Chapter titles are listed 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

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*

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