EN 1990 Expert Group: Recommendations for the evolution of EN 1990

Annex C Chapters 5 to 7 (23 January 2013)

Note: The original text of Annex C given in the 3rd column is in blue colour, original text of Section 6 is in green colour.

Clause	EN 1990:2002 + A1:2004 incorporating corrigenda December 2008 and April 2010	Recommendations for the evolution of EN 1990 and notice of future possible changes to Clause	Background for recommendation.
C1 Scope and field of applications	(1) This annex provides information and theoretical background to the partial factor method described in Section 6 and annex A. This Annex also provides the background to annex D, and is relevant to the contents of annex B. (2) This annex also provides information on – the structural reliability methods; – the application of the reliability-based method to determine by calibration design values and/or partial factors in the design expressions – the design verification formats in the Eurocodes.	NOTE: The majority of structures can be designed according to the suite of Eurocodes EN 1990 to EN1999 without any need for the application of the material presented in this annex. Application may however be considered useful for design situations that are not well covered and for possible extensions of the code.	Further guidance may be found in ISO 2394, JCSS Probabilistic Model Code and JCSS Risk Assessment in Engineering - Principles, System Representation & Risk Criteria.
C2 Symbols		Added new symbols: P_{ft} target failure probability β_t target reliability index Deleted: Prob(.) Probability	
C4 Overview of reliability methods	(3) In both the Level II and Level III methods the measure of reliability should be identified with the survival probability $P_s = (1 - P_f)$, where P_f is the failure probability for the considered failure mode and within an appropriate reference period. If the calculated failure probability is larger than a pre-set target value P_0 then the structure	(3) In both the Level II and Level III methods the measure of reliability should be identified with the survival probability $P_s = (1 - P_f)$, where P_f is the failure probability for the considered failure mode and within an appropriate reference period. If the calculated failure probability is larger than a pre-set target value P_{ft} then the structure	

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	should be considered to be unsafe.	should be considered to be unsafe.	
C.5 Reliability index β	(1) In the Level II procedures, an alternative measure of reliability is conventionally defined by the reliability index β which is related to P_f by: $P_f = \Phi(-\beta)$ (C.1) where Φ is the cumulative distribution function of the standardised Normal distribution. The relation between P_f and β is given in Table C1. **Table C1 - Relation between β and P_f** P_f: 10^{-1}, 10^{-2}, 10^{-3}, 10^{-4}, 10^{-5}, 10^{-6}, 10^{-7} β: 1,28, 2,32, 3,09, 3,72, 4,27, 4,75, 5,20 (2) The probability of failure P_f can be expressed through a performance function g such that a structure is considered to survive if $g > 0$ and to fail if $g \leq 0$: $P_f = \text{Prob}(g \leq 0)$ (C.2a) If R is the resistance and E the effect of actions, the performance function g is : $g = R - E$ (C.2b) with R, E and g random variables. (3) If g is Normally distributed, β is taken as : $\beta = \frac{\mu_g}{\sigma_g}$ (C.2c) where :	**C.5 Probability of failure and reliability index β** **C.5.1 Uncertainty modelling** (1) Fundamentally, the calculation of the probability of failure shall take basis in all available knowledge, and the uncertainty representation shall include all relevant causal and stochastic dependencies as well as temporal and spatial variability. The appropriate choice of method for the calculation of the failure probability depends on the characteristics of the problem at hand, and especially on whether the problem can be considered as being time-invariant and whether the problem concerns individual failure modes or systems. **C.5.2 Time-invariant reliability problems** (1) In case the problem does not depend on time (or spatial characteristics), or may be transformed such that it does not, e.g. by use of extreme value considerations, three types of methods may in general be used to compute the failure probability P_f, namely: a) FORM/SORM (First/Second Order Reliability Methods) b) Simulation techniques, e.g. crude Monte Carlo simulation, importance sampling, asymptotic sampling, subset simulation and adaptive sampling c) Numerical integration. (2) In the FORM the probability of failure P_f is related to the reliability index β by $P_f = \Phi(-\beta)$ (C.1)	

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μ_g is the mean value of g, and
σ_g is its standard deviation,

so that :

$$\mu_g - \beta\sigma_g = 0 \qquad \text{(C.2d)}$$
and

$$P_f = \text{Prob}(g \leq 0) = \text{Prob}(g \leq \mu_g - \beta\sigma_g) \qquad \text{(C.2e)}$$
For other distributions of g, β is only a conventional measure of the reliability

$P_s = (1 - P_f)$.

</td><td>

where Φ is the cumulative distribution function of the standardised Normal distribution. The relation between P_f and β is given in Table C1.

Table C1 - Relation between β and P_f
<table>
<tr><td>P_f</td><td>10^{-1}</td><td>10^{-2}</td><td>10^{-3}</td><td>10^{-4}</td><td>10^{-5}</td><td>10^{-6}</td><td>10^{-7}</td></tr>
<tr><td>β</td><td>1,28</td><td>2,32</td><td>3,09</td><td>3,72</td><td>4,27</td><td>4,75</td><td>5,20</td></tr>
</table>
(3) The probability of failure P_f can be expressed through a performance function g such that a structure is considered to survive if $g > 0$ and to fail if $g \leq 0$:

$$P_f = P(g \leq 0) \qquad \text{(C.2a)}$$
(4) If R is the resistance and E the effect of actions, the limit state equation or performance function g is:

$$g = R - E \qquad \text{(C.2b)}$$
with R and E statistically independent random variables.

NOTE: In case of dependency between the load effect and the resistance, as e.g. often may be the case in geotechnical design, the procedure should be applied to other independent basic variables.

(5) If R and E are Normally distributed, β is obtained as:

$$\beta = \frac{\mu_R - \mu_E}{\sqrt{\sigma_R^2 + \sigma_E^2}} \qquad \text{(C.2c)}$$
where:

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		μ_R, μ_E are mean values of *R* and *E* σ_R, σ_E are standard deviations of *R* and *E* (6) For other formulations of the limit state equation or non-Normal distributions the reliability index can be determined by an iterative procedure and the probability of failure obtained approximately by (C.1). NOTE: For calculation of the reliability index see ISO 2394 or Probabilistic Model Code of JCSS [xx]. **C.5.3 Time-variant reliability problems** (1) Two classes of time-dependent problems are considered, namely those associated with – failures caused by extreme values, and – failures caused by the accumulation of effects over time. (2) In the case of failure due to extreme values, a single action process may be replaced by a random variable representing the extreme characteristics (minimum or maximum) of the random process over a chosen reference period, typically the life time or one year. If there is more than one stochastic process involved, they should be combined, taking into account the dependencies between the processes. (3) An exact and general expression for the failure probability of a time varying process on a time interval (0,*t*) can be derived from integration of the conditional failure rate $h(\tau)$ according to:	

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		$P_f(0,t) = 1 - \exp\left[-\int_0^t h(\tau)d\tau\right]$ (C.3) (4) The conditional failure rate is defined as the probability that failure occurs in the interval (τ, $\tau + d\tau$), given no failure before time τ. When the failure threshold is high enough it may be assumed that the conditional failure rate $h(\tau)$ can be replaced by the average out-crossing intensity $\nu(\tau)$: $\nu(t) = \lim_{\Delta \to 0} \frac{P(g(X(t)) > 0 \cap g(X(t+\Delta) \leq 0)}{\Delta}$ (C.4) (5) If failure at the start (t = 0) explicitly is considered: $P(0,t) = P_f(0) + [1 - P_f(0)]\,[1 - \exp\, _0^t\, \nu(\tau)\, d\tau]$ (C.5) in which $P_f(0)$ is the probability of structural failure at (t = 0). The mathematical formulation of the out-crossing rate ν depends on the type of loading process, the structural response and the limit state. For practical application the formula (C.5) may need to be extended to include several processes with different fluctuation scales and/or constant in time random variables. (6) In the case of cumulative failures (fatigue, corrosion etc.), the total history of the load up to the point of failure may be of importance. In such cases the time dependency may be accounted for by subdividing the considered time reference period into intervals and to model and calculate the probability of failure as failure of the logical series system comprised by the individual time intervals.	

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<tr><td>C.6 Target values of reliability index β</td><td>(1) Target values for the reliability index β for various design situations, and for reference periods of 1 year and 50 years, are indicated in Table C2. The values of β in Table C2 correspond to levels of safety for reliability class RC2 (see Annex B) structural members.

NOTE 1 For these evaluations of β
– Lognormal or Weibull distributions have usually been used for material and structural resistance parameters and model uncertainties ;
– Normal distributions have usually been used for self-weight
– For simplicity, when considering non-fatigue verifications, Normal distributions have been used for variable actions. Extreme value distributions would be more appropriate.

NOTE 2 When the main uncertainty comes from actions that have statistically independent maxima in each year, the values of β for a different reference period can be calculated using the following expression

$\Phi(\beta_n) = [\Phi(\beta_1)]^n$ (C.3)

where
β_n is the reliability index for a reference period of n years, design situations, and for reference periods of 1 year and 50 years
β_1 is the reliability index for one year.

Table C2 - Target reliability index β for Class RC2 structural members [1)]
<table>
<tr><td>Limit state</td><td colspan="2">Target reliability index</td></tr>
<tr><td>Ultimate</td><td>1 year</td><td>50 years</td></tr>
<tr><td>Fatigue</td><td>4,7</td><td>3,8</td></tr>
<tr><td>Serviceability (irreversible)</td><td></td><td>1,5 to 3,8</td></tr>
<tr><td></td><td>2,9</td><td>1,5</td></tr>
</table></td><td>(1) Decisions with respect to the design, repair, strengthening, maintenance, operation and decommissioning of structures should take basis in risk assessments, whereby it is ensured that benefits are optimized and at the same time that life safety risks are managed in accordance with society preferences.

NOTE Risk assessment should performed in accordance with ISO 13824:2009 Bases for design of structures - general principles on risk assessment of systems involving structures.

(2) Risk based decision making should in principle include all consequences associated with the decisions, including consequences caused by structural failures but also in terms of the benefits achieved from the operation of the structures. The risk related to a decision a is in general defined as $R(a) = \sum_{i=1}^{n_E} P_i C_i$ where n_E is the number of possible events with P_i and C_i being the probability and the consequence associated with event i . The possible events arising out of the decision a should include all direct and indirect consequences for all phases of the life cycle of the structure.

(3) The specified maximum acceptable failure probabilities should be chosen in dependency on the consequence and the nature of failure, the economic losses, the social inconvenience, and the amount of expense and effort required to reduce the probability of failure. If there is no risk of loss of human lives associated with structural failures the target failure probabilities may be selected solely on the basis of an economic optimization. If structural failures are associated with risk of loss of human lives the marginal life saving costs principle applies and</td><td></td></tr>
</table>

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	[1] See Annex B [2] Depends on degree of inspectability, reparability and damage tolerance. (2) The actual frequency of failure is significantly dependent upon human errors which are not considered in partial factor design (See Annex B). Thus β does not necessarily provide an indication of the actual frequency of structural failure.	this may be used through the Life Quality Index. In all cases the acceptable failure probabilities should be calibrated against well-established cases that are known from past experience to have adequate reliability. (4) The specified maximum failure probabilities relevant for ultimate and serviceability limit state design, should reflect the fact that criteria for such limit states do not account for human errors. These probabilities are not directly related to the observed failure rate, which is highly influenced by failures involving some effects of human errors. (5) When dealing with time-dependent structural properties, the effect of the quality control and inspection and repair procedures on the probability of failure should be taken into account. This may lead to adjustments to specified values, conditional upon the results of inspections. Specified failure probabilities should always be considered in relation to the adopted calculation and probabilistic models and the method of assessment of the degree of reliability. (6) Target values for the reliability index β for various design situations, and for reference periods of 1 year and 50 years, are indicated in Table C2. The values of β in Table C2 correspond to levels of safety for reliability class RC2 (see Annex B) structural members. **Table C2 - Target reliability index β for Class RC2 structural members [1]** Limit state | Target reliability index Ultimate | 1 year | 50 years Fatigue | 4,7 | 3,8	

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<tr><td>Serviceability (irreversible)</td><td>2,9 to 4,7</td><td>1,5 to 3,8</td></tr>
<tr><td></td><td>2,9</td><td>1,5</td></tr>
<tr><td colspan="3">1) See Annex B
2) Depends on degree of inspectability, reparability and damage tolerance.</td></tr>
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NOTE 1 For these evaluations of β
– Lognormal or Weibull distributions have usually been used for material and structural resistance parameters and model uncertainties ;
– Normal distribution has usually been used for self-weight
– Three parameter Lognormal distribution or extreme value distribution have usually been used for variable actions.
– Lognormal distribution is often used to model uncertainties related to fatigue loads.
NOTE 2 When the main uncertainty comes from actions that have statistically independent maxima in each year, the values of β for a different reference period can be calculated using the following expression
$$\Phi(\beta_n) = [\Phi(\beta_1)]^n \qquad (C.6)$$
where
β_n is the reliability index for a reference period of n years,
β_1 is the reliability index for a reference period of one year.
(7) The actual frequency of failure is significantly dependent upon human error which is not considered in partial factor design (See Annex B). Thus β does not necessarily provide an indication of the actual frequency of structural failure.
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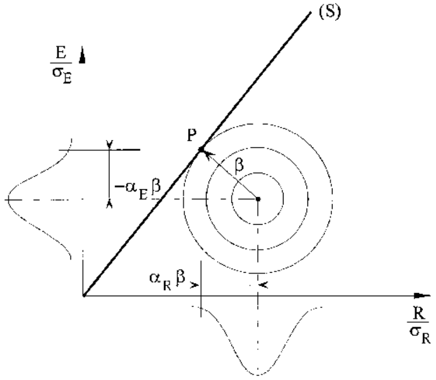
C7 Approach for calibration of design values	 (S) failure boundary $g = R - E = 0$ P design point **Figure C2 - Design point and reliability index β according to the first order reliability method (FORM) for Normally distributed uncorrelated variables).** (2) Design values should be based on the values of the basic variables at the FORM design point, which can be defined as the point on the failure surface (g = 0) closest to the average point in the space of normalised variables (as diagrammatically indicated in Figure C2). (3) The design values of action effects E_d and resistances R_d should be defined such that the probability of having a more unfavourable value is as follows: $P(E > E_d) = \Phi(+\alpha_E \beta)$ (C.6a)	**C7.1 Basis for calibration of design values** (1) The reliability elements including partial factors γ and ψ factors should be calibrated in such a way that the target reliability index β_t is best achieved. The calibration procedure (see Fig. C.2) follows several steps: a. Selection of a set of reference structures b. Selection of a set of reliability elements (e.g. partial factors, ψ factors) c. Designing the structures according to the selected set of reliability elements d. Calculation the reliability indices for the designed structures e. Calculation the difference $D = \sum w_i (\beta_i - \beta_t)^2$ (w_i is the weight factor i) f. Repeating steps (b) to (f) for getting minimum value of difference D NOTE: The choice of the target value of reliability index β_t should be based on optimisation procedure. Different values of reliability index β_t may be needed for different failure modes. (2) The set of partial factors and ψ factors that leads to the lowest value of D is the desired set. More detail procedure how to provide this optimisation is described in several sources (e.g. in ISO 2394). The probabilistic models for loads and resistances of the JCSS Probabilistic Model Code [xx] may be used.	Need for explanation of basis of calibration of reliability elements is based on requests of users.

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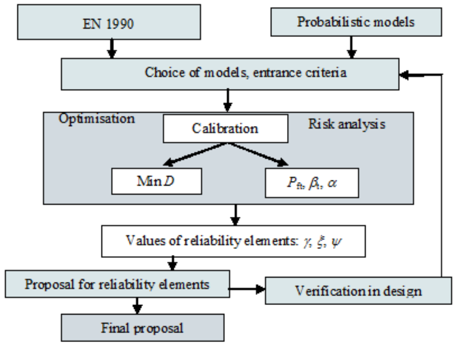
	$P(R \leq R_d) = \Phi(-\alpha_R \beta)$ (C.6b) where β_n is the target reliability index (see C6) α_E and α_R, with $\|\alpha\| \leq 1$, are the values of the FORM sensitivity factors. The value of α is negative for unfavourable actions and action effects, and positive for resistances. α_E and α_R may be taken as - 0,7 and 0,8, respectively, provided $0{,}16 < \sigma_E/\sigma_R < 7{,}6$ (C.7) where σ_E and σ_R are the standard deviations of the action effect and resistance, respectively, in expressions (C.6a) and (C.6b). This gives $P(E > E_d) = \Phi(-0{,}7\beta)$ (C.8a) $P(R \leq R_d) = \Phi(-0{,}8\beta)$ (C.8b) (4) Where condition (C.7) is not satisfied $\alpha = \pm 1{,}0$ should be used for the variable with the larger standard deviation, and $\alpha = \pm 0{,}4$ for the variable with the smaller standard deviation where σ_E and σ_R are the standard deviation. (5) When the action model contains several basic variables, expression (C.8a) should be used for the leading variable only. For the accompanying actions the design values may be defined by $P(E > E_d) = \Phi(-0{,}4 \times 0{,}7 \times \beta) = \Phi(-0{,}28\beta)$ (C.9) NOTE For $\beta = 3{,}8$ the values defined by expression (C.9) correspond approximately to the 0,90 fractile.	 **Figure C2 Illustration of a calibration procedure of reliability elements.** **C7.2 The design value method** (1) The design value method is directly linked to the basic principle of EN 1990 according to which it should be verified that no limit state is exceeded when the design values of all basic variables are used in the models of structural resistance R and action effects E. A design of a structure is considered to be sufficient if the limit states are not reached when the design values are introduced into the models. In symbolic notation this is expressed as $E_d < R_d$ (C.7) where the design values of action effect E_d and resistance R_d are given as $E_d = E\{F_{d1}, F_{d2}, \ldots a_{d1}, a_{d2}, .. \theta_{d1}, \theta_{d2, \ldots}\}$ (C.8a) $R_d = R\{X_{d1}, X_{d2}, \ldots a_{d1}, a_{d2}, .. \theta_{d1}, \theta_{d2, \ldots}\}$ (C.8b)	

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(6) The expressions provided in Table C3 should be used for deriving the design values of variables with the given probability distribution.

Table C3 – Design values for various distribution functions

Distribution	Design values
Normal	$\mu - \alpha\beta\sigma$
Lognormal	$\mu\, exp(-\alpha\beta V)$ for $V = \sigma/\mu < 0{,}2$
Gumbel	$u - \frac{1}{a} ln\{-ln\, \Phi(-\alpha\beta)\}$
	where $u = \mu - \frac{0{,}577}{a};a = \frac{\pi}{\sigma\sqrt{6}}$

NOTE In these expressions μ, σ and V are, respectively, the mean value, the standard deviation and the coefficient of variation of a given variable. For variable actions, these should be based on the same reference period as for β.

(7) One method of obtaining the relevant partial factor is to divide the design value of a variable action by its representative or characteristic value.

where

F_d is the design value of action

X_d is the design value of resistance property

a_d is the design value of geometrical property

θ_d is the design value of model uncertainty.

(2) For some particular limit states (e.g. fatigue) a more general formulation may be necessary to express a limit state.

(3) If only two basic variables E and R are considered then the design values of action effects E_d and resistances R_d should be defined such that the probability of having a more unfavourable value is as follows

$$F_E(e_d) = \Phi(+\alpha_E\beta_t) \qquad (C.9a)$$

$$F_R(r_d) = \Phi(-\alpha_E\beta_t) \qquad (C.9b)$$

where

Φ is the cumulative distribution function of the standardised Normal distribution

β_t is the target reliability index with reference period T (see C6)

α_E and α_R, with $|\alpha| \leq 1$, are the values of the FORM sensitivity factors for action and for resistance. The value of α is negative for unfavourable actions and action effects, and positive for resistances.

(4) In common cases the coefficients of sensitivity for leading unfavourable actions and action effects α_E = -0,7 and α_E = -0,28 for accompanying unfavourable actions may be taken and the coefficient of sensitivity for resistance α_R = 0,8 provided that the ratio between standard deviations of the load effect σ_E and resistance σ_R is in a range

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$0{,}16 < \sigma_E/\sigma_R < 7{,}6$ (C.10)

NOTE 1 Where condition (C.10) is not satisfied, $\alpha = \pm 1{,}0$ should be used for the variable with the larger standard deviation, and $\alpha = \pm 0{,}4$ for the variable with the smaller standard deviation.

NOTE 2 For α_E = -0,28 the values defined by expression (C.9) correspond approximately to the 0,90 fractile.

(5) The design value F_d of the action and resistance R_d may be expressed from (C.9) as

$F_d(\beta_t) = F_F^{-1}[\Phi(-\alpha_E \beta_t)]$ (C.11a)
$R_d(\beta_t) = F_R^{-1}[\Phi(+\alpha_R \beta_t)]$ (C.11b)

where $F(.)^{-1}$ is an inverse cumulative distribution function.

(6) The expressions provided in Table C3 should be used for deriving the design values of variables with the given probability distribution.

Table C3 – Design values for various distribution functions

<table>
<tr><th>Distribution</th><th>Design values</th></tr>
<tr><td>Normal</td><td>$\mu - \alpha\beta\sigma$</td></tr>
<tr><td>Lognormal</td><td>$\mu \exp(-\alpha\beta V)$ for $V = \sigma/\mu < 0{,}2$</td></tr>
<tr><td>Gumbel</td><td>$u - \frac{1}{a}\ln\{-\ln \Phi(-\alpha\beta)\}$
where $u = \mu - \frac{0{,}577}{a}; a = \frac{\pi}{\sigma\sqrt{6}}$</td></tr>
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<tr>
<td>Weibull</td>
<td>$x_{sup} - \sqrt[-c_1]{c_2}\,\sqrt[-c_1]{-\ln(\Phi^{-1}(-\beta))}$
where $x_{sup} = \mu + u_p\,\sigma$
$u_p = \dfrac{\Gamma(\frac{c_1+1}{c_1}) - \sqrt[c_1]{-\ln(\Phi^{-1}(-\beta))}}{\sqrt{\Gamma(\frac{c_1+2}{c_1}) - \Gamma^2(\frac{c_1+1}{c_1})}}$</td>
</tr>
</table>
NOTE In these expressions μ, σ, V and a are, respectively, the mean value, the standard deviation, the coefficient of variation and the skewness of a given variable. For variable actions, these should be based on the same reference period as for β.
(7) One method of obtaining the relevant partial factor is to divide the design value of a variable action by its representative or characteristic value.
C7.3 Material partial factors
(1) The resistance model is assumed to be obtained by the following general model, see Annex D:
$R = b\,\delta\,\mathrm{R}(X, a)$ (C.12)
where
$\mathrm{R}(X, a)$ is the resistance model as defined in a relevant materials standard
X is strength (and stiffness) parameter(s). Each of the strength parameters is modelled as a Lognormal stochastic variable with coefficient of variation V_X.
a is the geometrical parameter(s)
δ is the model uncertainty related to resistance model (can be determined using the method in the Annex D ‘Design assisted by testing’). δ is modelled as a Lognormal stochastic variable with mean value 1 and coefficient of variation V_δ
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		b is bias in resistance model (can be determined using the method in the Annex D 'Design assisted by testing'). (2) The design value of the resistance R_d can be determined by different models, see Cl. 6.3.5. (3) **Model 1** where design values are determined for the material strength parameters $R_d = \frac{\mathrm{R}(X_{\mathrm{d}}, a_{\mathrm{d}})}{\gamma_{\Delta}}$ (C.13) where a_{d} is the design value for geometrical data. X_{d} is the design value for strength parameters γ_{Δ} is the partial factor related to the model uncertainty for the resistance model – including possible uncertainty related to transformation from laboratory to real structure and bias in resistance model. If more than one strength parameter is used in the resistance model, then design values are applied for each strength parameter in (4). (4) The design value of a strength parameter(s) X_{d} is determined by $X_{\mathrm{d}} = \eta \frac{X_{\mathrm{k}}}{\gamma_{\mathrm{m}}}$ (C.14) where η is the conversion factor taking into account load	

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		duration effects, moisture, temperature, scale effects, etc. X_k is the characteristic value of strength parameter generally defined by the 5% fractile γ_m is the partial factor for strength parameter depending on the coefficient of variation V_X, see Table C4. NOTE If the resistance model is linear in the strength parameters then $R_d = R(X_d, a_d)$ and X_d for each of the strength parameters is obtained using a partial factor $\gamma_M = \gamma_m \gamma_\Delta$. (5) **Model 2** where a characteristic resistance is obtained using characteristic values of the material strength parameters $$R_d = \frac{R(\eta X_k, a_k)}{\gamma_M} \quad (C.15)$$ where γ_M is the partial factor related to uncertainty of the strength parameters X through the resistance function R(X,a), V_R. (6) **Model 3** where a characteristic resistance is estimated based on tests $$R_d = \frac{R_k}{\gamma_M} \quad (C.16)$$ where R_k is the characteristic resistance estimated based on tests, see the Annex D 'Design assisted by testing'. R_k is generally defined by the 5% fractile	

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		γ_M is the partial factor related to uncertainty of the resistance obtained based on tests, V_R . (7) In model 1 the partial factor γ_m depends on the uncertainty of the strength parameter(s) and γ_Δ depends on the uncertainty of the resistance model, incl. bias $$\gamma_\Delta = \frac{\gamma_\delta}{b} \quad \text{(C.17)}$$ where γ_δ is partial factor depending on the model uncertainty with coefficient of variation V_δ , see Table C5. (8) In model 2 the total uncertainty of the resistance depends on the model uncertainty δ and the uncertainty related to the strength parameters X though the resistance function $R(X,a)$. The material partial factors are correspondingly obtained from $$\gamma_M = \frac{\gamma_\delta \gamma_R}{b} \quad \text{(C.18)}$$ where γ_R is partial factor depending on the resistance uncertainty with coefficient of variation V_R . Coefficient V_R depends on the uncertainties of the strength parameters though the resistance function $R(X,a)$, see Table C4 γ_δ is partial factor depending on the model uncertainty with coefficient of variation V_δ . (9) In model 3 the partial factor γ_M depends on the uncertainty of the test results including statistical	

<table>
<tr><th>Clause</th><th>EN 1990:2002 + A1:2004 incorporating corrigenda December 2008 and April 2010</th><th>Recommendations for the evolution of EN 1990 and notice of future possible changes to Clause</th><th>Background for recommendation.</th></tr>
<tr><td></td><td></td><td>
uncertainty

$\gamma_{\mathrm{M}} = \gamma_R$ (C.19)

where
γ_R is partial factor depending on the resistance uncertainty with coefficient of variation V_R. Coefficient V_R depends on the uncertainties of the resistance obtained based on tests, see Table C4.

(10) The material partial factors in Tables C4 and C5 should be calibrated such that failure probabilities for the relevant failure modes are close to the target reliability level in Table C5.

(11) The material partial factors for ultimate limit states in the persistent and transient design situations should be in accordance with Tables C4 and C5.

NOTE 1 The values in Tables C4 and C5 can be altered e.g. for different reliability levels in the National annex.

NOTE 2 The partial factors in Tables C4 and C5 are calibrated without taking into account the bias b and with the characteristic value for the model uncertainty equal to 1.

Table C4 γ_{m}, γ_R - partial safety factor for strength parameter or resistance.
<table>
<tr><td>Coefficient of variation for strength parameter in model 1, V_X or resistance in model 2 and 3, V_R</td><td>$\leq$ 5 %</td><td>10 %</td><td>15 %</td><td>20 %</td><td>25 %</td></tr>
<tr><td>γ_m in model 1 or γ_R</td><td></td><td></td><td></td><td></td><td></td></tr>
</table>
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in model 2 and 3					

Table C5 γ_δ - partial safety factor for model uncertainty.

Coefficient of variation for model uncertainty for resistance model in model 1, V_δ	$\leq$ 5 %	10 %	15 %	20 %	25 %
γ_δ					

C7.4 Partial factors of actions

(1) The partial factors of actions may be determined using the design value method. For a specific load case where material properties are not to be considered, the design values of the effects of actions E_d (exp. (6.2) in EN 1990) may be expressed as:

$$E_d = \gamma_{Sd} E\{\gamma_{F,i} F_{rep,i} ; a_d\} \quad i \geq 1 \qquad (C.20)$$

where

a_d is the design value of the geometrical data

γ_{Sd} is a factor for model uncertainties in modelling the effects of actions or in particular cases, in modelling the actions.

(2) The design effects of actions may be commonly simplified for the design of common structures (exp. (6.2a, 6.2b) in EN 1990):

$$E_d = E\{\gamma_{F,i} F_{rep,i} ; a_d\} \quad i \geq 1 \qquad (C.21)$$

where

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		$\gamma_{F,i} = \gamma_{Sd} \times \gamma_{f,i}$ (C. 22) NOTE Further guidance is given for non-linear structural analyses. (3) The partial factor of action F is based on the ratio between the design value F_d and the characteristic value F_k of an action given as $\gamma_F = F_d / F_k$ (C.23) **C7.4.1 Partial factors of permanent actions** (1) Characteristic value of a permanent action G_k may be commonly considered as a mean value (see EN 1991-1-1) based on nominal values of geometry and mean densities, therefore $G_k = \mu_G$. (2) In case that the variability of permanent action is greater than 5 %, or it is important to take into account this variability, it should be considered by 5% lower and 95% upper fractiles. NOTE Normal distribution for permanent actions may be commonly applied. The lower and upper fractiles of the permanent action may be specified as $G_{k,inf} = \mu_G - 1{,}64\ \sigma_G = \mu_G\ (1 - 1{,}64\ V_G)$ $G_{k,inf} = \mu_G + 1{,}64\ \sigma_G = \mu_G\ (1 + 1{,}64\ V_G)$ where V_G is the coefficient of variation μ_G is the mean σ_G is the standard deviation.	

Clause	EN 1990:2002 + A1:2004 incorporating corrigenda December 2008 and April 2010	Recommendations for the evolution of EN 1990 and notice of future possible changes to Clause	Background for recommendation.
		(3) The design value of the permanent action G_d may be determined as $G_d = \mu_G - \alpha_G \beta \sigma_G = \mu_G (1 + 0{,}7\beta V_G)$ (C.24) (4) The partial factor for self-weight γ_g is given as the ratio between the design and characteristic values $\gamma_g = G_d / G_k = \mu_G (1 - \alpha_G \beta V_G) / \mu_G = 1 - \alpha_G \beta V_G$ (C.25) where V_G is the coefficient of variation of permanent action. In common cases the coefficient of variation of self-weight of a structure (e.g. concrete, steel) may be assumed to be from 3 to 5 %. For other permanent actions the coefficient of variation is commonly higher, up to 10 %. Example: In case that the coefficient of variation V_G = 0,05 is assumed for self-weight of a structure and the self-weight is a leading action (expressions (6.10) or (6.10a)) in the fundamental combination of actions in EN 1990), then for the coefficient of sensitivity $\alpha_G = -0{,}7$ and the target value of reliability index $\beta_t = 3{,}8$, the partial factor is determined as $\gamma_g = 1 - \alpha_G \beta V_G = 1 + 0{,}7 \times 3{,}8 \times 0{,}05 \approx 1{,}15$ If the self-weight is a non-dominant action ($\alpha_G = -0{,}28$), see expression (6.10b), the partial factor can be determined as $\gamma_g = 1 + 0{,}28 \times 3{,}8 \times 0{,}05 = 1{,}05$ It should be noted that the coefficient γ_{sd} for model	

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		uncertainties should also be taken into account which is commonly in a range from 1,05 to 1,15. In case that the coefficient for model uncertainties γ_{sd} = 1,1 is considered then the partial factor γ_G for a leading permanent action is given as $\gamma_G = 1{,}15 \times 1{,}1 \approx 1{,}27$ and for an accompanying permanent action $\gamma_G = 1{,}05 \times 1{,}1 \approx 1{,}16$ **C7.4.2 Partial factors for variable actions** (1) Similar procedure may be applied for estimation of partial factors for variable actions *Q*. Commonly lognormal distribution, Gamma or extreme value distribution may be apply for modelling of variable actions including climatic actions. (2) The characteristic values of a climatic actions (wind, snow, icing, temperature) are specified according to EN 1990 in a way that the annual probability of their exceeding should be 0,02 (mean return period of 50 years). NOTE In some cases, e.g. in phases of transient design situation and depending on the character of loading it may be more suitable to use other probability *p* or other return period (see e.g. EN 1991-1-6 for transient design situations and shorter periods of execution). (3) In case that the Gumbel distribution should be applied (which is recommended in some Parts of EN 1991), then the *p*-fractile of a climatic action *Q* for a certain reference period is given as $Q_p = \mu_Q \{1 - V_Q [0{,}45 - 0{,}78 \ln N + 0{,}78 \ln(-\ln p)]\}$ (C.26)	

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		where V_Q denotes the coefficient of variation of climatic action for the basic period (e.g. 1 year) and N is the number of basic periods during the reference period (often the assumed working life of a structure, e.g. 100 years for a bridge). (4) The characteristic value of a climatic action (e.g for p = 0,98 in the basic reference period) may be determined as $Q_k = \mu_Q \{1 - V_Q [0,45 + 0,78 \ln(-\ln 0,98)]\}$ (C.27) and the design value of action $Q_d = \mu_Q\{1 - V_Q [0,45 - 0,78 \ln N + 0,78 \ln(-\ln(\Phi^{-1}(-\alpha_E \beta))]\}$ (C.28) where Φ is the standard Normal distribution function β is the reliability index corresponding to the reference period α_E is the FORM coefficient of sensitivity being 0,7 for dominant and 0,28 for non-dominant loads N is the number of basic periods in the reference period (e.g. N = 100 if the design life time is 100 years and the basic period 1 year). Note that sometimes p is chosen dependently on the design life time. (5) The partial factor of a climatic action is based on the expressions (C.29) and (C.30) $\gamma_q = \dfrac{1 - V_Q(0,45 - 0,78 \ln N + 0,78 \ln(-\ln(\Phi^{-1}(-\alpha_E \beta)))}{1 - V_Q(0,45 + 0,78 \ln(-\ln 0,98))}$	

Clause	EN 1990:2002 + A1:2004 incorporating corrigenda December 2008 and April 2010	Recommendations for the evolution of EN 1990 and notice of future possible changes to Clause	Background for recommendation.
		(C.29) under the assumption of a Gumbel distribution. NOTE 1 In some cases other probabilistic distributions may be more suitable, e. g. Weibull or three parameter lognormal distributions. NOTE 2 Direct application of the three parameter or Lognormal or extreme value probabilistic distributions for specification of partial factors for climatic actions (e.g. snow, wind) commonly leads to greater values of partial factors than recommended in Eurocodes. However, commonly a hidden safety may be found based on several factors (see e.g. the Background document to EN 1990).	
		C7.5 Calibration of partial factors for fatigue (1) The SN-approach is used together with the Miner's rule for linear fatigue accumulation. NOTE Fatigue failure of welded details is considered in this clause. The same principles can be used for fatigue failure of other fatigue critical details. (2) For linear SN-curves the number of cycles, *N* to failure with constant stress range, $\Delta\sigma$ is: $N(\Delta\sigma) = \left(\frac{\Delta\sigma}{\Delta\sigma_C}\right)^{-m} 2\cdot 10^6 = K\cdot\Delta\sigma^{-m}$ (C.30) where $\Delta\sigma_C$ is the characteristic fatigue strength defined as the 5% quantile m is the slope of SN-curve (Wöhler exponent) K is the SN-curve parameter	

Clause	EN 1990:2002 + A1:2004 incorporating corrigenda December 2008 and April 2010	Recommendations for the evolution of EN 1990 and notice of future possible changes to Clause	Background for recommendation.
		(3) For variable amplitude fatigue loading the design value of the Miner's sum should fulfil: $$\sum_i \frac{n_i}{2\cdot 10^6}\left(\frac{\gamma_{Ff}\Delta\sigma_i}{\Delta\sigma_C/\gamma_{Mf}}\right)^m \leq 1 \qquad (C.31)$$ where γ_{Mf} is the partial factor for fatigue strength γ_{Ff} is the partial factor for fatigue load n_i is the number of cycles with fatigue stress range $\Delta\sigma_i$ (4) For non-linear SN-curves the design value of the Miner's sum should fulfil: $$\sum_i \frac{n_i}{N(\gamma_{Mf}\gamma_{Ff}\Delta\sigma_i)} \leq 1 \qquad (C.32)$$ (5) The partial factor for fatigue strength γ_{Mf} is obtained from: $$\gamma_{Mf} = \lambda_{Mf}\cdot\gamma_{M0f} \qquad (C.33)$$ where γ_{M0f} is the partial factor for fatigue strength depending on uncertainties related to the SN-curve and the Miner's rule λ_{Mf} is the factor accounting for bias and other fatigue strength uncertainties not included in γ_{M0f}, such as scales and temperature effects. (6) The partial factor for fatigue load γ_{Ff} is obtained from:	

Clause	EN 1990:2002 + A1:2004 incorporating corrigenda December 2008 and April 2010	Recommendations for the evolution of EN 1990 and notice of future possible changes to Clause	Background for recommendation.
		$\gamma_{Ff} = \lambda_{Ff} \cdot \gamma_{F0f}$ (C.34) where γ_{M0f} is the partial factor for fatigue stress depending on uncertainties related to fatigue load and stress assessment λ_{Mf} is the factor accounting for bias and other fatigue stress uncertainties not included in γ_{F0f} such as different load spectra. (7) The partial factors γ_{M0f} and γ_{F0f} in Tables C5 and C6 are calibrated such that failure probabilities for the relevant failure modes are close to the target reliability level in Table C2. The partial factor γ_{M0f} depends on the coefficient of variations V_{logK} for the fatigue strength parameter, log$K$ and V_{Δ} for the Miner's sum. The partial factor γ_{F0f} depends on the coefficient of variation, V_{Ff} for the fatigue load and stress. NOTE 1 The values in Tables C5 and C6 can be altered *e.g.* for different reliability levels in the National annex. NOTE 2 The values in Tables C5 and C6 can be altered depending on consequences of failure and the associated target reliability. NOTE 3 The values in Tables C5 and C6 can be altered if inspections are performed depending on the reliability of the inspection method using a POD (Probability Of Detection) curve and a fracture mechanics approach to fatigue crack growth. NOTE 4 The fatigue strength parameter, log$K$ can be assumed Normal distributed with V_{logK} depending on the actual SN-curve. The Miner sum can be assumed Lognormal distributed with $V_{\Delta} \approx 0$ for constant amplitude loading and $V_{\Delta} \approx 0,3$ for variable	

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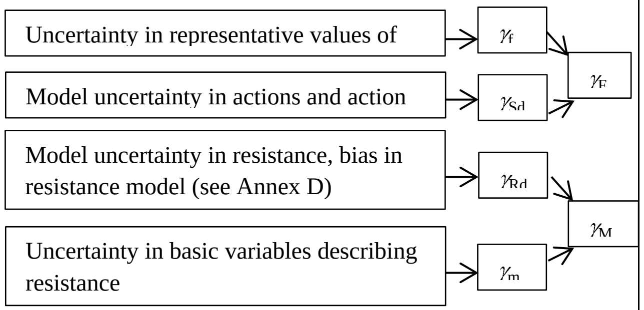
		amplitude loading. The uncertainty for the fatigue stress ranges can be assumed Lognormal distributed with a factor representing uncertainty for the fatigue load and a factor representing uncertainty for the calculation of stress ranges given fatigue loading. The coefficient of variation for uncertainty related to fatigue loading from e.g. rotating machines can be assumed ≈ 0 whereas for fatigue loading from e.g. wind induced vortex shedding it can be assumed ≈ 0,3.	

Table C6. γ_{M0f} **- partial factor for fatigue strength.**

Coefficient of variation, $V_{\log K}$ for fatigue strength parameter, log$K$	≤ 10 %	20 %	30 %
γ_{M0f} for V_{Δ} = 0 %			
γ_{M0f} for V_{Δ} = 30 %			

Table C7. γ_{F0f} **- partial factor for fatigue stress.**

Coefficient of variation, V_{Ff} for fatigue stress	≤5 %	10 %	15 %	20 %	25 %	30 %
γ_{F0f}						

Clause	EN 1990:2002 + A1:2004 incorporating corrigenda December 2008 and April 2010	Recommendations for the evolution of EN 1990 and notice of future possible changes to Clause	Background for recommendation.
C9 Partial factors in EN 1990		 Figure C3 – Relation between individual partial factors	
C10 ψ_0 factors	Expression for general distribution in Table C4 for ψ_0 for the case of two variable actions $\frac{F_s^{-1}\{\Phi(0,4\beta')^{N_1}\}}{F_s^{-1}\{\Phi(0,7\beta)^{N_1}\}}$	Expression in Table C8 for ψ_0 for the case of two variable actions $\frac{F_s^{-1}\{\Phi(0,4\beta')^{N_1}\}}{F_s^{-1}\{\Phi(0,7\beta')^{N_1}\}}$	