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RELATIONSHIP BETWEEN EUROCODE 1 AND THE « MATERIAL » ORIENTED EUROCODES

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SUMMARY

After having recalled the basic philosophy of the safety checking format of ENV 1991-1 the paper presents how the particular « material » Eurocodes are consistent with the principles adopted in Eurocode 1 - Part 1 : « Basis of design ». This paper emphasizes 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 affected to the limit state functions. The evaluation of the partial safety factors depends upon some assumptions concerning the choice of the reliability level prescribed in Eurocode 1 which are underlined in this paper.

1. INTRODUCTION

EUROCODE 1 Part1 (ENV 1991-1) « Basis of design » is the reference design code which describes the principles and requirements for safety, serviceability and durability of structures. As such the ENV 1991-1 as to be regarded as the basic reference document upon which the other EUROCODES (the so-called « material oriented » or « design » Eurocodes (ENVs 1992 to 1999) are consistent with. The principles and safety format of ENV 1991-1 are in line with the ISO 2394.

The fundamental requirements imply by Eurocode 1, are such that structures or structural elements are designed, with an appropriate degrees of reliability, as to:

- sustain actions liable to occur during construction and use,
- perform adequately during their intended life,
- maintain sufficient structural integrity during and after accidental loads (fires, explosions, earthquake,...,
- have adequate durability.

ENV 1991-1 sets out a common basis for defining design rules for buildings and civil engineering works by reference to a set of limit states beyond which the considered structure no longer satisfies the fundamental requirements. The limit states are classified into two main categories :

- the ultimate limit states, which are those corresponding to the maximun load carrying resistance of the structure which results, if reached, in structural failure or in complete unserviceability,

- the serviceability limit states, which are those related to criteria governing the normal use of the structure.

The safety checking format of ENV 1991-1 is a design method (partial factors of safety design) in which appropriate degrees of safety are provided by the definition of characteristic values (or representative values) and a number of partial factors.

The variability of the actions on a structure is taken into consideration by defining them in terms of design values derived from characteristics (F_k) or representative values which are factored by a partial factor (γ_f) as :

$$F_d = F_k \gamma_f$$

The variability of the strengths and other properties of the construction materials is treated in the same way by defining characteristic strengths (determined on a statistical interpretation of data) or on representative values of strengths (on appraisal of experience). Here also the characteristic (or representative value) of the strength is factored by a partial factor to give the design value of the resistance as :

$$R_d = \frac{R_k}{\gamma_R}$$

The characteristic value is defined in terms of a prescribed probability of not being exceeded for loads, or attained for resistances.

Thus the verification of a structure with regard to a particular limit state is expressed as :

$$E_d = E(F_{d1}, F_{d2}, \ldots) \le R_d$$

Where E_d is the effect of actions such as internal forces, moments or more generally stresses, strains or displacements. This effect of actions gives the response of the structure to a given set of loads (or actions).

In the determination of loads response, the proper method of structural analysis (elastic or elastic-plastic analysis with or without second order geometrical effect including consideration for partial strength and rigidity of joints) is prescribed in each « design » code according to criteria which assess explicitly the validity of the relevant method of analysis to be used.

In the previous equation, the design values are defined to achieve the required reliability expressed in terms of the so-called reliability index β, which is related, under some assumptions, to the failure probability by :

$$P_f = \Phi(-\beta)$$

where $\Phi(*)$ is the distribution function of the Gaussian probability density function.

Indicative target values of β is given in table 1.1 for the design working life and for one year and are reproduced from Annex A of ENV 1991-1. The choice of the various target values takes into account the possible consequences of failure in terms of risk to human life or injury, economic losses and degree of social inconvenience resulting from failure.

Table 1.1 - Indicative values for target reliability index β

Limit state	Target reliability index (design working life)	Target reliability index (one year)
Ultimate	3,8	4,7
Fatigue	1,5 to 3,8 1)	-
Serviceability (irreversible)	1,5	3,0
1) Depends on degree of inspectability, repairability and damage tolerance.		

In order to make the definition of the resistance design value R_d, for limit state verifications, independant on the variation of the action effects and to achieve a basis common for all « material » oriented Eurocodes, R_d was defined such that the probability of having a more unfavourable value is given by :

$$P(R < R_d) = \Phi(-\alpha_R \beta) = \Phi(-0{,}8\beta)$$

Where α_R is the associated sensitivity factor (or the First Order Reliability Method weight factor). The value of $\alpha_R = 0{,}8$ (and $\alpha_E = 0{,}7$, see Annex A of ENV 1991-1) was found acceptable for a wide range of variability for resistance (and loading).

In the following it will be seen how the « material » Eurocodes 2,3, 4 and 5 relate to the modern principles which are adopted in Eurocode 1.

2. EUROCODE 2 CODE FORMAT AND RELATED PARTIAL FACTOR

2.1 Introduction

EUROCODE 2 as part of the European Regulation System deals with design and construction of buildings and engineering works in plain, reinforced and prestressed concrete. It is concerned with the essential requirements for resistance, serviceability and durability of concrete structures. Execution is covered to the extent that is necessary to indicate the quality of the construction materials and products which should be used and the standard of workmanship on site needed to comply with the assumptions of the design values.

Based on a long tradition the national codes in all European countries are characterized by an various level of design rules and practical experiences. This lead to results which - even when based on the same physical model - differ more or less significantly (fig. 2.1).

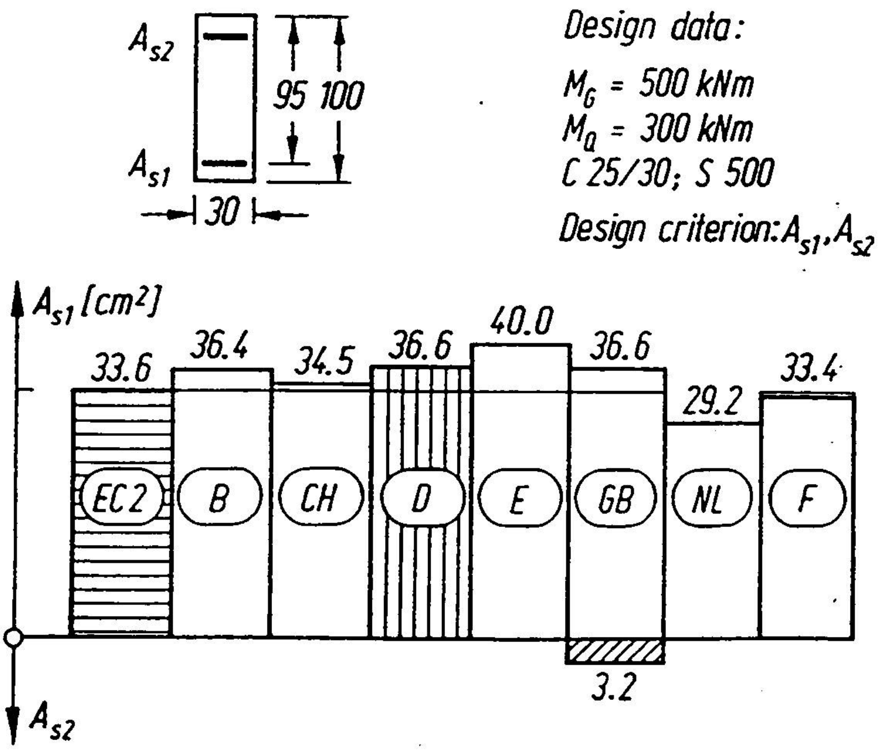

Figure 2.1 - Design of rectangular cross-section according to national regulations of some European countries (characterized by their registration plate)

The main objective of the work on Eurocode 2 therefore was not the total unification of the design rules, but a gradual approximation by publication of EC 2 in form of an European Prestandard (ENV), distinction of clauses in **Principles** and **Application Rules** and using indicative ("boxed") values.

During the ENV period Eurocode 1 "Basis of Design" was developed further due to the first experiences in partial application of all ENV's. In relation to EC 2 these developments concern the more precise consideration of limit state equations and the numerical revision of the reliability format for design, especially the partial safety factors for materials.

2.2 Limit state equations

2.2.1 Ultimate limit states

In EC 2 the ultimate limit states include the

- loss of equilibrium of the structures or any part of it modelled by a rigid body (EC 2, 2.3.1)
- ultimate limit states for bending and longitudinal force (EC 2, 4.3.1)
- ultimate limit states for shear (EC 2, 4.3.2)
- ultimate limit states for torsion (EC 2, 4.3.3)
- ultimate limit states for punching (EC 2, 4.3.4

as well as
- ultimate limit states induced by structural (buckling, lateral buckling of slender beams, EC 2, 4.23.5.

In design situations where dynamic actions are likely to occure, the ultimate state of fatigue needs to be considered. A corresponding design concept is provided in Part 2 of Eurocode 2 for concrete bridges.

In all these cases it shall be verified that

$$S_d \leq R_d, \tag{2.1}$$

S_d is the design value of an internal force or moment (or of a respective vector of several internal forces or moments) and
R_d is the corresponding design resistance, associating all structural properties with the respective design values.

Therefore, the partial safety for the persistent and transient situations are given in Table 2.1

Table 2.1 - Partial safety factors for actions in building structures for persistent and transient design situations

	Permanent actions	Variable actions	Prestressing
Favourable effect	1.0 [1)]	-- [2)]	0.9 or 1.0
Unfavourable effect	1.35 [1)]	1.5	1.2 or 1.0

1) In this verification the characteristic value of the permanent action is multiplied by the factor 1.1 and the favourable part by the factor 0.9.
2) See Eurocode 1: in normal cases for building structures $\gamma_{Q,info} = 0$.

2.2.2 Serviceability limit states

For concrete structures the serviceability limit states include

- a limitation of stresses under serviceability conditions (EC 2, 4.4.1)
- the serviceability limit states cracking (EC 2, 4.4.2)
- the serviceability limit states deformation (EC 2, 4.4.3).

In these cases it shall be verified that

$$E_d \leq C_d \tag{2.2}$$

where:

C_d is a nominal value or a function of certain design properties of materials related to the design effects of the actions considered, and

E_d denotes the design value of the actions effect (e.g. stresses in steel or concrete, crack width, displacement or acceleration), determined on the basis of one of the combinations.

Therefore, the partial safety factors γ_G and γ_Q are taken as 1.0 except where specified otherwise.

2.3 Partial safety factor for materials

The design resistance R_d in expression (2.1) has normally the following form:

$$R_d = R_d\left[\frac{x_k}{\gamma_M}, a_{nom}\right] \tag{2.3}$$

where:
X_k Characteristic value of the relevant material property, normally strength
γ_M Partial safety factor for materials, see Table 2.2
a_{nom} Nominal value of geometrical data

Table 2.2 - Partial safety factors γ_M for materials in Eurocode 2; all values are indicative

Material/ Combination	Concrete γ_c	Reinforced and prestressed steel γ_S
Fundamental	1.5	1.15
Accidental	1.3	1.00

The evaluation of these partial factors γ_M in Table 2.2 is based on the following derivation:

$$\gamma_M = X_k/X_d \tag{2.4}$$

The characteristic value X_k in Equ. (2.4) is defined as that value of strength below which 5 % of population of all possible strength measurements are expected to fall. For the 5 %-fractile this means for X_k:

$$X_k = \mu_x \exp(-k\, V_x) \tag{2.5}$$

where:
μ_x mean value
V_x coefficient of variation and
k = 1.645, if 5 % of all possible strength values are below the characteristic value.

The design value X_d in Equ. (2.4) is defined as:

$$X_d = \mu_x \exp(-\,\alpha_R\ \beta\, V_x) \tag{2.6}$$

where:
α_R = 0,8 the FORM weight factor
β = 3,8 the target value for the reliability index

With α_R $\beta = 0,8 * 3,8 = 3,04$, $V_x = V_R$ and $k = 1,645$ as well as $V_x = V_f$ the partial safety factor Υ_M in Equ. (2.4) is:

$$\Upsilon_M = \exp(3,04\ V_R - 1,64\ V_f) \tag{2.7}$$

The coefficient of variation for the design value V_R is defined as:

$$V_R = \sqrt{V_m{}^2 + V_G{}^2 + V_f{}^2} \tag{2.8}$$

where:

V_m coefficient of variation for model uncertainty
V_G coefficient of variation for geometry of element
V_f coefficient of variation for property

Table 2.3 contains the values of the various coefficients which were used for concrete and steel.

Table 2.3 Coefficients of variation for concrete and steel

Material	V_m	V_G	V_f	V_R
Concrete	0.05	0.05	0.15	0.166
Reinforcing Steel	0.05	0.05	0.05	0.087
Structural Steel	0.03	0.03	0.03	0.052

By adopting the values for V_R and V_f in Table 2.3 the partial safety factors are (see (2.7)):

* for concrete: $\Upsilon_M = \exp(3.04 * 0.166 - 1.645 * 0.15) \approx 1.3$
* for reinforcing steel: $\Upsilon_M = \exp(3.04 * 0.087 - 1.645 * 0.05) \approx 1.2$
* for structural steel: $\Upsilon_M = \exp(3.04 * 0.087 - 1.645 * 0.05) \approx 1.12$

EC2 takes into account the uncertainty that compressive strength of concrete is controlled using test specimens not taken from the structure. Therefore, the conversion factor Υ_{conv} with $\Upsilon_{conv} = 1.15$ was introduced corresponding with the design value of conversion factor η_d in EC1.

That means for the partial safety factor of concrete Υ_c:

$$\Upsilon_c = \Upsilon_M * \Upsilon_{con} = 1.3 * 1.15 \approx 1.5 \text{ (see Table 2.2)}$$

For industrial production and well-established quality control EC2 makes a compromise between the two possible values or the partial factors for steel, e.g. $\Upsilon_S = 1.15$ (see Table 2.2).

2.4 Conclusion

This abstract is based on the following background documents:

- Background Documentation; Part 1 of EC1: Basis of Design, First Draft 01.95
- Background Document; ENV 1992-1-2: Structural Fire Design of Concrete Structures, Draft 06.95.

3. EUROCODE 3 CODE FORMAT AND PARTIAL SAFETY FACTOR EVALUATION

3.1 Introduction

Eurocode 3 claims to be based on the best scientific and professional information available today, as almost all resistance rules were calibrated against available test results.

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

As national design codes of the European Union and EFTA member countries reflect various level of experiences or knowledges and various design practices, it was difficult to reach a final consensus on the best safe and economical design formulae to be adopted for Eurocode 3.

It was clear, from the Eurocode 3 first revision period, that the conflicting ideas on particular design requirements or strength design model could only be solved by background appraisal studies and calibration against tests.

At last, few knowledge and experience existed in various newly developed fields such as high strength steel material elaborated by the steel industry (such S460 ML) or new detailing methods as, for example, welded lattice hollow section connections or semi-rigid bolted beam-to-column connections. The applicability of conventional design rules or newly developed design rules needed to be proved and partial safety factors had to be adequately determined to achieve a coherent and consistent safety level through the entire Eurocode 3 design code.

These were the main reasons which led the various working groups in charge of preparing Eurocode 3 to undertake detailed test calibration studies.

The next paragraphs review the main principles and statistical approach which were based upon to support basic provisions and reliability level to calibrate design formula of Eurocode 3 and to reduce the number of optional (boxed) values for partial safety factors to a minimum.

3.2 Strength functions

The design resistances R_d, as pointed out earlier in paragraph 1, are defined by relation (1.2). Characteristic values of R_k, and partial safety factors γ_R are determined by comparisons of the results given by a sound mechanical model of the strength functions (R_{cal}) with the results obtained from experimental tests (R_{exp}). Then a statistical evaluation of these comparisons is carried out to determine the design values R_d and the associated values of γ_R complying with the target reliability index (mainly $\beta = 3{,}80$). The γ_R takes account of the deviations of the material properties and the deviation from geometrical properties from the characteristic values.

This procedure for the evaluation of R_k and γ_R or, equivalently, R_d is detailed in two main annexes : Annex Y : Design assisted by testing and more specifically in Annex Z : Determination of design resistance from tests of ENV 1993-1-1.

3.3. Main steps of the calibration procedure

Lets have a strength function denoted by $g_R(X)$, where X are the basic random variables (e.g. geometrical, material properties,...) assumed to follow a lognormal probability density function. The strength function must be representative of a sound mechanical (or physical) interpretative model of the mode of failure observed during the experiment. The calibration procedure in Annex Z proceeds from the following main steps :

- STEP 1 : Evaluation of the statistical characteristics of the basic variables :

All available experimental tests are collected from existing literature. In this step, the determination of mean values and standard deviation (or characteristic values) of basic random variables (X) may be obtained directly from the statistical analysis of the test data if well documented. If not, representative values may be assumed from existing foreknowledge or from an estimate of the coefficient of variation of the variables from other sources.

- STEP 2 : Plot experimental value against calculated values of the strength function (fig. 3.1) :

The values of the strength function (R_{cal}), calculated with the measured parameters (X), are compared with the tests results (R_{exp}).

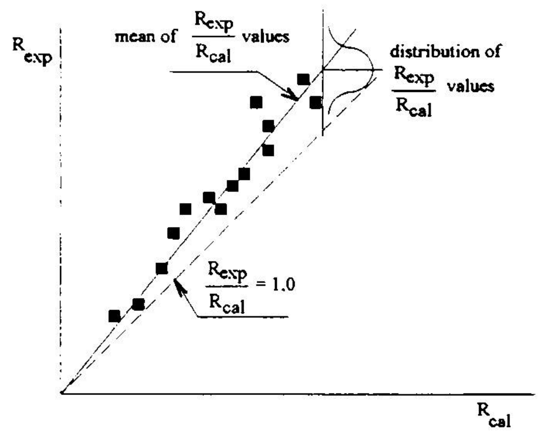

Figure 3.1 - Comparison of the theoretical strengths R_{cal} with test results R_{exp}

- STEP 3 : Check the correlation between experimental and calculated values :

From the statistical evaluation, the strength function will be corrected, if necessary, by an additional factor $\bar{b}$ (mean value corrective factor). 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.

- STEP 4 : Sensitivity analysis :

The sensitivity of the strength function R_{cal} is checked against the various parameters X by plotting the ratio R_{exp}/R_{cal} versus a particular variable X (fig. 3.2). In case the ratio R_{exp}/R_{cal} is too much scattered or non uniformly distributed along the range of the variable X, either the strength function should be improved or the test poulation should be subdivided into subsets i for which the deviation of $R_{exp}/R_{cal}(X_i)$ is merely uniform.

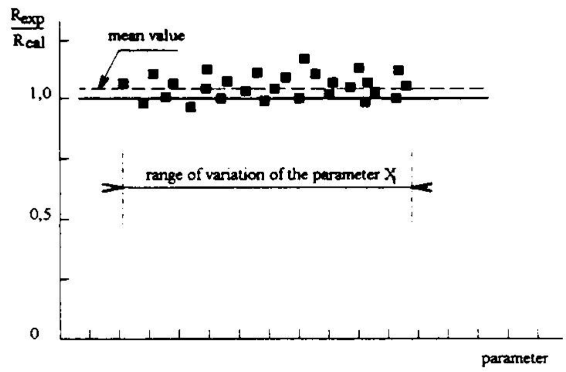

Figure 3.2 - Sensitivity of the strength function with respect to parameter X_i

- STEP 5 : Non Gaussian distribution :

The distribution of R_{exp}/R_{cal} for the test population considered is checked by plotting the values on a Gaussian paper (fig. 3.3) and the lower tail distribution is determined in case the distribtion is not Gaussian.

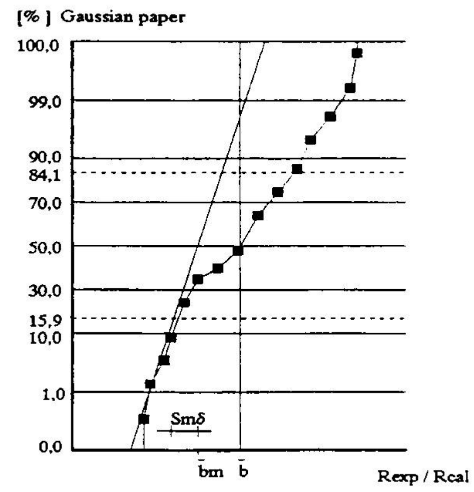

Figure 3.3 - Determination of the lower tail distribution from plotting the test population on a Gaussian paper

- STEP 6 : Determination of the characteristic strength function and the related partial safety factor :

The characteristic strength function is evaluated from the basic statistical information on all variables as established in step 1. 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 through the definition of a fractile factor k_s. This last, is determined according to the relevant number of test results. The fractile factor is usually established for a 5% fractile and a level of confidence of 75%.

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

$$\gamma_R = \frac{R_k}{R_d}$$

- STEP 7 : Comparison of the γ_R value obtain from step 6 and the classified γ_M values specified in Eurocode 3 :

In line with ENV 1991-1 the Eurocode 3 has introduced a set of differentiated partial resistance factors γ_M to limit the number of partial safety factor to take into consideration in the verification by the partial factor method. The relevant classes of γ_M factors is given in table 3.1.

Table 3.1 - List of fixed γ_M factors in Eurocode 3 - Part 1

$\gamma_{M0} = 1{,}00$	for all limit states associated with yield strength without stability phenomena.
$\gamma_{M1} = 1{,}10$	for all limit states associated with the yield strength and with stability phenomena.
$\gamma_{M2} = 1{,}25$	for all limit states associated with the tensile strength f_u and rupture of the material (e.g. net section failure, failure of connectors or welds,...).

In case where γ_R is not identical with γ_M for the relevant class, R_k is modify according to the following relation :

$$R_{k,mod} = R_k \frac{\gamma_M}{\gamma_R}$$

3.4. Conclusions

In Eurocode 3, a great number of calibration studies have been performed according to the basic principles specified in Eurocode 1 - Part 1 to achieve a coherent and uniform safety level through the entire design code. An exert from the list of backgroung documents with these calibration studies is given in table 3.2

Table 3.2 - Exert from the list of background documents for Eurocode 3 - Part 1

Background doc.	**List of limit state functions calibrated**
3.01	- Design against brittle fracture
5.01	- Justification of $\gamma_{M0} = 1{,}00$
5.02	- b/t ratios for classification of cross sections
5.03 to 5.04	- Columns, beams and beam-columns with cross section of class 1 to class 4
5.05	- Shear buckling
5.06	- Web crippling
5.07	- Hollow section lattice girder connections
5.08 to 5.09	- Imperfections
6.01 to 6.03	- Bolted connections
6.04 to 6.07	- Welded connections
6.09 to 6.10	- Beam to column connections
9.01 to 9.03	- Detail classes for fatigue
A01 to A02	- Connections for thin walled members
D01 to D04	- Design rules for S460 members and connections

4. EUROCODE 4 CODE FORMAT AND RELATED PARTIAL FACTOR

4.1. Introduction, Definition of γ_M values in Eurocode 4

In a similar way as for Eurcode 3 the drafting panel for Eurocode 4 agreed that the resistance rules should as far as possible be checked by calibration to tests. As the definition of partial safety factors γ_M is different in EC2 and EC 3 a thorough check was necessary for Eurocode 4 to specify γ_M for the design and for the procedure to determine design resistances from tests.

In Eurocode 3 the partial safety factor $\gamma_M = \gamma_m\ \gamma_{Rd}$ is related to the characteristic resistance function and though it is used as a unique value it includes uncertainties of the mechanical model (γ_{Rd}) and the material properties (γ_m).

$$R_d = R\,(X_{1k}\,,\, X_{2k}\,,\, ...X_{ik})\ \gamma_H = R\,(X_{1k}\,,\, X_{2k}\,,\, ...\ X_{ik})/\gamma_m\ \gamma_{Rd} \tag{4.1}$$

In Eurocode 2 where the resistance is a function of the strength of concrete and reinforcement a design format is used, where the partial safety factors γ_M are directly included in the resistance function and related to the strengths of materials.

$$R_d = R\left[\frac{X_{1k}}{\gamma_{M1}}, \frac{X_{2k}}{\gamma_{M2}}, ... \frac{X_{ik}}{\gamma_{Mi}}\right] = R\left[\frac{X_{1k}}{\gamma_{m1}\ \gamma_{Rd1}}\ ,\ \frac{X_{2k}}{\gamma_{m2}\ \gamma_{Rd2}}\ ,\ ... \frac{X_{ik}}{\gamma_{mi}\ \gamma_{Rdi}}\right] \tag{4.2}$$

For a composite member normally the resistance function is non-linear and may depend on three strengths (concrete f_{ck}, structural steel f_{yk} and reinforcement f_{sk}) and additionally on the resistance of shear connectors. In general it is not possible to use unique partial safety factors for composite members because of the interaction of the various members at a limit state.

Therefore for composite members the partial safety factor has been split into $\gamma_M = \gamma_m \gamma_{Rd}$ according to equation 4.3 where

γ_m takes into account the possibility of unfavourable deviations of the material properties and systematic part of the conversion factor and its uncertainties and

γ_{Rd} takes into account the uncertainties of the resistance model of the composite member.

$$R_d = \frac{1}{\gamma_{Rd}} R \left[\frac{X_{1k}}{\gamma_{m1}}, \frac{X_{2k}}{\gamma_{m2}}, \dots \frac{X_{ik}}{\gamma_{mi}} \right] = \frac{1}{\gamma_{Rd}} R \left[\frac{f_{ck}}{\gamma_{mc}}, \frac{f_{sk}}{\gamma_{ms}}, \frac{f_{yk}}{\gamma_{ma}} \right] \tag{4.3}$$

For ease of use it was agreed to fix the partial factors γ_{mi} by using the nominal values $\gamma_{Ma} = 1{,}1$ for structural steel, $\gamma_{Ms} = 1{,}15$ for reinforcement and $\gamma_{Mc} = 1{,}5$ for concrete as given in EC2 and EC3.

$$R_d = \frac{1}{\gamma_{rd}} R \left[\frac{f_{yk}}{\gamma_{Ma}}, \frac{f_{ck}}{\gamma_{Mc}}, \frac{f_{sk}}{\gamma_{Ms}} \right] \tag{4.4}$$

The partial safety factor γ_{rd} in equation 4.4 can then be interpreted as a special safety factor taking into account uncertainties of the"composite effect" of the various mechanical models because the values γ_{Ma}, γ_{Mc}, and γ_{Ms} according to EC2 and EC3 include already the γ_{Rd} values of EC2 and EC3. In Eurocode 4 the partial safety factor γ_{rd} is for instance relevant in all cases where buckling of steel influences the strength function (local web-buckling and lateral torsional buckling of beams or buckling of composite columns).

For design rules which are not influenced by the design formats of Eurocode 2 and 3, Eurocode 4 uses the same procedure as Eurocode 3 to determine the design resistances by comparison of the results of strength functions R_t with the results from tests R_e and evaluation of these comparisons to determine the design values R_d. A typical example is the determination of the design rules for headed studs in solid slabs. An example is given in chapter 4.3 .

The cost of testing for composite members (beams and columns) is such that replication is rare. Normally the test data consist of a group of specimens with different sizes, strengths of materials and loading and they do not represent a homogeneous population. Caused by the particular definition of the partial safety factor in Eurocode 4 some modifications are necessary for the determination of the partial safety factor γ_{rd} in the procedure of Annex Z of Eurocode 3. More explanation are given for the design method for composite columns /4.1/ in chapter 4.2 .

4.2 Determination of partial safety factors for composite columns

The procedure for the determination of γ_{rd} for the simplified design method given in Eurocode 4 Part 1-1, clause 4.8.3 was applied with the following steps:

1. The strength function R_t is given by the simplified method according to clause 4.8.3 of EC4. 208 tests were cross-checked using the measured strengths of steel, concrete, reinforcement and of the geometrical parameters. Due to lack of information in some test reports geometrical basic variables had to be presented by nominal values. Tests without sufficient information regarding the strengths of materials and the type of load introduction were not taken into account.

2. The experimental test results R_e were compared with the results R_t given by the strength function of EC4. The design value R_d was determined in accordance with Annex Z of Eurocode 3. The results are given in figure 4.1 and table 4.1 . The influence of the coefficients of variation of the basic variables and the determination of V_{rt} according EC3, Annex Z was studied on 6 representative composite columns with sections according to figure 4.2 . The following basic variables were taken into account:

   - cylinder strength of concrete f_c
   - yield strength of steel f_y and reinforcement f_s
   - flexural stiffness of concrete, steel and reinforcement
   - reduction coefficient χ for the relevant related slenderness $\bar{\lambda}$ (uncertainties of the buckling curves according to EC3)

3. 40 representative columns with cross-sections given in figure 4.2 were then calculated for several types of loading with the mean values of basic variables and typical values of the related slenderness $\bar{\lambda}$ and the steel contribution ratio δ. The design resistance of the representative columns was then determined with the results of table 4.1 .

$$R_d = 0{,}6653\ R_t \tag{4.5}$$

4. The same representative columns were then calculated with the characteristic strengths of materials and the respective partial safety factors $\gamma_{Ma} = 1{,}1$, $\gamma_{Mc} = 1{,}5$ and $\gamma_{Ms} = 1{,}15$ according to Eurocode 4. The design resistance is given by equation 4.4 .The unknown partial safety factor γ_{rd} results from the comparison of the design values of equation 4.4 and 4.5 .

$$\gamma_{rd} = \frac{R\left[\dfrac{f_{yk}}{\gamma_{Ma}}, \dfrac{f_{ck}}{\gamma_{Mc}}, \dfrac{f_{sk}}{\gamma_{Ms}}\right]}{R_d} \tag{4.6}$$

For the representative columns the values of γ_{rd} in equation 4.6 ranged between 1,03 and 0,80 with a mean value of 0,93. For the ENV-period of Eurocode 4 part 1 it was decided to use a conservative value $\gamma_{rd} = 1{,}0$ for all types of cross-sections and loadings.

4.3 Determination of partial safety factors for headed studs in solid slabs

The test evaluation carried out for headed studs in solid slabs is a typical example for the use of Annex Z of Eurocode 3, where the test results must be splitted in subsets. It is known as a fact, that the resistance of headed studs is influenced by the stud material, the diameter of the stud and the strength and modulus of elasticity of concrete /4.2/.

mean value correction factor	$\bar{b}$ = 1,1600
estimator for the mean value of resistance R_m	R_m = 1,16 R_t
coefficient of variation of the random error term δ	V_δ = 0,1641
coefficient of variation V_π of the basic variables	V_π = 0,070
coefficient of the random variable r	V_r = 0,1784
characteristic value of the resistance	R_k = 0,8533 R_t
design value of resistance	R_d = 0,6653 R_t

Table 4.1 Determination of the design strength of composite columns according to Annex Z of EC3 (208 tests)

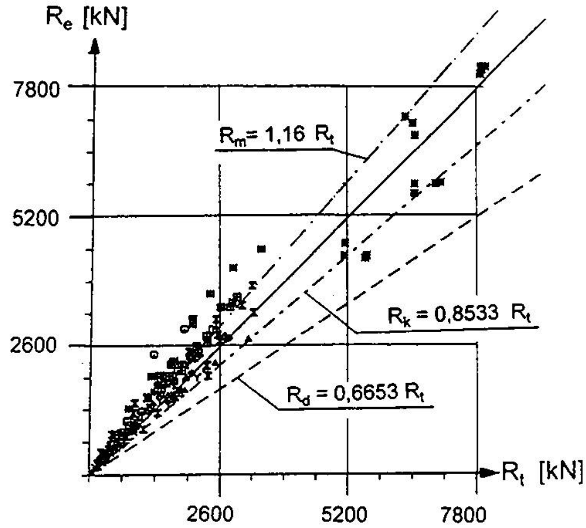

Figure 4.1 Comparison of test results (R_e) with the strength function (R_t), characteristic and design values according to Annex Z of EC3

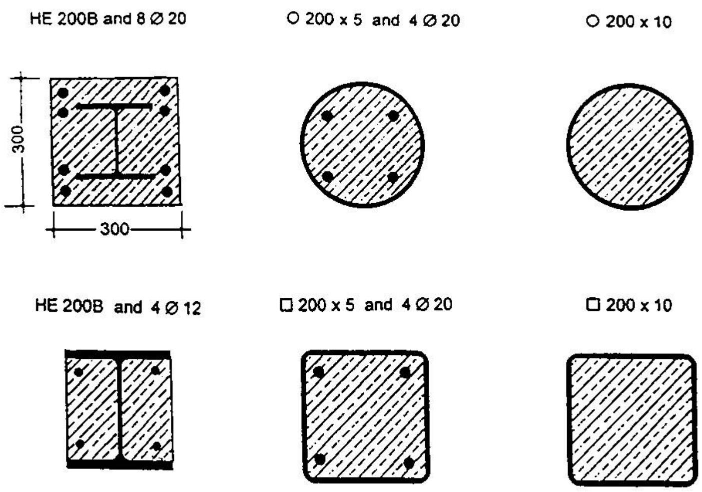

Figure 4.2 Representative cross sections for the determination of γ_{rd}

For higher concrete grades the resistance is governed by the ultimate strength of the stud material and for lower concrete grades by the material properties of concrete. Therefore two subsets with the resistance functions according table 4.2 were taken into account.

The results of the test evaluation for the determination of the design values are given in table 4.2 and figure 4.3. Using the nominal values for the concrete and stud material and the nominal partial safety factor γ_M = 1,25 for connections in accordance with Eurocode 3 the design rules of Eurocode 4 came out.

	Subset 1	Subset 2
number or tests	41	35
resistance function	$R_t = 0{,}36\ d^2\sqrt{E_c\ f_c}$	$R_t = 0{,}85\ \frac{\pi d^2}{4}\ f_u$
mean value correction factor	$\bar{b}$ = 1,038	$\bar{b}$ = 1,179
mean value of resistance R_m	R_m = 1,038 R_t	R_m = 1,179 R_t
coefficient of variation of δ	V_δ = 0,136	V_δ = 0,101
coefficient of variation V_π	V_π = 0,139	V_π = 0,078
coefficient of the random variable R	V_r = 0,194	V_r = 0,127
characteristic value	R_k = 0,727 R_t	R_k = 0,934 R_t
design value	R_d = 0,547 R_t	R_d = 0,773 R_t
partial safety factor γ_M	γ_M = 1,33	γ_M = 1,21

Table 4.2 Test evaluation for headed studs in solid slabs according to Annex Z of EC3

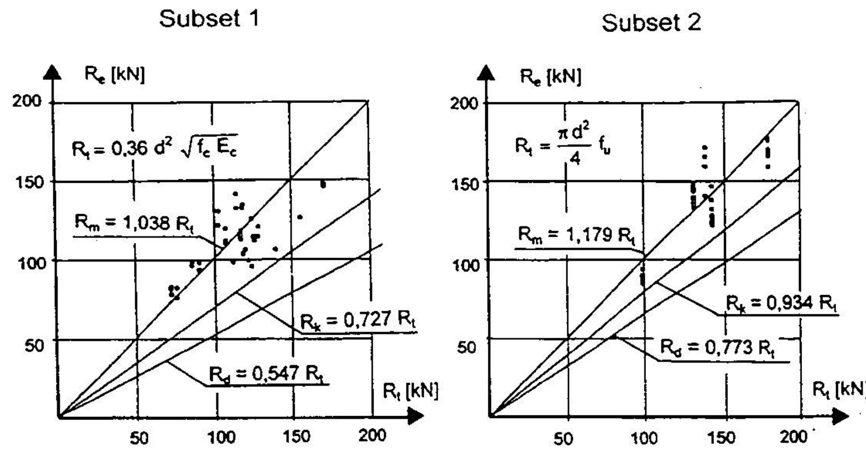

Figure 4.3 Evaluation of test results for headed studs in solid slabs according to Annex Z of EC3

4.4 Conclusions

For the main design rules in Eurocode 4 calibration studies have been carried out using the basic procedures in Eurocode 1-Part 1-1 to determine a coherent and uniform safety level. For composite columns and headed studs the special procedure for composite members is explained.

Further studies were carried out for the following subjects:
- resistance of composite beams to bending [4.3],
- resistance of composite beams to lateral torsional buckling [4.4],
- studs in combination with profiled steel sheeting [4.2], [4.8],
- fatigue resistance of headed studs in solid slabs [4.5],
- load introduction of composite columns [4.6],
- limitation of crack width in continuous composite beams [4.7].

In addition to the above reference [4.3] we would also like to make reference to studies in the UK [4.9] where slightly different conclusions were obtained. Some further discussions seems to be necessary.

References

4.1 Roik, K., Bergmann, R.:
Report on Eurocode 4: Composite Columns, Research Project Harmonisation of European Codes, Ministerium für Raumordnung, Bauwesen und Städtebau 1989

4.2 Roik, K., Hanswille, G.:
Report on Eurocode 4: Stud Connectors, Research Project Harmonisation of European Codes, Ministerium für Raumordnung, Bauwesen und Städtebau, 1989

4.3 Johnson, R.P., Huang, D.J.:
Partial Safety Factors γ_M for Composite Beams in Bending, found from Test Data, Research Report CE38, University of Warwick, Department of Civil Engineering, 1991

4.4 Roik, K., Hanswille, G.:
Report on Eurocode 4: Lateral torsional Buckling, Research Project Harmonisation of European Codes, Ministerium für Raumordnung, Bauwesen und Städtebau, 1990

4.5 Roik, K., Hanswille, G.:
Report on Eurocode 4: Limit state of fatigue for headed studs, Research Project Harmonisation of European Codes, Ministerium für Raumordnung, Bauwesen und Städtebau, 1990

4.6 Roik, K., Bergmann, R.:
Report on Eurocode 4: Load Introduction for Composite Columns, Research Project Harmonisation of European Codes, Ministerium für Raumordnung, Bauwesen und Städtebau, 1990

4.7 Roik, K., Hanswille, G.:
Report on Eurocode 4: Cracking of Concrete, Research Project Harmonisation of European Codes, Ministerium für Raumordnung, Bauwesen und Städtebau , 1990

4.8 Stark, J.W.B., Van Hove, B.W.E.M.:
Statistical analysis of push-out tests on stud connectors in composite steel and concrete structures, TNO-Report BI-91-163, 1991

4.9 Johnson, R.P., Huang, D.J.:
Calibration of safety factors γ_M for composite steel and concrete beams in bending, Proc. Institution of Civil Engineers, Struct. and Bldg., 104, 193-203, May 1994.

5. EUROCODE 5 CODE FORMAT AND RELATED PARTIAL FACTOR

5.1 Introduction

Eurocode 5: Part 1.1 (EC5) was developed using the CIB Structural Timber Design Code of 1983[CIB 1983] as the basis, and in relation to discussions at CIB-W18 meetings. Proceedings of these meetings are available, and a complete list of papers can be obtained from the latest issue [CIB-W18 1995]. EC5 refers to a large number of supporting documents which either were prepared or are being prepared by other CEN Technical Committees.

The development of an international Code of Practice for design of timber structures is a very complex process because of the need to account for various facets of material behaviour and different practices in different countries. Among the material aspects, the dependence of properties of timber and wood-based products on their moisture contents and the durations of actions is the most complex to deal with. EC5 has tried to simplify the design of timber structures as much as possible; for example through the introduction of Strength Classes, instead of using the traditional method of dealing with individual species and grades. EC5 does not deal specifically with new materials such as laminated veneer lumber (LVL) and Parallam, but its principles may be adopted for them.

5.2 Relation between Eurocodes 1 and 5

Combination of actions

The consideration and combination of actions in EC5 follow the procedures of EC1-1, except for the serviceability limit states(see below). EC5 instructs the user to refer EC1-1 for the combination factors(ψ), but the local NAD may specify the values of these boxed items differently.

The values of partial factors for actions, γ_{Gsup}, γ_{Ginf}, γ_Q of Cases A and B of Table 9.2 of EC1-1 are prescribed also by EC5 in Table 2.3.3.1 and Clause 2.3.3.1(3). However, additionally, EC5 provides a set of reduced partial factors which may be used with occasionally occupied one storey buildings of moderate span such as storage buildings, sheds, greenhouses, agricultural buildings and small silos, and also with lighting masts, light partition walls and sheeting.

The action combinations used in EC1-1 and EC5 for SLS verifications are not the same. While EC1-1 specifies three combinations in Equations 9.16 to 9.18, and also provides a simplified set of equations for buildings(Eqs. 9.19 and 9.20), EC5 specifies a single combination (Equation 4.1a) which does not correspond to any of the above. Although the EC5 combination

$$\Sigma G_{k,j} + Q_{k,1} + \Sigma \psi_{1,i} Q_{k,i} \qquad (i>1)$$

is similar to the characteristic(rare) combination of EC1-1(Equation 9.16), the frequent value coefficient ψ_1 is used for variable actions with i>1, instead of the combination value coefficient ψ_0 used in EC1-1. The result is a lesser action value than given by the characteristic combination. This dissimilarity between the two Codes needs to be resolved, and already there is a proposal for such a revision[Racher and Rouger, 1994].

Design value of resistance

In EC5, the determination of resistance is carried out according to Equation 9.7a of EC1; *i.e.* the partial factor γ_M is applied to the characteristic strength, and the geometrical properties are not factored.

Equation 9.4 of EC1-1 specifies the design value of material property (X_d) as

$$X_d = \eta X_k/\gamma_M \text{ or } X_k/\gamma_M$$

where γ_M is the partial factor and η is the strength conversion factor. EC5 uses the first definition for timber and wood-based products and the second for metals, such as steel, used in bolts, nails screws, etc. The strength conversion factor η, denoted by k_{mod} in EC5, accounts for the load(action) duration and moisture effects which are always more important for timber than for other construction materials. Depth effects, accounted for by the conversion factor k_h, and treated separately, may also be considered as a part of η.

The characteristic material properties are determined under standard conditions of moisture equilibrium at a relative humidity of 65% and an ambient temperature of 20°C. A test time to failure of 5minutes is aimed at, but with an allowable variation of ±2 minutes. These properties are converted to those at other conditions using the k_{mod} values given in Table 3.1.7 of EC5. A typical set of k_{mod} values are reproduced here in Table 5.1. The k_{mod} value depends on the Service Class of the structure and the Load-duration Class of the actions. Service Classes, of which there are three, are dependent on the exposure conditions of the structure. There are five Load-duration Classes.

In using different action combinations, the designer of timber structures has to pay special attention to the k_{mod} values to be used for determining design material properties. The critical action combination for a given case would be highly dependent on the k_{mod} value to be used. For action combinations with variable actions, the k_{mod} value is to be selected according to the action with the shortest duration .

Load duration class	Service Class		
	1	2	3
Permanent	0.60	0.60	0.50
Long term	0.70	0.70	0.55
Medium term	0.80	0.80	0.65
Short term	0.90	0.90	0.70
Instantaneous	1.10	1.10	0.90

Table 5.1 k_{mod} for solid timber, glued laminated timber and plywood.

Calibration of EC5

The calibration of design equations in the current version of EC5 was carried out with respect to current practice in member countries. The calibration procedure was constrained by other material dependent Eurocodes. The partial factors on actions have been constrained by the fact that all Eurocodes need the same material independent relations for the actions side of the design equations. This is a problem which occurred also during the reliability-based calibration of the Canadian timber code[Foschi *et.al.* 1989]. During the calibration of EC5, even the partial factor on material properties γ_M was constrained by those already used for steel (1.1) and concrete(1.5), and the value was kept between them at 1.3 for ultimate limit state verifications: a lower value probably being questioned by users of other materials[Larsen 1992a].

Some of the restrictions during calibration of EC5 occurred because there was no safety philosophy formulated for the design of timber structures [Larsen, 1992a].- This is an area which should be given priority. The formulation of a reliability-based limit state design philosophy, which does not neglect accumulated knowledge as reflected in current designs, may be initiated through preliminary studies on the reliability levels in existing timber structures. Such a study would also help to identify areas in which more information needs to be gathered.

5.3 Treatment of Durability

Section 2.5 of EC1-1 refers to the importance of durability requirements in structural design. The structural and aesthetic performances of timber and wood-based materials depend, to a large extent, on its preservation. Hence, durability considerations, including detailing for this purpose, take a very important part in a design procedure. Two types of durability problems are considered by EC5, *viz.* durability of timber and wood-based materials against biological attack (i.e by either insects or fungi) and resistance of metal fasteners and other components against corrosion. The corrosion protection needed is specified according to the Service Classes (see above). Timber and wood-based materials are required to have adequate natural durability or be given a preservative treatment. These, specified according to a set of Hazard Classes, are to be carried out as specified in relevant ENs.

References

5.1. CIB (1983) Structural Timber Code. CIB Report, Publication 66.
5.2. CIB-W18 (1995); Proceeding of Meeting 28, Copenhagen, Denmark.
5.3. Foschi, R.O., Folz, B.R. and Yao, F.Z.(1989); *Reliability-based design of wood structures,* Structural Research Series, Report No. 34, Dept. of Civil Eng., The Univ. of British Columbia, Vancouver, Canada.
5.4. Larsen, H.J.(1992a); *Latest developments of Eurocode 5*; Proc. CIB-W18, Meeting 25, Ahus, Sweden.
5.5. Larsen, H.J. (1992b); *EC5: Design of Timber Structures*, Proc., IABSE conf. on Structural Eurocodes, Davos, Switzerland.
5.6. Larsen, H.J.(1995); *Limit state design and safety format,* in Timber Engineering Step-1, Edited by Blass, H.J. *et.al.*, Centrum Hout, The Netherlands.
5.7. Racher, P. and Rouger, F.(1994); *Serviceability limit states - A proposal for updating Eurocode 5 with respect to Eurocode 1.* Proc. CIB-W18, Meeting 27, Sydney, Australia.
5.8. Racher, P.(1995); *Actions on structures,* in Timber Engineering Step-1, Edited by Blass, H.J. *et. al.*, Centrum Hout, The Netherlands.