Plenary Session 4

Integration with other Eurocodes

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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 :

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

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).

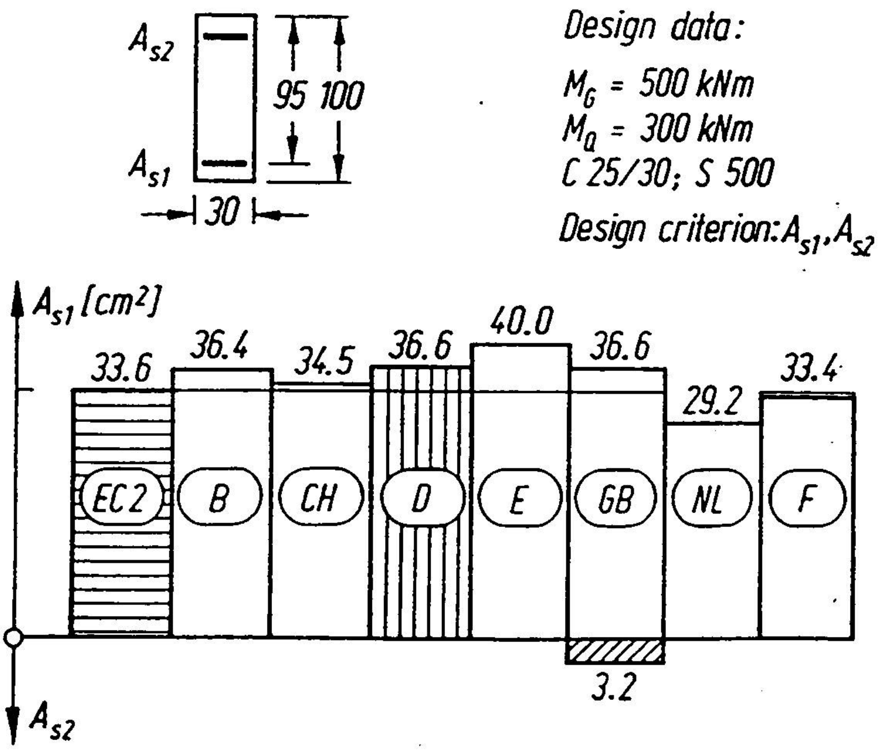

Figure 2.1 - Design of rectangular cross-section according to national regulations of some European countries (characterized by their registration plate)

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

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

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

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

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:

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

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

$$V_R = \sqrt{V_m^{\,2} + V_G^{\,2} + V_f^{\,2}} \qquad (2.8)$$

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}).

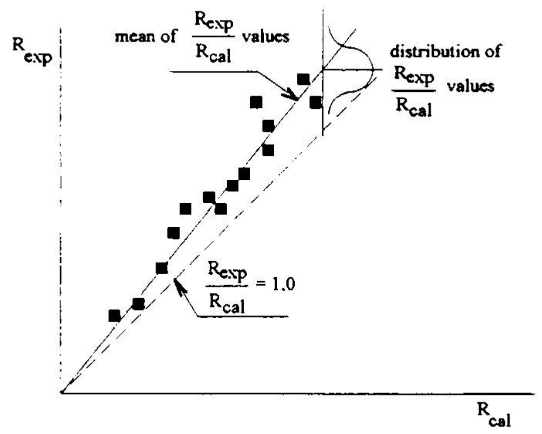

Figure 3.1 - Comparison of the theoretical strengths R_{cal} with test results R_{exp}

- 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.

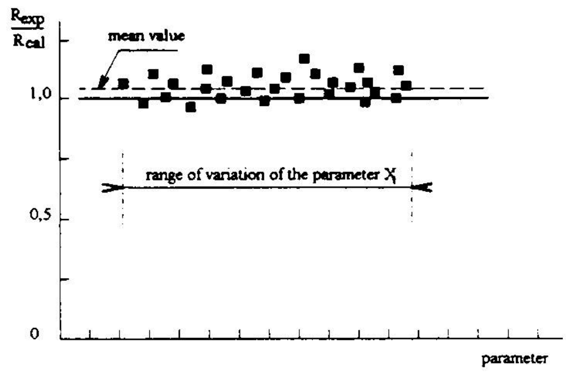

Figure 3.2 - Sensitivity of the strength function with respect to parameter X_i

- 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.

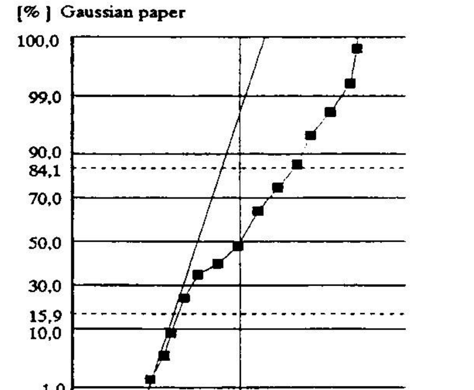

Figure 3.3 - Determination of the lower tail distribution from plotting the test population on a Gaussian paper

- 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.

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

For ease of use it was agreed to fix the partial factors γ_{mi} by using the nominal values γ_{Ma} = 1,1 for structural steel, γ_{Ms} = 1,15 for reinforcement and γ_{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)

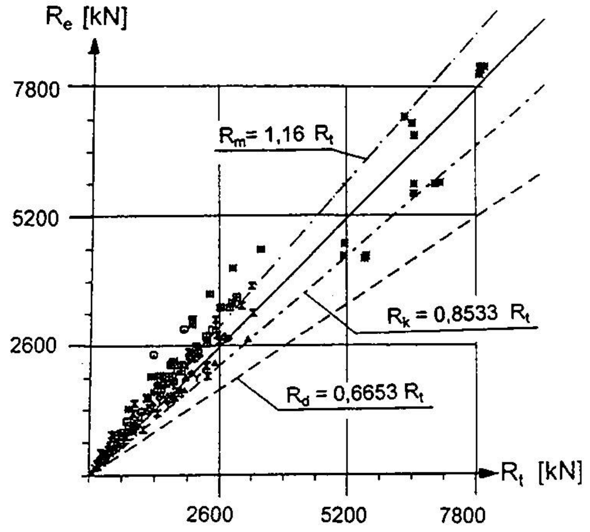

Figure 4.1 Comparison of test results (R_e) with the strength function (R_t), characteristic and design values according to Annex Z of EC3

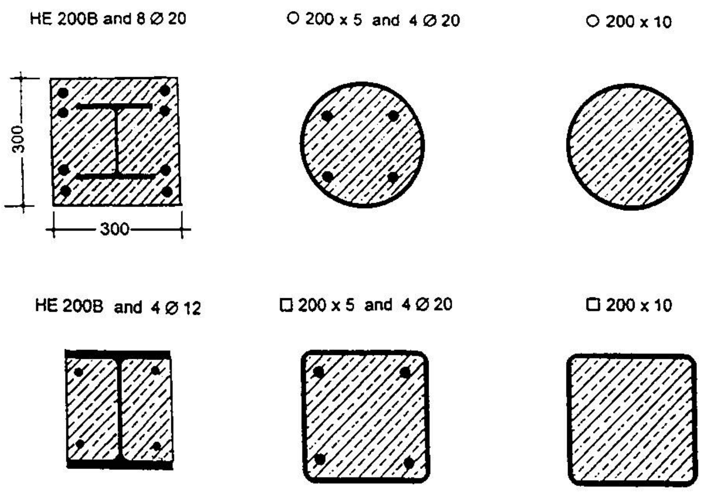

Figure 4.2 Representative cross sections for the determination of γ_{rd}

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

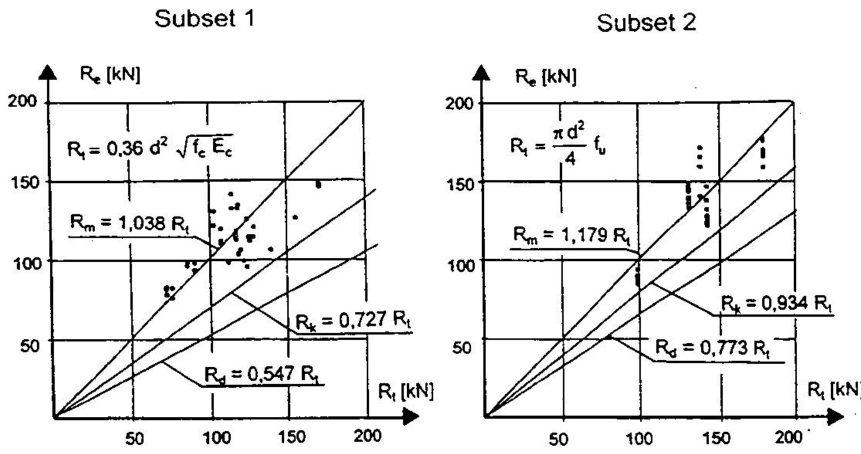

Figure 4.3 Evaluation of test results for headed studs in solid slabs according to Annex Z of EC3

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.

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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, $\gamma_{G,sup}$, $\gamma_{G,inf}$, 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.

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Basis of design in Eurocode 7

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Summary

Eurocode 7 provides rules for Geotechnical Design for all structures designed to the Eurocodes. Its development has required very careful definition and application of the concepts of limit state design, partial factors and characteristic values of material properties. The code is consistent with Eurocode 1 and the other Eurocodes, whilst remaining within the principles of sound geotechnical engineering. Examples are presented of applications to foundations and retaining structures.

1. Introduction

Eurocode 7 (EC7) provides rules for Geotechnical Design for all structures designed to the Eurocodes. The geotechnical community, and in particular the Project Team for EC7 Part 1, have accepted the challenge to provide a code which is consistent with Eurocode 1 (EC1) and the other Eurocodes, and also with the principles of sound geotechnical engineering. This has required very careful definition and application of the concepts of limit state design, partial factors and characteristic values of material properties. The code allows design by calculation, by prescription, by testing and by the Observational Method.

EC7 Part 1 provides general rules for design. Following the introductory Section 1, Basis of Geotechnical Design is considered in Section 2. Sections 3 to 5 deal with the investigation of natural ground and requirements for construction, monitoring and maintenance both of engineered fill and of structures supported by the ground. The remaining sections consider particular types of structures in more detail: shallow foundations, piled foundations, retaining structures, slopes and embankments. These later sections necessarily add details to the basis of design for specific cases. In this paper, clause numbers are noted in [square brackets].

EC7 Parts 2 and 3, which have not yet reached ENV status, deal with the use of laboratory and field testing, respectively, in geotechnical design.

2. EC7 Section 2 - Basis of Geotechnical Design

2.1 Recognition of complexity - Geotechnical categories

All terrestrial construction has a geotechnical component. However, the difficulty and complexity of this component varies greatly from one situation to another, and the degree of expertise and attention required therefore also varies. An attempt has been made to classify this requirement by defining *Geotechnical Categories*. Category 1 includes only small structures in simple, well understood situations and may not require the involvement of a civil engineer in the design. Category 2 is for normal structures, requiring at least a qualified civil engineer, whilst Category 3 is for particularly difficult situations where experienced specialists are required.

The code is directed primarily at Category 2. It is foreseen that quantitative measurement and calculation may be unnecessary for Category 1, whilst Category 3 may require procedures beyond the scope of the code. No attempt is made to vary the values of safety factors between the categories; it is considered that safety in geotechnical engineering is governed more by the quality of geotechnical investigation and workmanship than by precision in calculation models and partial factors [2.4.1(2)].

In the writer's opinion, it is unclear whether this system of categorisation can be applied successfully to typical projects in which some geotechnical elements are very straightforward whilst others present considerable difficulty.

2.2 Limit state design

The concepts of limit state design are applied to geotechnics in EC7. Four features which have caused much discussion are noted here.

a) Limit states are generally defined in terms of *damage* to *structures*. Damage to the ground is rarely of significance in itself. Traditional geotechnical calculations have related either to pseudo-elastic states or to plastic mechanisms, but these do not necessarily correspond directly to serviceability and ultimate limit states. In particular, structures are sometimes brought to ultimate collapse by ground movements when the ground itself is far from a state of plastic mechanism. Examples of this include heave due to swelling clays and negative skin friction (downdrag) on piles.
b) The concept of an 'action' requires careful definition, particularly in earth pressure problems and other situations of ground-structure interaction. The key definition is considered to be *a force (or imposed displacement) which is a known quantity at the start of the current calculation* - that is, a force which is not a reaction [2.4.2(1)P]. This allows the possibility that partial load factors can be applied to actions, which would be difficult, or impossible, for reactive forces. It also implies that some forces will be reactions in one calculation but will be classed as actions in a later calculation.
c) Even in ultimate limit state design, it is essential that all design strengths can be mobilised simultaneously. Thus compatibility of strains and displacements must be considered, though very large displacements may sometimes be allowed.
d) In the design of retaining walls, it is common that the earth pressures on the wall reduce as deformation proceeds. Thus the earth 'load' on the wall may be lower in ultimate limit

state conditions than in service. This is an unusual situation for structural design.

e) Serviceability limit states, involving displacements, are often critical to the design of structures in geotechnical situations. However, displacements are difficult to calculate and traditional designs have largely covered this difficulty by use of appropriate factors of safety in analyses of plastic mechanisms. Since the values of partial factors are partly based on experience of successful designs, it can be expected that this will remain the case, though it involves a dilution of the concept of limit state design.

2.3 Characteristic values of ground material properties

EC1, Section 5, requires that characteristic values of material properties shall have "a prescribed probability of not being attained in a hypothetical unlimited test series". It also requires that "a conversion factor shall be applied where it is necessary to convert the test results into values which can be assumed to represent the behaviour of the material in the structure or the ground". It proposes that characteristic values should be defined as a 5% fractile for strength parameters and as the mean value for stiffness parameters. Attention will be concentrated here on strength parameters, whilst the 'mean value' requirement for stiffnesses will be discussed later.

These requirements of EC1 are not rigorously consistent. However, an attempt has been made to apply them, with due regard to the following special features of geotechnical design.

a) In geotechnical design, the designer usually is in possession of site-specific information which gives him much more knowledge of the uncertainties of material properties than the code drafter could possibly have. This is the reverse of the normal situation in structural design.
b) Poor performance of a small element of the ground, of the size involved in a field or laboratory test, is usually of no consequence to the performance of a structure. This is sometimes not the case in structural design. Thus geotechnical characteristic values will often be mean values spacially, though they are not to be means in a probabilitic sense.
c) It is often good practice for the designer to consider in combination several sets of data in order to derive an appropriate characteristic value. These will have varying degrees of relevance and reliability and will often include some conflicts. Observation of the performance of other structures is one important source of data, together with geological and other background information.
d) The operative properties of the ground may be changed by construction activities.

EC7 requires that the designer takes all of these factors into consideration in assessing characteristic values. It then requires "a cautious estimate of the value affecting the occurrence of a limit state" [2.4.3(5)P]. EC7's approach means that characteristic values are somewhat subjective, depending on the knowledge and experience of the designer. However, an alternative approach in which these were disregarded in order to attain uniformity in the assessment of characteristic values would involve dangerous neglect of vital information. The writer has presented an example of the assessment of a typical, but complex set of field and laboratory data for one site (see Krebs Ovesen (1995)).

An application rule points out that statistical analysis may be used provided that proper account is taken of *a priori* knowledge. Where statistical methods are used, "the characteristic value should be derived such that the calculated probability of a worse value *governing the occurrence of a limit state* is not greater than 5%". Schneider (see Krebs Ovesen(1995)) has proposed a statistical approach which covers Item (b) above, taken alone, when deriving a characteristic value from normally distributed results of tests on small samples. He suggests that, to give a value with a 5% probability of *governing the occurrence of a limit state*, the characteristic value should be ½ a standard deviation from the mean of the test results. In simple cases, this is probably consistent with typical engineering assessments of test results, such as that described as moderately conservative by Padfield and Mair (1984).

2.4 Actions - Cases A, B and C

EC1, Table 9.2 requires that designs should be verified for Cases A, B and C 'separately as relevant'. The unfortunate words 'as relevant' are copied into EC7, but succeeding clauses make it clear that all designs must satisfy all three cases *in all respects* - geotechnical and structural. The bracketed values of partial factors are shown in Table 1; they are varied for the three cases for geotechnical material strengths as well as for actions.

Case	Actions			Ground Properties			
	Permanent		Variable				
	Unfavourable	Favourable	Unfavourable	$\tan \phi'$	c'	c_u	q_u [1)]
Case A	[1.00]	[0.95]	[1.50]	[1.1]	[1.3]	[1.2]	[1.2]
Case B	[1.35]	[1.00]	[1.50]	[1.0]	[1.0]	[1.0]	[1.0]
Case C	[1.00]	[1.00]	[1.30]	[1.25]	[1.6]	[1.4]	[1.4]

1) Compressive strength of soil or rock.

Table 1. EC7 Table 2.1: Partial factors — ultimate limit states in persistent and transient situations

In EC7, Case A is used only to check against buoyancy. This is a geotechnical equivalent to the 'stability' check of structural design, since it involves a balance of actions with no, or little involvement of the strength of materials. In practice, however, it is sometimes the case that the strength of ground plays a partial role in preventing buoyancy problems. It was therefore considered appropriate to provide partial material factors for use in these cases; these could, perhaps, be made equal to those of Case C. The writer questions whether the value of 0.95 applied to beneficial weight is sufficiently low to give safety in buoyancy problems.

Case B originated from structural engineering considerations and Case C from geotechnical, thus in Case B safety is derived from load factors and in Case C from ground material factors. During the drafting process it was realised that both cases had merit; no logic has been found to suggest that a case considered necessary for geotechnical stability can be disregarded in checking structural strength, or *vice versa*. It was considered that if partial factors were applied simultaneously to structural materials, actions and ground materials the results would be more pessimistic than those of other aspects of the structural design. The two cases therefore have factors applied to structural material strengths (as in other Eurocodes), together with either factored actions or factored ground materials. Researchers working for

The European Sheet Piling Association have suggested that this procedure leads to unnecessarily conservative designs. This will be considered later in an example.

EC7 acknowledges that application of Case B may sometimes lead to physically unreasonable design actions. This is particularly true when factors are applied to water pressures. In these cases EC7 allows the same factors to be used as model factors, applied directly to structural internal forces and bending moments.

2.5 Design by prescriptive measures, testing and the Observational Method

EC7 recognises that design may be based on three procedures other than use of normal calculations. Use of combinations of the four approaches is encouraged.

Prescriptive measures are, in effect, commonly recognised conservative details. They may be used, for example, in design for corrosion or frost protection, drainage requirements or even safe bearing pressures. *Design by testing* is particularly relevant to piling and ground anchors. An example will be given below.

The Observational Method is an approach in which the design may be modified on the basis of the observed performance of the partly completed structure. It can be used to permit construction to start, using a less pessimistic design than normally required, provided that contingency plans are clearly established in case the optimism is not justified by observation. The requirements of this approach are given in some detail, with the intention that design on this basis will have no greater risk of failure than on the basis of normal calculations.

3. Examples

3.1 Shallow foundation

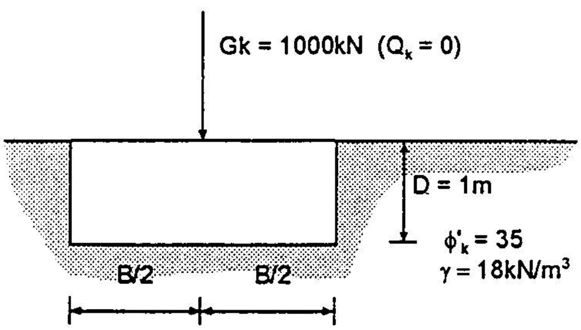

Figure 1. Centrally loaded footing.

Figure 1 shows a centrally loaded square footing to be constructed in sand for which the characteristic angle of shearing resistance ϕ_k is 35° and the characteristic unit weight γ_k is 18 kN/m³. Section 6 of EC7 provides bearing capacity factors and also states that "For rigid foundations, the bearing pressure may be assumed to be distributed linearly. More detailed analysis of soil-structure interaction may be used to justify a more economic design,..." [6.8].

For this problem, Case A is irrelevant. A calculation for Case B leads to a required footing width of 1.05m with a maximum bending moment of 177kNm assuming a linear distribution of bearing pressure (169kNm/m across the 1.05m width of the square footing). Case C requires a width of 1.29m with a maximum bending moment of 161kNm (125kNm/m). The footing width is therefore governed by Case

C and must be at least 1.29m. If it is now assumed that the bearing pressure is distributed linearly for the loading of Case B, and the width is 1.29m, the maximum bending moment becomes 218kNm (169kNm/m across the 1.29m width). This is the ULS *design* moment for structural design, unless a more complicated calculation is carried out to justify a non-linear distribution of bearing pressure.

Calculations for the serviceability limit state, in terms both of crack widths and settlement, follow traditional lines.

3.2 Piled foundation

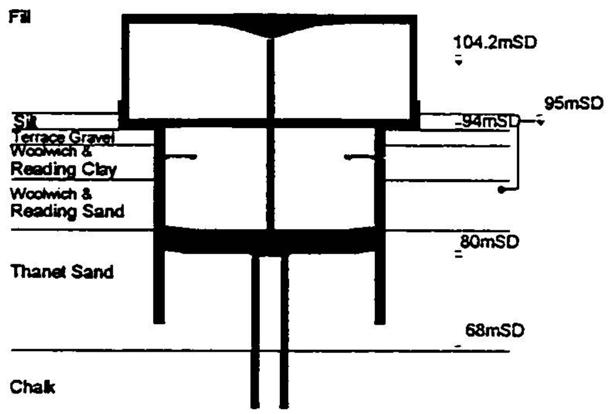

Figure 2. Underground metro station

Case	A	B	C
Water level mOD	110	104.2	110
γ_G (favourable)	0.95	1.0	1.0
γ_{water}(unfavourable)	1.0	1.35	1.0
Design tension force F_d kN	4823	5595 *Structural design*	4379
Conversion factor ξ	1.5	1.5	1.5
Partial factor γ_m	1.4	1.0	1.6
$\xi\,\gamma_m$	2.1	1.5	2.4
Required test result R_{tm} kN	10128	8393	10510 *Critical*

Table 2. Calculations for tension pile.

Figure 2 shows a cross section through a proposed underground metro station. Tension piles are required to hold down the base against buoyancy forces, which depend on the likely future piezometric level of the water in the aquifer, which is uncertain. The design is to be based on a load test.

Table 2 shows the results of calculations. For cases A and C the 'worst credible' water level of 110mSD (site datum) has been used. However, in Case B design actions are generally derived by factoring characteristic values, so a 'worst probable' level of 104.2mSD has been used as a characteristic value. The table shows that in this case the ULS *design* tension force in the pile is given by Case B as 5595kN, which should be used for structural design.

Section 7 of EC7 requires that where only a single load test is used, giving a measured resistance R_m, the characteristic resistance of the pile be taken to be R_m/ξ, where ξ=1.5. (The section also requires that the result of the load test be shown by calculation to be within reasonable expectations.) Section 7 gives partial factors for pile design, different from those of Table 1 above, which convert characteristic values to design values.

In this situation it is found that the structural design is governed by Case B, whilst the required result of the pile load test is governed by Case C. Further calcualtions, not included in Table 2, show that if the dead weight of the structure is increased, the case governing the load test requirement becomes first Case B and then Case A.

3.3 Sheet pile wall

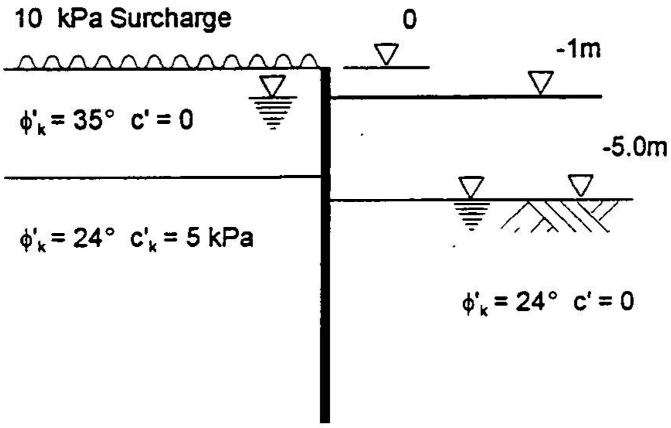

Figure 3. Sheetpile

Figure 3 shows a proposed sheet pile retaining wall for which Table 3 shows some calculated results. Section 8 of EC7 requires that when walls are supported by passive pressure, a top layer of the passive material, in this case 0.4m thick, be disregarded in the calculation. In other respects, the calculation follows the requirements of Table 1 above. EC7 does not specify how the calculations are to be carried out, except that equilibrium must be achieved.

* computed # assumed	Length (m)	BM
Case B - simple active / passive pressures	9.6 *	371
Case C - simple active / passive pressures	11.3 *	431
Case C - SAFE finite element program	11.3 #	350
Case C - FREW pseudo-finite elements	11.3 #	327
Case C - SPOOKS plasticity solution	11.5 *	376

Table 3. Computed lengths and ULS design bending moments (kNm/m) for sheet pile wall.

In rows 1 and 2 of Table 3, simple active and passive earth pressures are assumed. Case C governs the design of wall length and gives a ULS design bending moment of 431kNm/m, compared with 371kNm/m for Case B. The length obtained from this calculation is in line with conventional calculations, or slightly shorter. However, concern has been expressed that the design bending moment for Case C is too high; the option of disregarding this and using the Case B moment has been proposed. Table 3 also shows the results of three calculations which use the same Case C material parameters, but allow for soil-structure interaction in deriving the earth pressure distribution. This is specifically permitted by EC7 and leads to considerably smaller bending moments, more in line with conventional design.

4. Serviceability limit state

Foundation failures involving ultimate collapse are rare, but serviceability failures are too common. The limit state approach of EC7 may help to identify separately the ultimate and serviceability limits, but the code does not provide much guidance on the assessment of serviceability, which is usually related to deformation and, therefore, ground stiffness. Partial factors of unity are applied in SLS design, and EC1 states that the characteristic values of material stiffnesses should be mean values. This appears to imply that 50% of all constructions should be expected to exceed serviceability limit states, which is not acceptable.

5. Concluding remarks

The current draft of Eurocode 7 achieves a large degree of consistency with Eurocode 1, using sound geotechnical principles. The following points have been found to be critical, and are still somewhat controversial.

a) Characteristic values for ground properties are defined as cautious estimates of the values actually occurring in the ground in such a way as to govern the occurrence of a limit state. To obtain these, adjustments may be needed to results of soil tests and allowance is to be included for the effects on soil parameters of construction activities. All relevant information is to be included in the designer's assessment of characteristic values. A strict statistical approach will often be unhelpful.

b) The final design, with the geometry as it will be built, is to be verified for all three cases A, B and C. Calculations are not needed for cases which, by inspection, will not govern the design.

c) EC7 gives rules for derivation of design paramter values, but allows the use of any means of analysis consistent with basic principles of mechanics, including compatibility of displacement. Studies of soil-structure interaction may therefore be used to improve efficiency in design of both foundations and retaining walls.

d) The requirement in EC1 that characteristic stiffnesses should be mean values appears to imply that 50% of all designs would be expected to exceed the serviceability limit states of displacement. This is not acceptable.

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Assessment of Eurocode 8 Seismic Force Calculation

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Summary

The basis of earthquake force calculations given in EC8 are reviewed in this paper. After a description of the procedure given in the code, a brief assessment of the various regulations is undertaken. Special emphasis is placed on the recommended spectral shapes and values for the horizontal and vertical earthquake components. Comparison with a carefully selected data set, mainly from strong near-field earthquakes, indicates that EC8 horizontal spectrum caters well for such potentially damaging events whilst the vertical spectrum is unconservative.

1. Introduction

Parts 1.1 (Seismic Action), 1.2 (General and Building) and 1.3 (Material Related Chapter) of EC8 have been voted upon and have been subsequently issued as ENV. This is a milestone in the development of EC8, which has occupied many researchers and practitioners for several years.

In the United Kingdom the British Standards Institution committee B/525/8 is entrusted with collating and distilling comments on the code. In spite of the very low seismic hazard level in the UK, and the known reluctance of official organisations in the UK to foster even the simplest of lateral robustness regulations, UK earthquake engineers have been particularly active in commenting on the code thus reflecting the quality of research and development, as well as intellectual interest, of UK engineers in the subject.

The work presented below is not intended to represent a European perspective, but rather a personal one. It is based on the author's work, in collaboration with colleagues from Imperial College and European, particularly within the network 'Prenormative Research in Support of EC8' funded by the European Community.

2. Basic Requirements and Seismic Action

EC8 requires seismic design to be undertaken for a single event, the design earthquake. This is in contrast to the Japanese and the New Zealand codes, where a serviceability and a maximum earthquake scenarios are explicitly given. It is implicit in EC8 that the safety verification for the design event would lead to a no-collapse condition under the maximum event. Whilst the no-collapse condition is deemed to be satisfied if the structure is designed to resist forces calculated using the behaviour factor q, and the elastic response spectrum, the serviceability criterion is imposed by specifying deformation (drift) limits.

Different levels of design reliability are included my means of an importance factor γ_I. This parameter implicitly also accounts for different return periods of the design event as well as different probabilities of exceedance of the design ground acceleration.

2.1. Limiting Ground Accelerations and Response Spectra

The code depicts that seismic design is not necessary where a design ground acceleration (not clearly defined) is equal to or less than 0.04g. Simplified procedures of design and detailing may be used for design ground accelerations of 0.1g or less. The design ground acceleration is evaluated as equal to the peak ground acceleration for medium to large events at moderate to long distances from the site. It is less than the peak ground acceleration for small events in the vicinity of the site.

2.1.1. Elastic Response Spectrum

The elastic spectrum for the horizontal component of earthquake ground motion is given as follows:

$T \leq T_B$

$$S_e = a_g\, s \left[1.0 + \frac{T}{T_B} (\eta\beta_o - 1.0) \right] \tag{1}$$

$T_C \geq T \geq T_B$

$$S_e = a_g\, s\, \eta\beta_o \tag{2}$$

$T_D \geq T \geq T_C$

$$S_e = a_g\, s\, \eta\beta_o \left[\frac{T_C}{T}\right]^{k_1} \tag{3}$$

$T \geq T_D$

$$S_e = a_g\, s\, \eta\beta_o \left[\frac{T_C}{T_D}\right]^{k_1} \left[\frac{T_D}{T}\right]^{k_2} \tag{4}$$

where S_e Spectral acceleration normalised by g

a_g Peak ground acceleration in g

s Soil condition parameter

η Damping correction (other than for 5% damping) given by

$$\sqrt{\frac{7}{2+\xi}} \geq 0.7$$

ξ percentage of critical damping

T_B, T_C, T_D	Corner periods
k_1, k_2	Exponents
β_o	Amplification factor for zero period acceleration

The various parameters are given in Table 1 below.

Soil Class	s	β_o	k_1	k_2	T_B	T_C	T_D
A	1.0	2.5	1.0	2.0	0.10	0.40	3.0
B	1.0	2.5	1.0	2.0	0.15	0.60	3.0
C	0.9	2.5	1.0	2.0	0.20	0.80	3.0

Table 1. Parameters for EC8 Horizontal Spectrum

The vertical spectrum is defined as a function of the horizontal spectrum as follows:

$$T < 0.15 \qquad S_{av} = 0.7\,S_{ah} \tag{5}$$

$$T > 0.5 \qquad S_{av} = 0.5\,S_{ah} \tag{6}$$

$$0.15 \leq T \leq 0.5 \qquad \text{linearly interpolated between 0.7 and 0.5} \tag{7}$$

In Figure 1, the shapes of typical vertical and horizontal spectra for EC8, intermediate soil class (B) are shown. This indicates that the code, in common with all existing seismic codes, defines the vertical spectral amplification as a fixed (70%) of the horizontal value. Moreover, whereas EC8 has opted for an improved spectral shape, where by there is a slight difference in the frequency content of the two components for the intermediate period range, the ratio of vertical-to-horizontal peaks drops to 0.5. Another novelty of EC8 is the introduction of a third corner period T_D to account for reduction of amplification in the very long period range (> 0.3 sec).

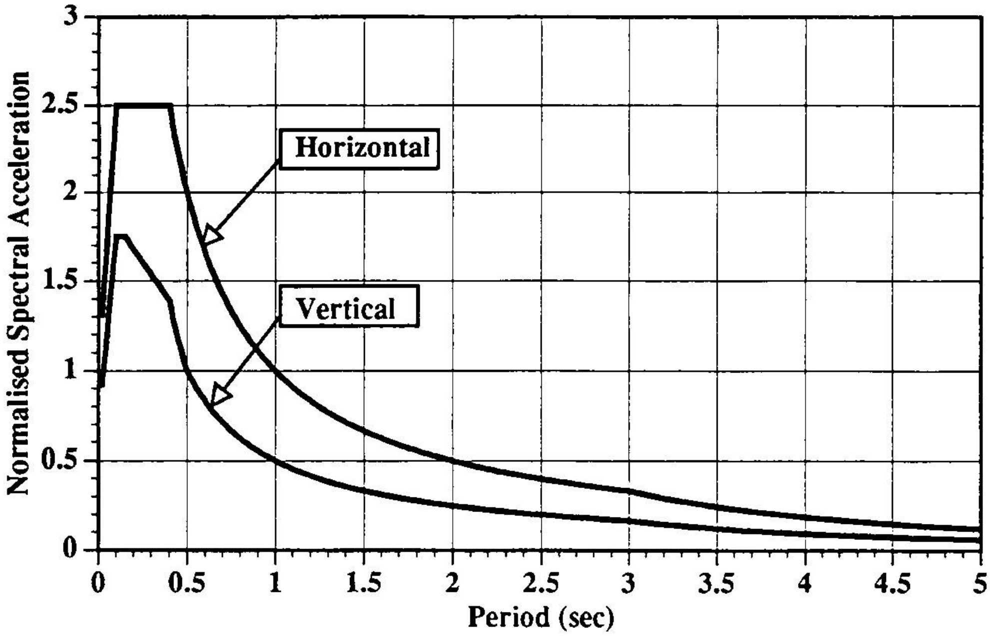

Fig. 1. Horizontal and Vertical Spectra for Soil Class B in EC8

2.1.2. Behaviour Factors 'q' and Design Spectra

Seismic design hinges on the selection of an appropriate behaviour factor (R in US practice and q in European practice). This parameter accounts for the force-delimiting mechanisms from hysteretic and other sources of damping for structures responding in a controlled inelastic manner. EC8 recommends q factors for various structural systems and construction material, to arrive at design spectra, given by the following:

$T \leq T_B$

$$S_d(T) = \alpha\, s\, [1 + \frac{T}{T_B}(\frac{\beta_o}{q} - 1)] \qquad (8)$$

$T_C \geq T \geq T_B$

$$S_d(T) = \alpha\, s \frac{\beta_o}{q} \qquad (9)$$

$T_D \geq T \geq T_C$

$$S_d(T) = \alpha\, s \frac{\beta_o}{q}\left[\frac{T_C}{T}\right]^{k_{d1}} \geq 0.2\,\alpha \qquad (10)$$

$T \geq T_D$

$$S_d(T) = \alpha\, s \frac{\beta_o}{q}\left[\frac{T_C}{T_D}\right]^{k_{d1}}\left[\frac{T_d}{T}\right]^{k_{d2}} \geq 0.2\,\alpha \qquad (11)$$

All the symbols are as defined for equations 1 through 4, whilst a is the ratio of ground acceleration to gravitational acceleration, and k_{d1} and k_{d2} are exponents assuming values dependent on the soil class (recommended at 2/3 and 5/3, respectively, in EC8).

3. Assessment of Elastic Spectra

The elastic spectra for horizontal and vertical action in EC8 are supposed to represent uniform hazard, ie. the probability of exceedance for all structural periods is approximately the same. In an attempt to assess the adequacy of the EC8 spectral shapes and amplification factors, a carefully selected earthquake data set was examined. These represent records within about 25 km from the recording station, at intermediate depths and magnitudes above m_s=5 (Ambraseys and Srbulov, 1995). In Figure 2, the 5% damped horizontal elastic spectrum of EC8 is compared to the mean and the mean plus one standard deviation spectrum from the selected data set, comprising 35 earthquakes (Elnashai and Papazoglou, 1995). It is evident that the EC8 horizontal spectrum is provides and excellent fit, especially when taking into account that the data set was selected without regard or relationship to Eurocode 8.

The spectra for the vertical components of the same set of earthquakes are compared to EC8 vertical spectrum in Figure 3. It is clearly demonstrated in the above that the vertical spectrum in EC8 requires major changes to render it representative of observed earthquake motion, especially within 20-30 km from the site. Herein, no comment is made regarding the structural significance of the vertical component of earthquake motion; analytical and observational assessment of this is given elsewhere (Elnashai and Papazoglou, 1995).

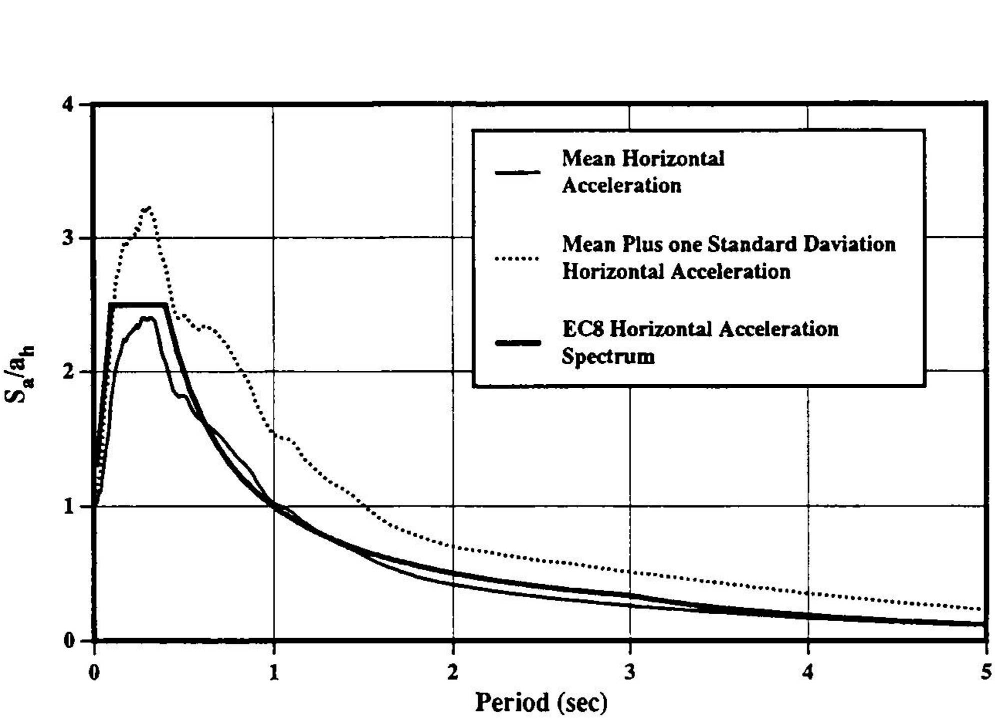

Fig. 2. Comparison of EC8 Horizontal Spectrum and 35 Earthquake Spectra

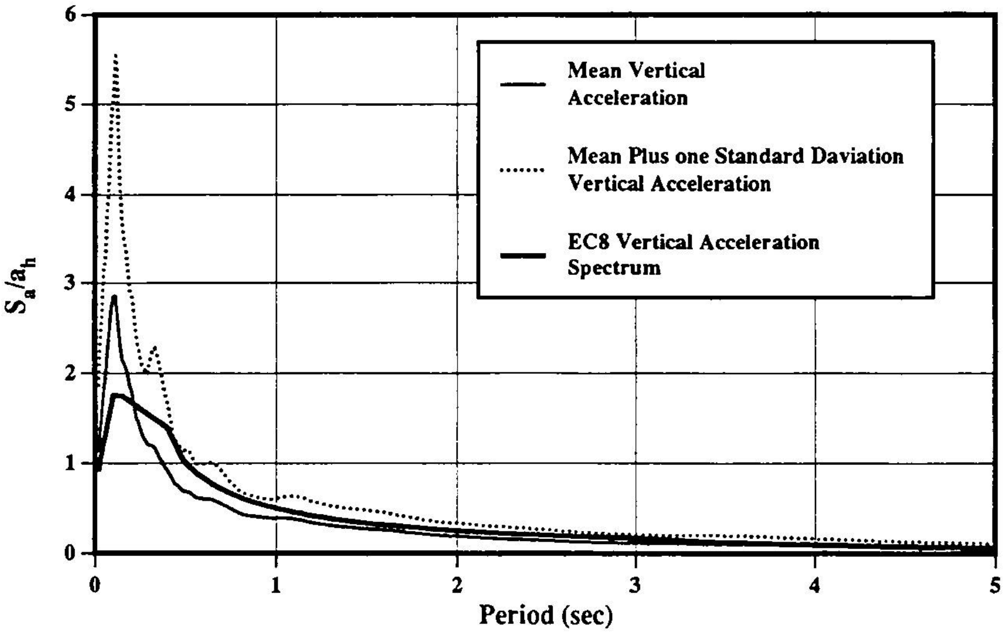

Fig. 3. Comparison of EC8 Vertical Spectrum and 35 Earthquake Spectra

The main reason for the discrepancy between the EC8 vertical spectrum and the measured spectra is the fixed ratio between ground accelerations in the horizontal and vertical directions employed by the code, in common with all existing seismic codes. Observations indicate that this ratio is dependent on earthquake magnitude and source-site distance. In the work of Elnashai and Papazoglou (1995), a variable ratio is proposed, based on engineering seismology studies by Ambraseys and Simpson (1995) and Abrahamson et al (1989). These relationships are depicted in Figure 4, where the code-specified fixed ratio is also indicated. Use of such an approach will render EC8 vertical spectra representative of true observations. Recommended spectral shapes were also given in Elnashai and Papazoglou (1995).

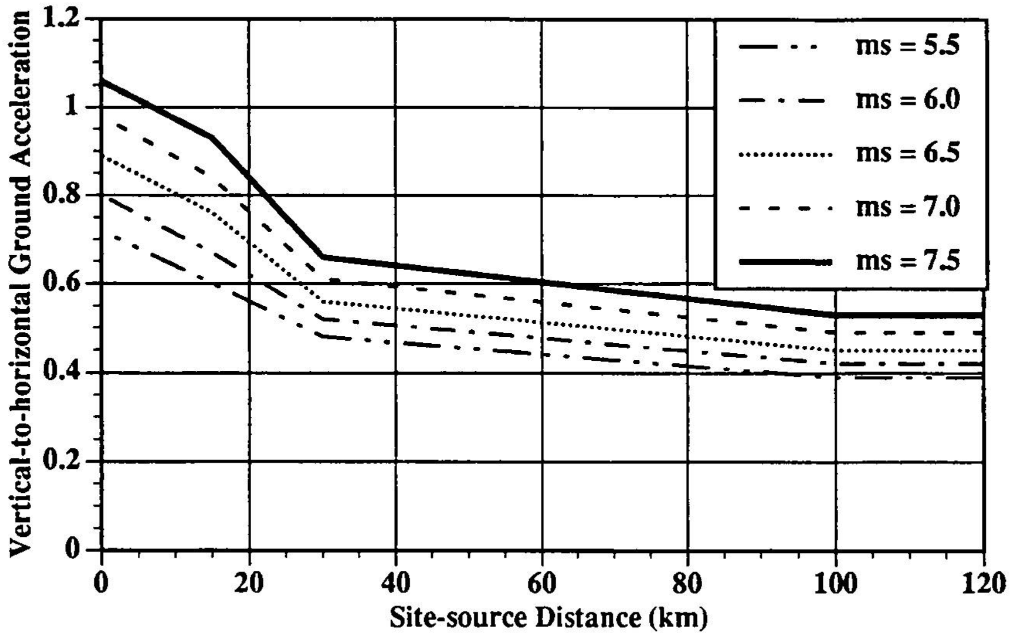

Fig. 4. Relationships for Vertical-to-horizontal Ground Acceleration (Elnashai and Papazoglou)

A question arises with regard to recommended damping ratio s for vertical spectra as well as response modification factors. Whereas the latter is discussed briefly in subsequent sections, the former requires considerable effort. It is not known what level of damping is appropriate to represent hysteretic and other sources of force reduction in vertical vibration modes. For conservatism, it is prudent to recommend low values, pending further research, hence 1% of critical is recommended for the time being.

4. Behaviour or Response Modification Factors

As postulated by equations 8 through 11, the behaviour factor q is used to scale the elastic forces to arrive at a set of design forces. EC8 gives q factors for RC and steel structures which vary from 1 (plastic limit design) to a maximum of 8. For RC structures, the behaviour factor is defined for various structural systems, such as moment resisting frames, coupled wall-frame and core-frame structures and assumes a constant value regardless of the specific details of the structure within its class. For steel, a method based on the structural overstrength (defined as the ratio of the load multiplies at yield and at ultimate) is employed. The overstrength term, through, is fixed to 1.2, with an exceptional allowance up to 1.6 in special cases.

For all systems, and indeed in all other seismic codes, the behaviour factor is period-independent. Many studies have highlighted the inadequacy of this approach as well as the concept of a behaviour factor solely linked to the ductility of the structure. Fajfar (1994.a) studied the relationship between an overstrength factor (q_s), a ductility-related behaviour factor (q_μ) and the final behaviour factor q, and recommended the use of the former two to arrive at the latter. In another study, Fajfar (1994.b) indicated that the use of a period-independent q factor may be a reasonable approximation, but it obscures the sources of force reduction under seismic loading. Various energy measures were also used to assess recommended values of response modification factors.

To highlight the period dependence of the ductility-based behaviour factor, Figure 5 is examined, where the ratio between ordinates of the elastic and inelastic spectra for the JMA Kobe record (Elnashai et al, 1995) are plotted versus period.

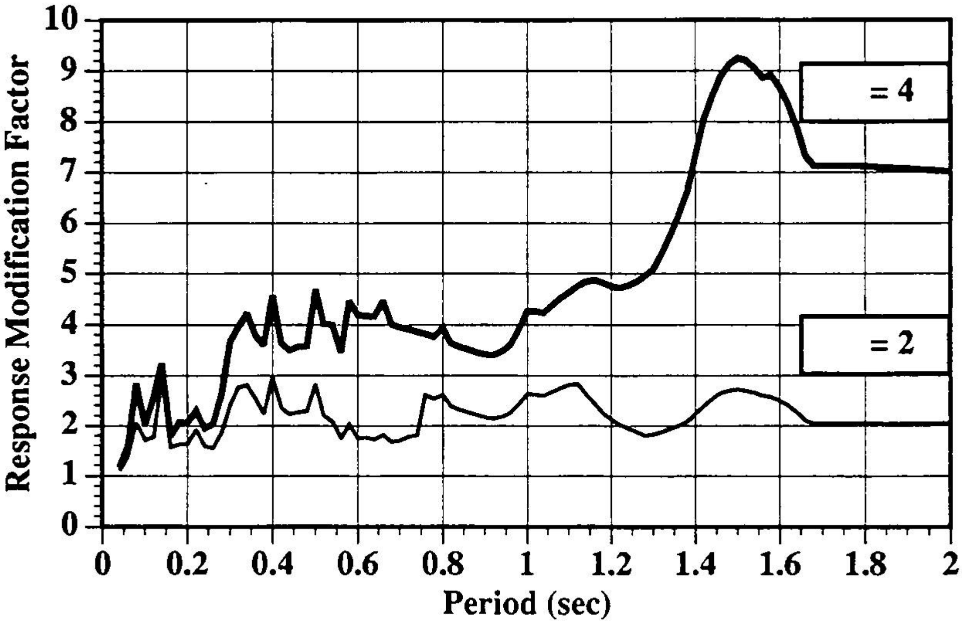

Fig. 5. Behaviour Factors for JMA Kobe Record; Ductility Factors 2 and 4

Several interesting observations emanate from Figure 5. Firstly, it is clear that in the very short period range, inelastic response does not greatly affect the force level generated within a structural system. Hence, it is reasonable to assume a ductility-related behaviour factor of unity in this range. Secondly, for the intermediate range, the behaviour factor increases monotonically, up to a point when it becomes constant and independent of period. Finally, the relationship between μ and q is less uniform for higher ductility (in Figure 5 μ of 4).

In the light of the above discussion, adopting an approach whereby the overall behaviour factor is an aggregate of two constituents; ductility-related and overstrength-related, would render the concepts underlying seismic design more transparent to the user.

5. Conclusions

The EC8 approach to elastic seismic force calculation in the horizontal direction is adequate and represents actual observations, whilst the vertical component is not adequately described. It is therefore recommended to utilise a ratio of vertical-to-horizontal ground acceleration which is dependent on magnitude and distance, instead of the constant factor used currently.

With regard to behaviour factors, Eurocode 8 recommends values independent of the period of vibration. Alternative approaches have been recommended in the literature. It is considered that regardless of whether these approaches lead to different values of q, their adoption leads to a more transparent procedure that is of greater value to the end-user.

Many issues remain subject to ongoing development and improvement. For instance, the dynamic amplification factors given in the recent National Earthquake Hazard Reduction Program (NEHRP) are worthy of consideration.

It is hoped that the ongoing ENV period and extensive studies on various parts of EC8 currently underway will result in improvements that will reinforce the position of the code as an international reference document.

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