Structural Eurocodes

RUB

EN 1990 - Basis of Structural Design

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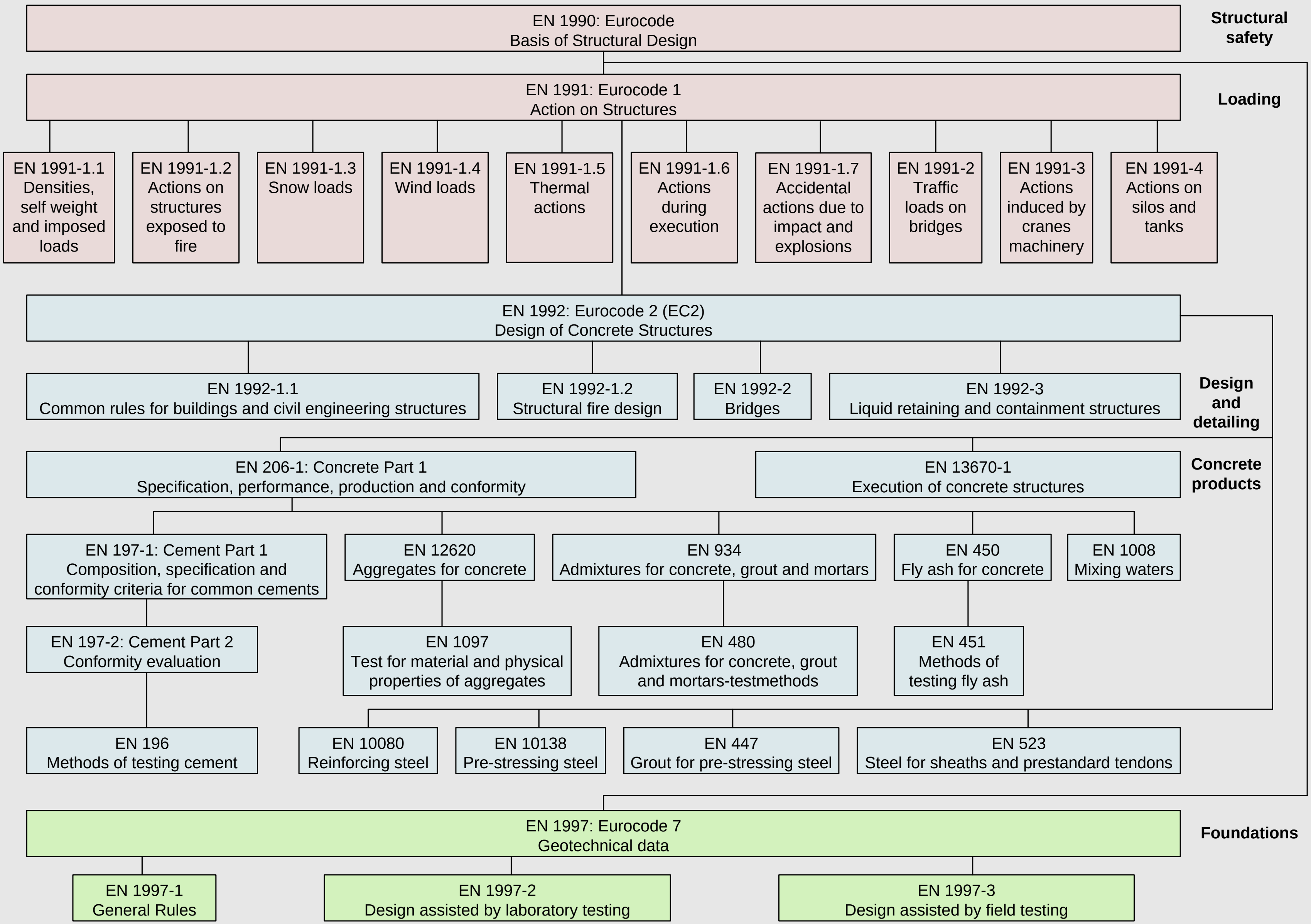
Structural Eurocodes



Eurocode 1: Actions on Structures

EN 1991-1-1	General actions – Desities, self-weight, imposed loads for buildings : 2002 + Correction AC:2009
EN 1991-1-2	General actions – Actions due to fire : 2003
EN 1991-1-3	General actions – Snow loads : 2003 + AC:2009
EN 1991-1-4	General actions – Wind actions : 2005(E) Amendment A1:2010 Correction AC:2010
EN 1991-1-5	General actions – Thermal actions : 2003 Correction AC:2009
EN 1991-1-6	General actions, Actions during execution : 2005 Correction AC:2008
EN 1991-1-7	General actions – Accidental actions : 2006 Correction AC:2010
EN 1991-2	Traffic loads on Bridges : 2004
EN 1991-3	Actions induced by cranes and machinery : 2006
EN 1991-4	General actions – Silos and tanks : 2006

Objectives of EN 1990

EN 1990 describes the Principles and requirements for safety, serviceability and durability of structures. It is based on the limit state concept used in conjunction with a partial factor method.

Overview

Risks in Civil Engineering

Measures of Reliability in a Probabilistic Concept

Reliability Verification in EN 1990

Partial Factor Concept in EN 1990

Limit States

- **Ultimate Limit State – ULS**
- **Serviceability Limit State - SLS**

Conclusions

Actions on Structures – EN 1991

Risks in Civil Engineering

Storm

Earthquake

Fire

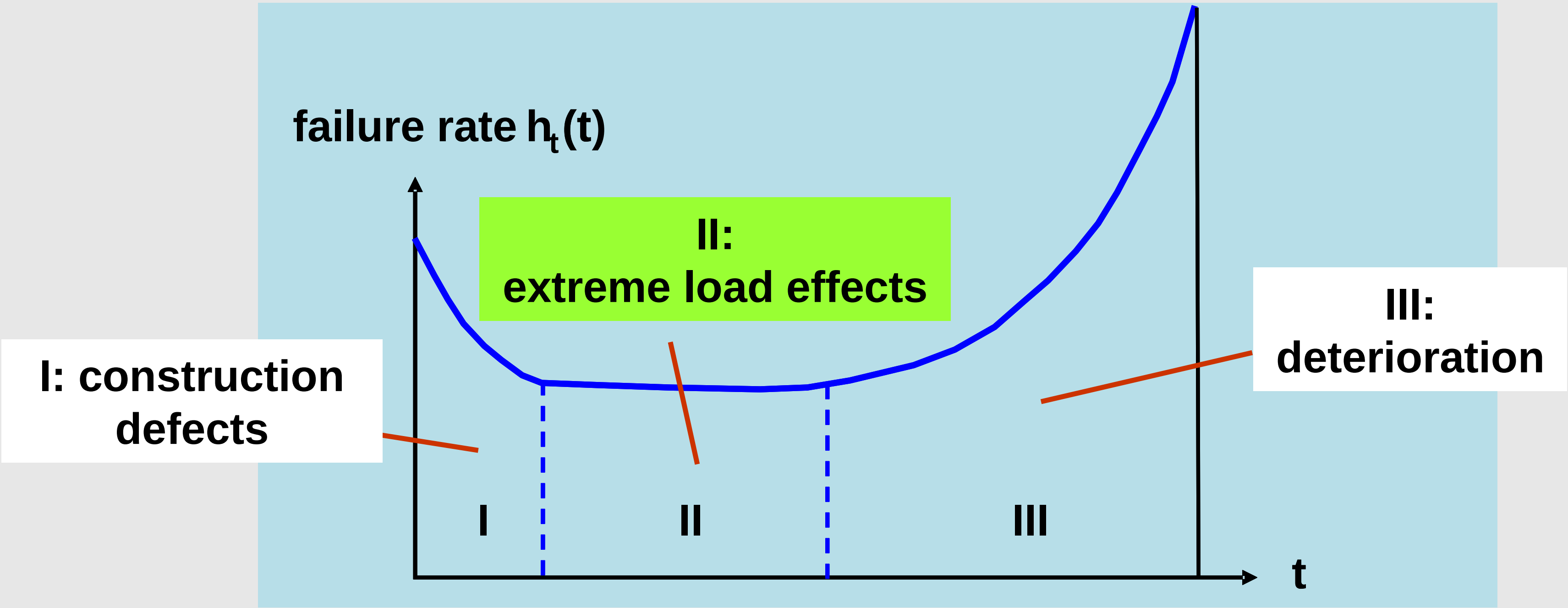
Water

Risks in Civil Engineering

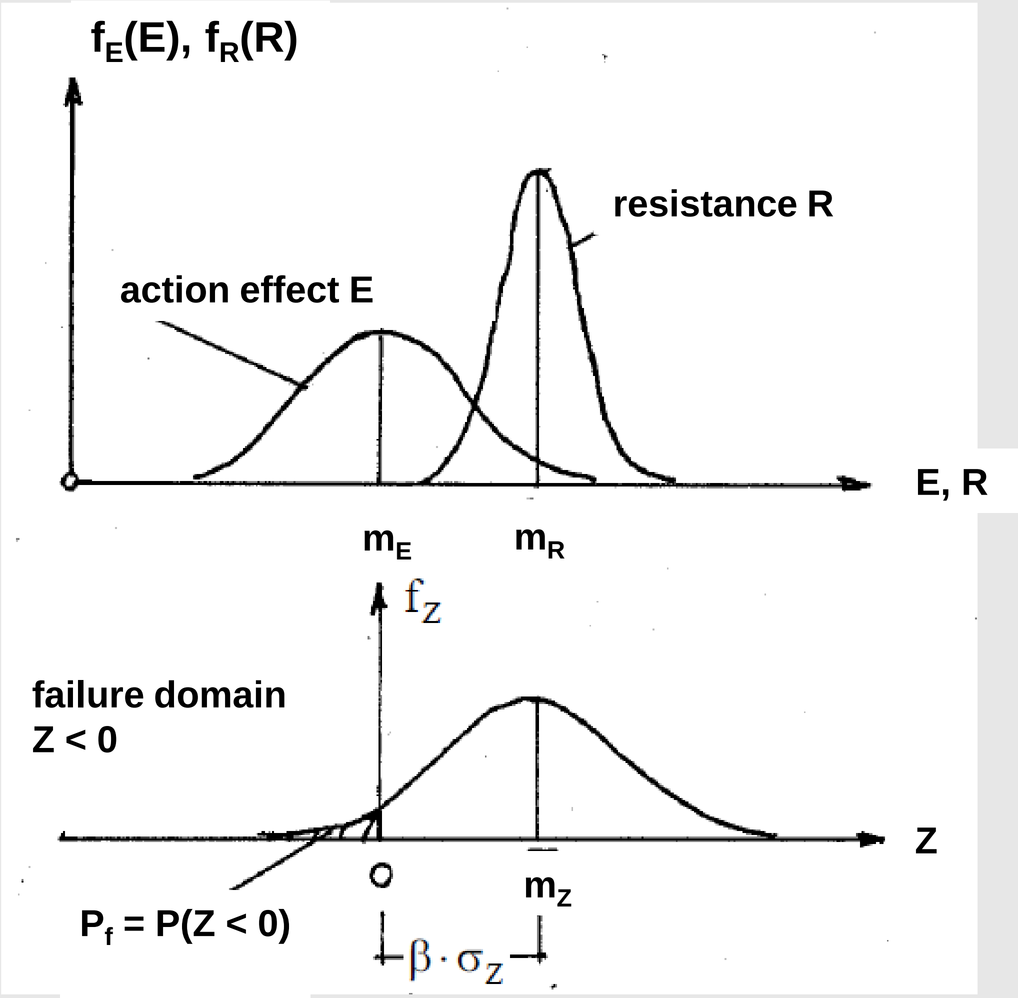
Failure Rate:

$$h_t(t) = \frac{P(T_L \leq t + \Delta t \mid T_L > t)}{\Delta t}$$

T_L – design life, t - time



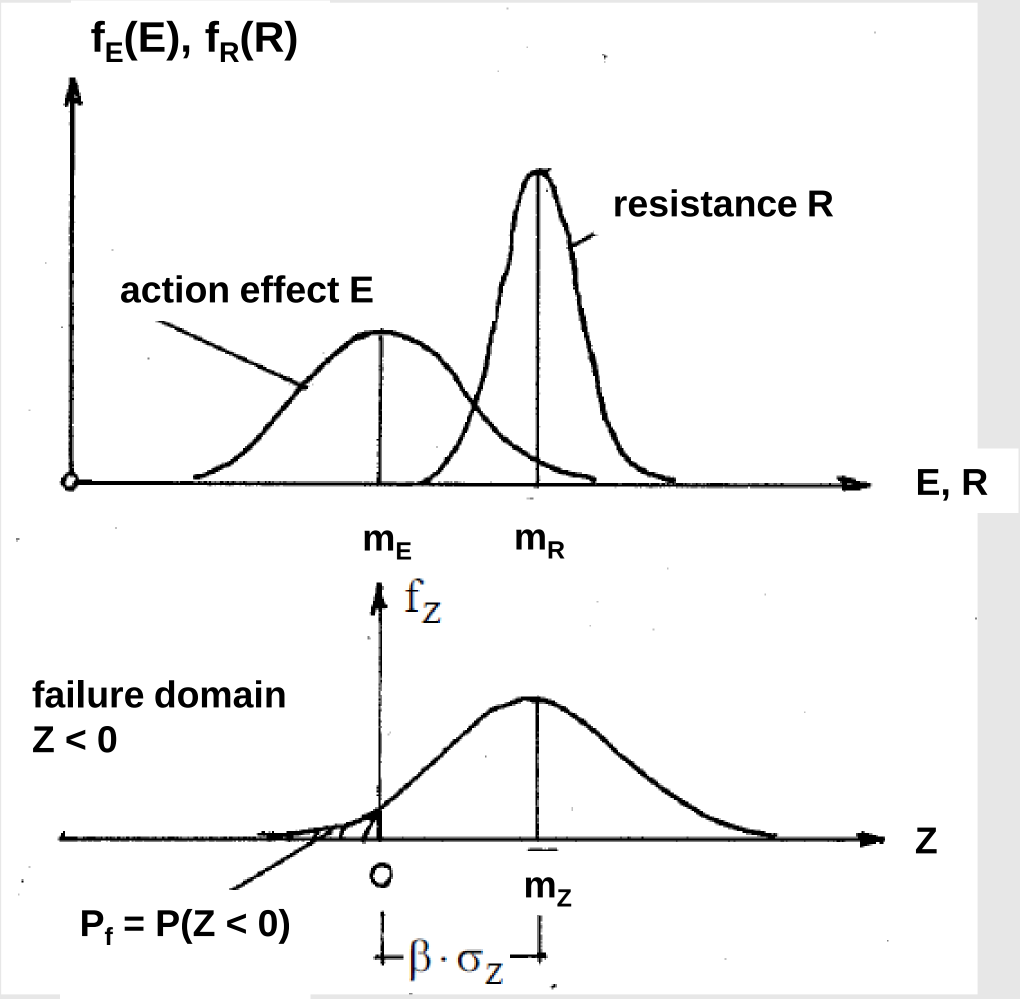
Measures of Reliability



safety zone: $Z = R - E$
with mean m_Z, stand. dev. σ_Z

survival: $Z > 0$; failure: $Z \leq 0$

Measures of Reliability



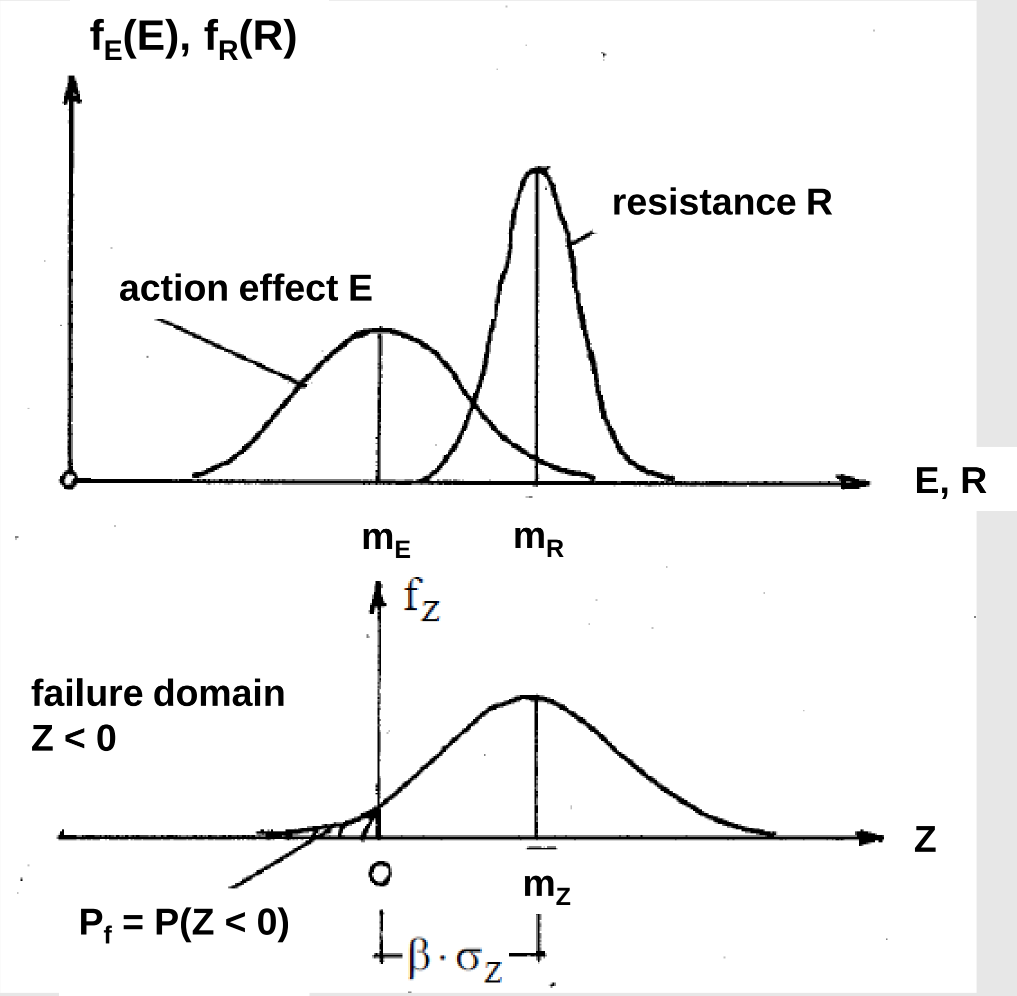
safety zone: $Z = R - E$
with mean m_Z, stand. dev. σ_Z

survival: $Z > 0$; **failure:** $Z \leq 0$

probability of failure P_f

$$P_f = P(Z \leq 0)$$

Measures of Reliability



A reliability index of $\beta = 0$ corresponds to a failure probability of $P_f = 0.5$!

safety zone: $Z = R - E$
with mean m_Z, stand. dev. σ_Z

survival: $Z > 0$; **failure:** $Z \leq 0$

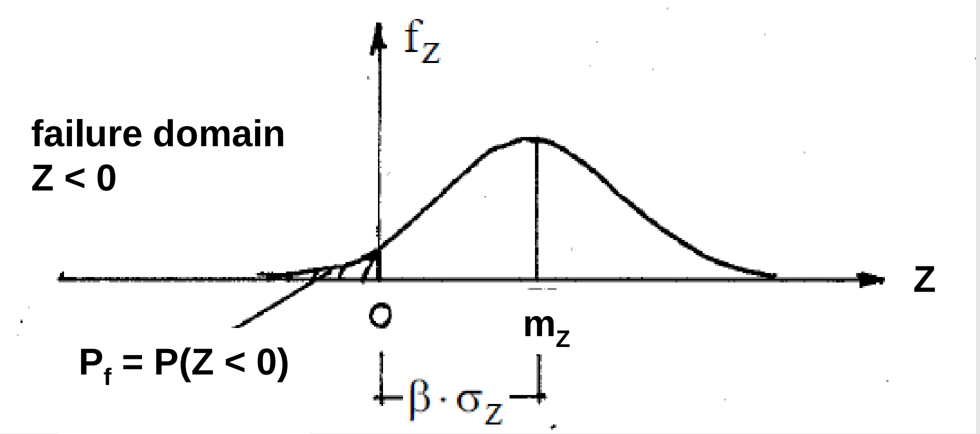
probability of failure P_f

$$P_f = P(Z \leq 0)$$

reliability index β

$$m_Z = \beta \cdot \sigma_Z$$

Measures of Reliability



The reliability index β is applied in EN 1990 for reliability verifications. It is related to the probability of failure, P_f, by

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

where Φ is the cumulative probability function of the Gaussian distribution

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

Reliability Verification in EN 1990

Design values of Action Effect, E_d and Resistance, R_d

Given are: target reliability index β

mean resistance m_R
standard deviation σ_R

mean action effect m_E
standard deviation σ_E

design requirement:

$$m_Z = \beta \cdot \sigma_Z$$

Replacing Z by E and R:

$$m_R - m_E = \beta(\alpha_R \cdot \sigma_R - \alpha_E \cdot \sigma_E)$$

where α are linear weight factors with ranges:

$-1 \leq \alpha_E < 0$

$0 \leq \alpha_R \leq 1$

Design verification using design values E_d and R_d:

$$m_R - \beta\alpha_R \cdot \sigma_R = m_E - \beta\alpha_E \cdot \sigma_E$$

$$R_d = m_R - \beta\alpha_R \cdot \sigma_R \geq E_d = m_E - \beta\alpha_E \cdot \sigma_E$$

Reliability Verification in EN 1990

Target values of Reliability

Table C2 - Target reliability index β for Class RC2 structural members [1)]

Limit state	Target reliability index	
	1 year	50 years
Ultimate	4,7	3,8
Fatigue		1,5 to 3,8 [2)]
Serviceability (irreversible)	2,9	1,5

[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 error, 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.

The target values of reliability are operational, indicative numbers

Reliability Verification in EN 1990

Reliability Differentiation in EN 1990

For the purpose of reliability differentiation, consequences classes (CC) may be established by considering the consequences of failure or malfunction of the structure as given in Table B1.

Table B1 - Definition of consequences classes

Consequences Class	Description	Examples of buildings and civil engineering works
CC3	**High** consequence for loss of human life, *or* economic, social or environmental consequences **very great**	Grandstands, public buildings where consequences of failure are high (e.g. a concert hall)
CC2	**Medium** consequence for loss of human life, economic, social or environmental consequences **considerable**	Residential and office buildings, public buildings where consequences of failure are medium (e.g. an office building)
CC1	**Low** consequence for loss of human life, *and* economic, social or environmental consequences **small or negligible**	Agricultural buildings where people do not normally enter (e.g. storage buildings), greenhouses

Reliability Verification in EN 1990

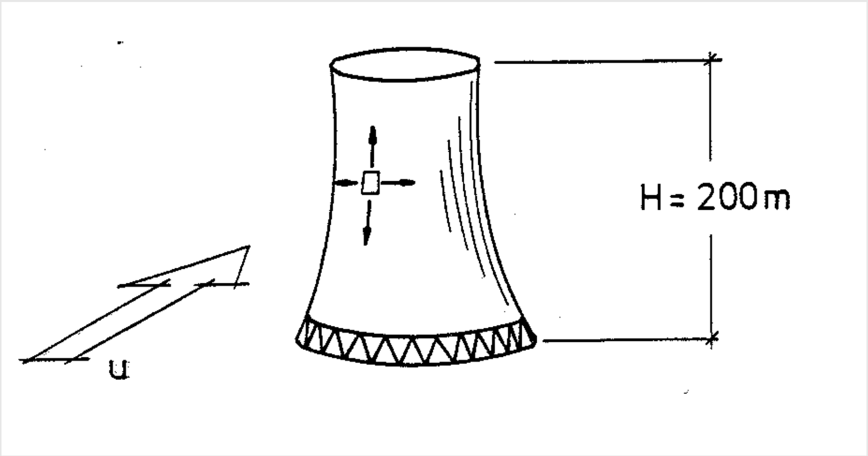
(2) Three reliability classes RC1, RC2 and RC3 may be associated with the three consequences classes CC1, CC2 and CC3.

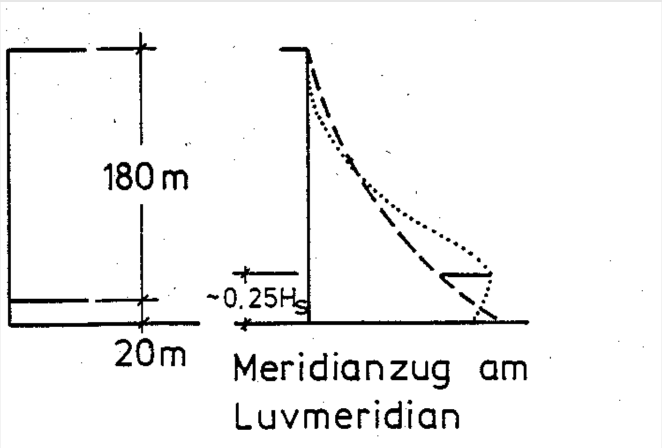
Table B2 - Recommended minimum values for reliability index β (ultimate limit states)

Reliability Class	Minimum values for β	
	1 year reference period	50 years reference period
RC3	5,2	4,3
RC2	4,7	3,8
RC1	4,2	3,3

NOTE A design using EN 1990 with the partial factors given in annex A1 and EN 1991 to EN 1999 is considered generally to lead to a structure with a β value greater than 3,8 for a 50 year reference period. Reliability classes for members of the structure above RC3 are not further considered in this Annex, since these structures each require individual consideration.

Partial Factor Concept in EN 1990





Actions: self weight g, wind load w

Resistance: yield strength of the reinforcement $A_s \cdot \beta_s$

Global safety factor γ_{tot} applied to design the shell for tensile strength:

$$\gamma_{tot} \cdot (n_w - n_g) \leq A_s \cdot \beta_s$$

Self-weight is compressive, it diminishes the tensile wind force:
The shell cannot carry $\gamma_{tot} \cdot w$ when designed with a global factor.

Such a goal would be achieved by the following design equation:

$$\gamma_{tot} \cdot n_w - n_g \leq A_s \cdot \beta_s$$

5.11.1965

In a strong gale, three Cooling Towers at the Ferrybridge Power Station, UK, collapse due to tensile failure of the reinforcement at the windward side

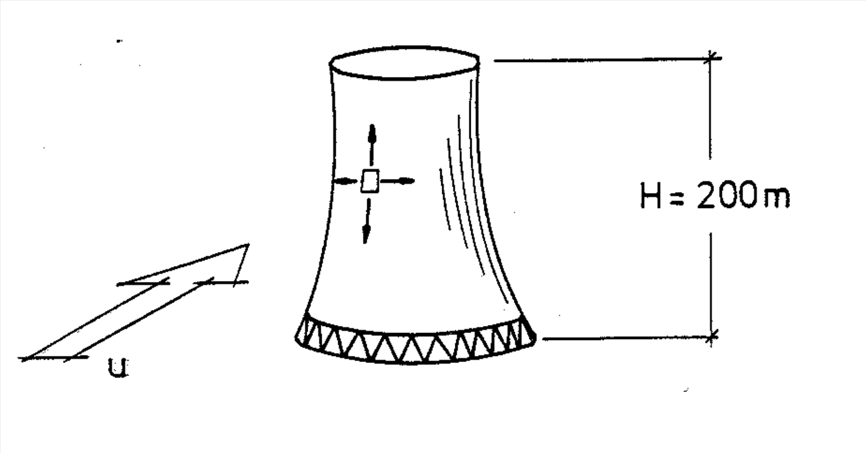
Principal failure causes

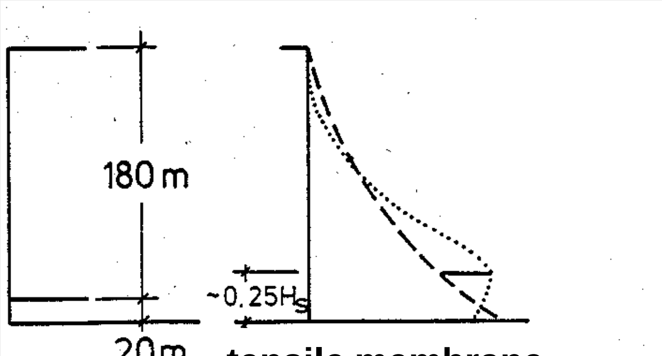
(1) Small shell bending stiffness due to *Single layer reinforcement*, low natural frequencies, increase of resonant response to turbulence;

2) *Load amplification due to flow Interference;*

(3) *Unified safety factor instead of partial concept*

Partial Factor Concept in EN 1990





tensile membrane force n at the windward meridian

Concept of partial safety factors

$$\gamma_W \cdot n_w - \gamma_G \cdot n_g \leq A_s \cdot \beta_s / \gamma_M$$

VGB-BTR 2005: $1{,}6 \cdot n_w - 1{,}0 \cdot n_g \leq A_s \cdot \beta_s / 1{,}15$

The shell is now designed to carry 1,6-times the nominal wind load against 1/1,15 times the nominal tensile strength.

Partial factor Concept in EN 1990

EN 1990 does not apply directly the ***design values*** but utilises the ***partial factor design*** consisting of the following steps:

(1) **Characteristic values** of the basic variables actions F_k, and of the material properties X_k are introduced.

Characteristic values are typically:

- for variable actions Q:	Q_k is the 0,98-quantile of the yearly extremes;
- for permanent actions G:	G_k is the mean value;
- for accidental actions A:	A_d is a nominal value used as design value;
- for strength of materials X:	X_k is the 5%-quantile.

(2) **Design values of actions F** are specified by using partial load factors γ_F:

$$F_d = \gamma_F F_k \quad \text{for a leading action, or}$$
$$F_d = \gamma_F \cdot \psi \cdot F_k \quad \text{for an accompanying action;}$$

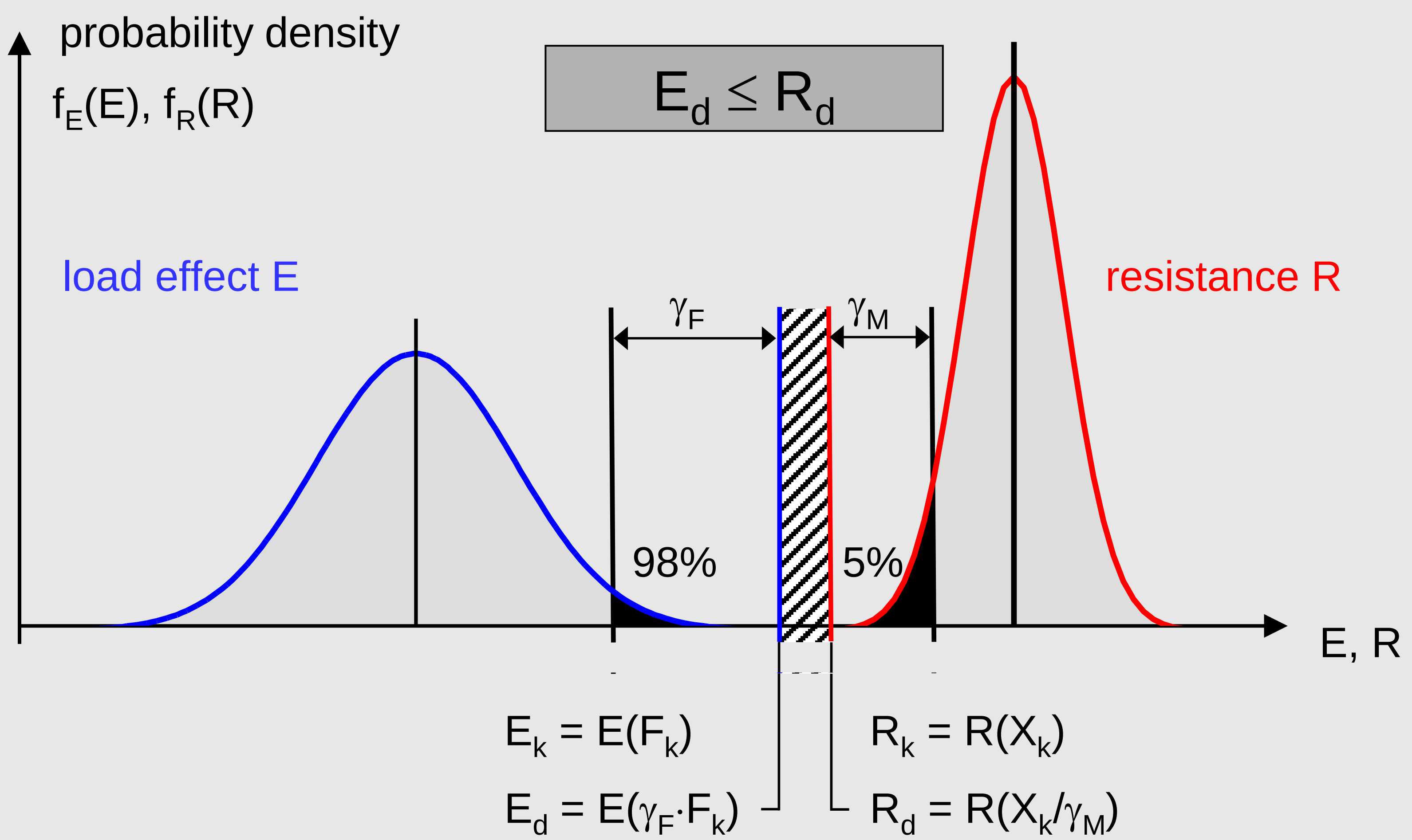
Design values of material properties X are specified by partial material factors γ_m : $X_d = X_k / \gamma_m$

(3) The **design values of action effect and resistance** are calculated as

$$E_d = E\{\gamma_F \cdot F_k; \gamma_F \cdot \psi \cdot F_k\} \leq R_d = R\{X_k/\gamma_m\}$$

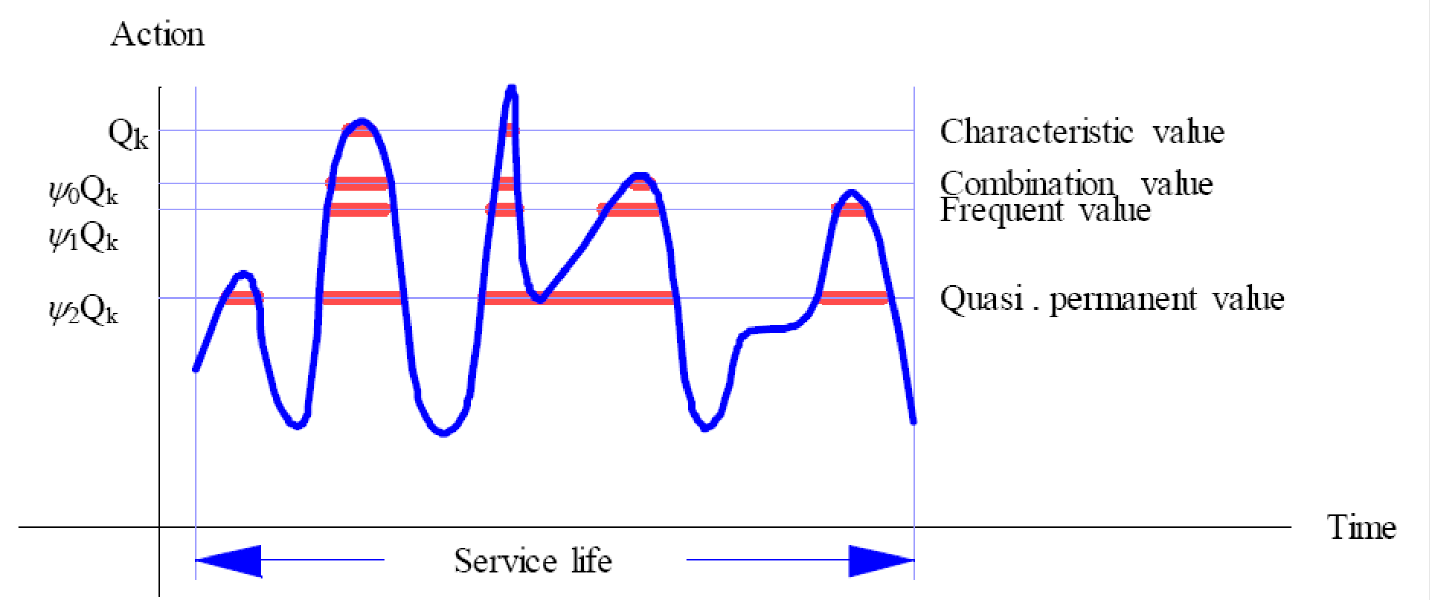
Partial factor Concept in EN 1990

Summary of Verification Procedure



Partial factor Concept in EN 1990

If **several variable actions** have to be considered, the combination of actions consists of the **leading action Q_{k1}** and the **accompanying actions $\psi \cdot Q_{kj}$,** where ψ is the factor for accompanying actions, $\psi \leq 1$



The factor ψ, covers the following situations:

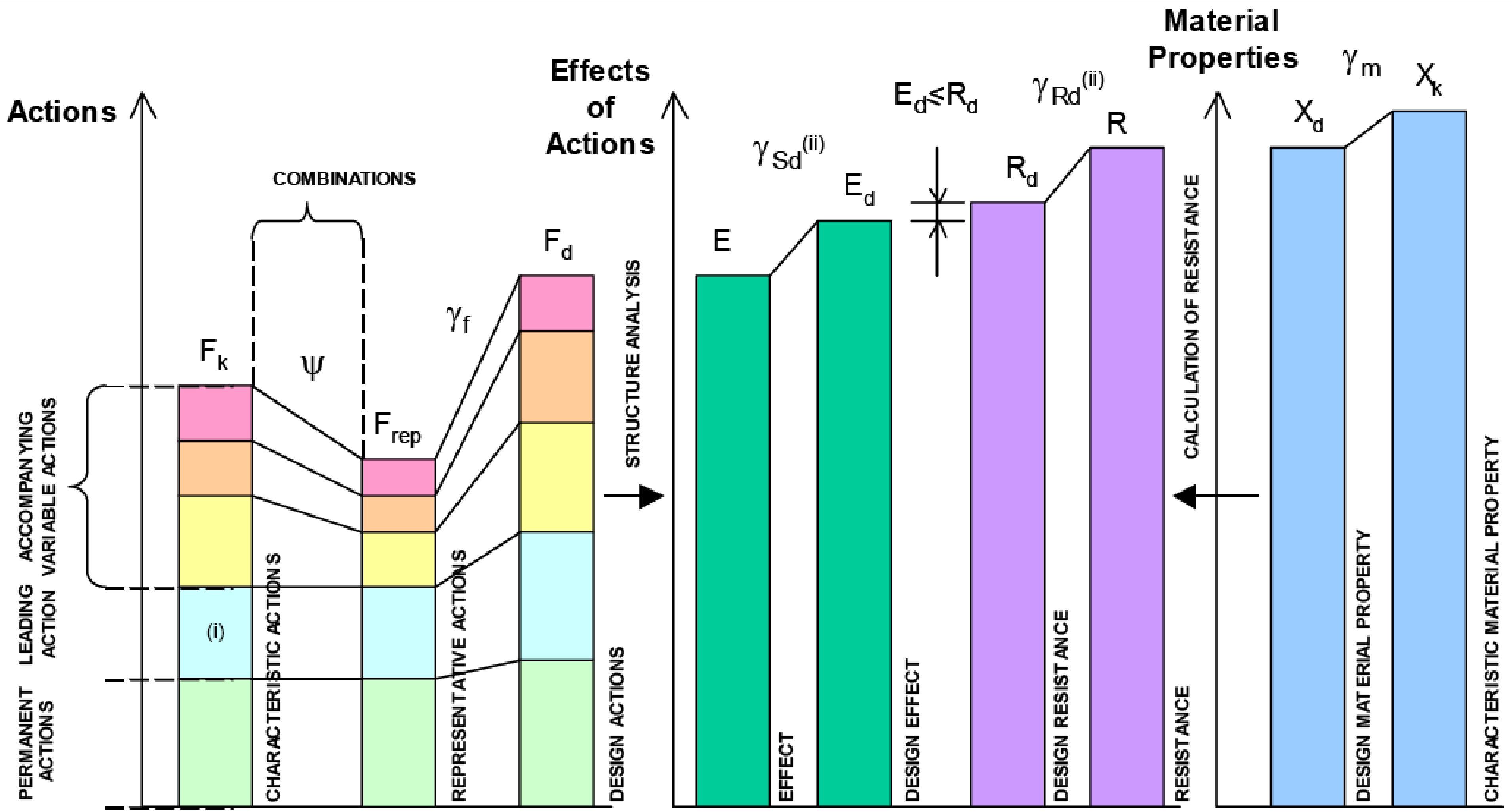
- the combination value of a variable action $\psi_0 \cdot Q_k$
- the frequent value of a variable action $\psi_1 \cdot Q_k$
- the quasi-permanent value of a variable action $\psi_2 \cdot Q_k$

Partial factor Concept in EN 1990

Variable loads and related ψ-factors

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, category (see EN 1991- 1.1)			
Category A: domestic, residential areas	0,7	0,5	0,3
Category B: office areas	0,7	0,5	0,3
Category C: congregation areas	0,7	0,7	0,6
Category D: shopping areas	0,7	0,7	0,6
Category E: storage areas	1,0	0,9	0,8
Category F: traffic area, vehicle weight ≤ 30kN	0,7	0,7	0,6
Category G: traffic area, 30kN < vehicle weight ≤ 160kN	0,7	0,5	0,3
Category H: roofs	0	0	0
Snow loads on buildings (see EN 1991-1-3)			
– Finland, Iceland, Norway, Sweden	0,70	0,50	0,20
– Remainder of CEN Member States, for sites located at altitude H > 1000 m a.s.l.	0,70	0,50	0,20
– Remainder of CEN Member States, for sites located at altitude H ≤ 1000 m a.s.l.	0,50	0,20	0
Wind loads on buildings (see EN 1991-1-4)	0,6	0,2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0,6	0,5	0
Note: The ψ values may set by the National annex.			

Partial factor Concept in EN 1990



Limit States

Ultimate Limit State – ULS

states associated with collapse or with other similar forms of structural failure

(1)P The limit states that concern :
- the safety of people, and/or
- the safety of the structure

shall be classified as ultimate limit states.

Serviceability Limit State – SLS

states that correspond to conditions beyond which specified service requirements for a structure or structural member are no longer met

(1)P The limit states that concern :
- the functioning of the structure or structural members under normal use ;
- the comfort of people ;
- the appearance of the construction works,

shall be classified as serviceability limit states.

Limit States: Ultimate Limit States

ULS in EN 1990

EQU : Loss of static equilibrium of the structure or any part of it considered as a rigid body, where :

- minor variations in the value or the spatial distribution of actions from a single source are significant, and
- the strengths of construction materials or ground are generally not governing ;

STR : Internal failure or excessive deformation of the structure or structural members, including footings, piles, basement walls, etc., where the strength of construction materials of the structure governs ;

GEO : Failure or excessive deformation of the ground where the strengths of soil or rock are significant in providing resistance ;

FAT : Fatigue failure of the structure or structural members.

FAT load combinations are given in the design codes EN1992 – EN1996

Ultimate Limit States: Static Equilibrium

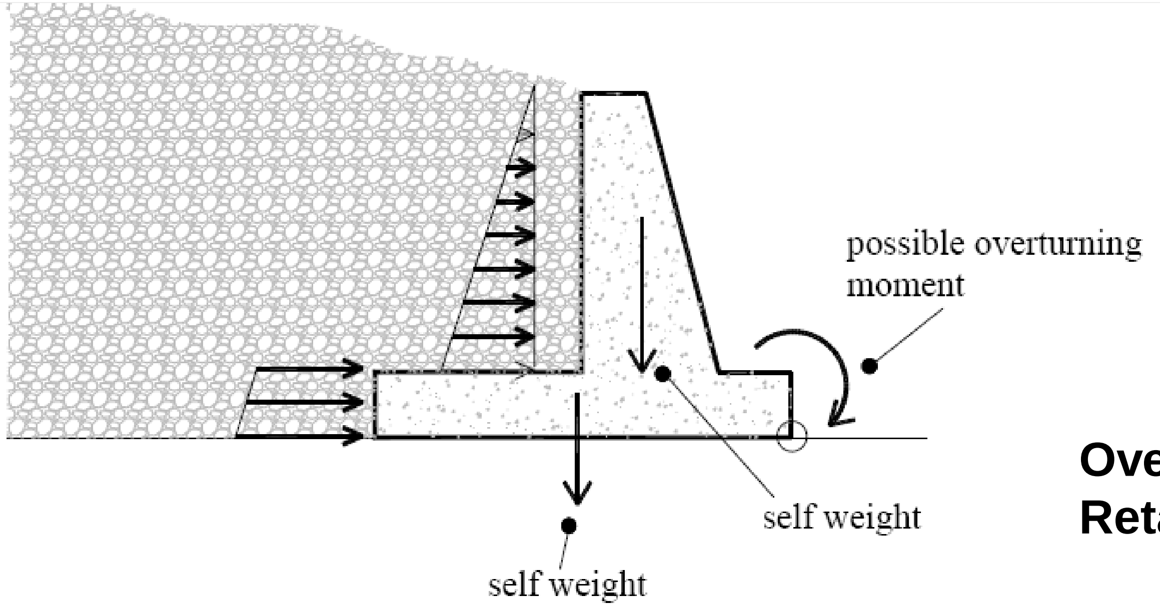
Verifications of static equilibrium

$$E_{d,dst} \leq E_{d,stb}$$

where :

$E_{d,dst}$ is the design value of the effect of destabilising actions ;

$E_{d,stb}$ is the design value of the effect of stabilising actions.



Overturning of a Retaining Wall

Static Equilibrium Limit States

$$E_{d,dst} \leq E_{d,stb}$$

$$\sum_{j\geq1} \gamma_{G,j} G_{k,j} "+" \gamma_P P "+" \gamma_{Q,1} Q_{k,1} "+" \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i}$$

Persistent and transient design situations	Permanent actions		Leading variable action (*)	Accompanying variable actions	
	Unfavourable	Favourable		Main (if any)	Others
(Eq. 6.10)	$\gamma_{Gj,sup} G_{kj,sup}$ **1,10**	$\gamma_{Gj,inf} G_{kj,inf}$ **0,90**	$\gamma_{Q,1} Q_{k,1}$ **1,50 / 0**		$\gamma_{Q,i} \psi_{0,i} Q_{k,i}$ **1,50 / 0**

(*) Variable actions are those considered in Table A1.1

NOTE 1 The γ values may be set by the National annex. The recommended set of values for γ are :

$\gamma_{Gj,sup} = 1,10$

$\gamma_{Gj,inf} = 0,90$

$\gamma_{Q,1} = 1,50$ where unfavourable (0 where favourable)

$\gamma_{Q,i} = 1,50$ where unfavourable (0 where favourable)

Structural Failure Limit States

$$\sum_{j\geq 1} \gamma_{G,j} G_{k,j} "+" \gamma_P P "+" \gamma_{Q,1} Q_{k,1} "+" \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i}$$

Persistent and transient design situations	Permanent actions		Leading variable action	Accompanying variable actions (*)	
	Unfavourable	Favourable		Main (if any)	Others
(Eq. 6.10)	$\gamma_{Gj,sup} G_{kj,sup}$	$\gamma_{Gj,inf} G_{kj,inf}$	$\gamma_{Q,1} Q_{k,1}$		$\gamma_{Q,i} \psi_{0,i} Q_{k,i}$
	1,35	**1,00**	**1,50**		**1,50**

(*) Variable actions are those considered in Table A1.1

Note regarding permanent actions resulting from **one** source:

The partial factor 1,35 applies for all actions originating from **self-weight** if the resulting total effect is unfavourable. Similarly, γ_{inf} = 1.00 is valid if the resulting total effect is favourable. This also applies if different materials are involved.

Structural Failure Limit States

$$\sum_{j\geq 1} \gamma_{G,j} G_{k,j} \text{"+"} \gamma_P P \text{"+"} \gamma_{Q,1} Q_{k,1} \text{"+"} \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \qquad (6.10)$$

or, alternatively for STR and GEO limit states, the less favourable of the two following expressions:

$$\begin{cases} \sum_{j\geq 1} \gamma_{G,j} G_{k,j} \text{"+"} \gamma_P P \text{"+"} \gamma_{Q,1} \psi_{0,1} Q_{k,1} \text{"+"} \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} & (6.10a) \\ \sum_{j\geq 1} \xi_j \gamma_{G,j} G_{k,j} \text{"+"} \gamma_P P \text{"+"} \gamma_{Q,1} Q_{k,1} \text{"+"} \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} & (6.10b) \end{cases}$$

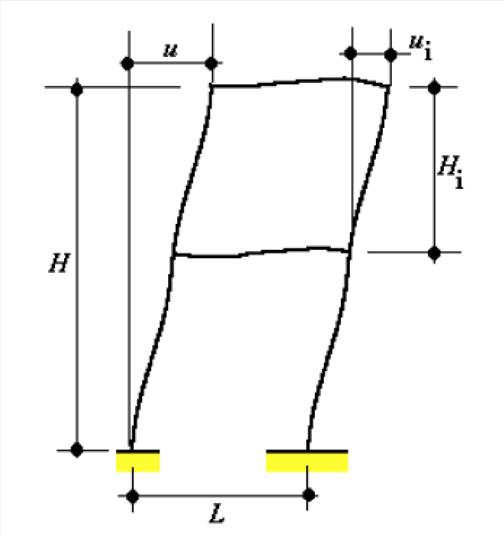
Where :

"+ " implies "to be combined with"
Σ implies "the combined effect of"
ξ is a reduction factor for unfavourable permanent actions G

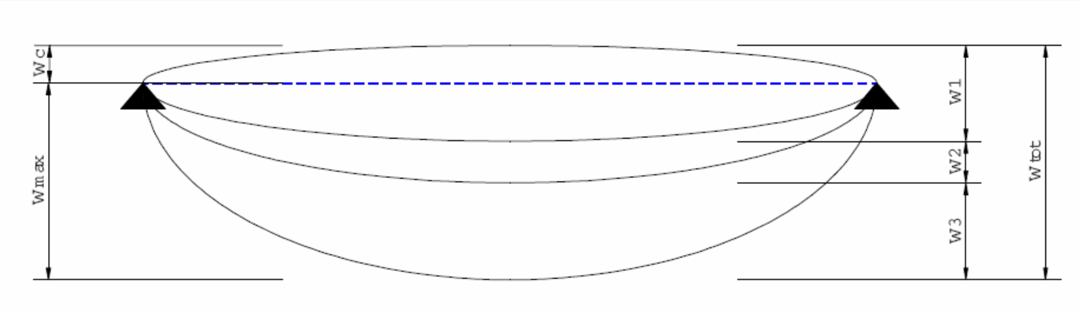
Serviceability Limit States

(1) Serviceability limit states in buildings should take into account criteria related, for example, to floor stiffness, differential floor levels, storey sway or/and building sway and roof stiffness. Stiffness criteria may be expressed in terms of limits for vertical deflections and for vibrations. Sway criteria may be expressed in terms of limits for horizontal displacements.

(2) The serviceability criteria should be specified for each project and agreed with the client.



Horizontal Displacements



Vertical Deflections

Serviceability Limit States

	Irreversible effects of Actions	Reversible effects of Actions	
Serviceability requirements	Characteristic Combination w_{tot} or w_{max}	Frequent Combination w_{max}	Quasi-permanent Combination w_{max}
Function and damage to non-structural members (e.g. partition walls, claddings, etc) (3) • Brittle • Non-brittle **Function** and damage to structural members	 $\leq L/500$ to $L/360$ $\leq L/300$ to $L/200$ $\leq L/300$ to $L/200$		
To avoid ponding of water. Roof covered with waterproof membrane		$\leq L/250$ (4)	
Comfort of user or functioning of machinery		$\leq L/300$	
Crane gantry girders, deflection due to static wheel loads		$\leq L/600$	
Appearance			$\leq L/250$

Design working life

Definition in EN 1990

assumed period for which a structure or part of it is to be used for its intended purpose with anticipated maintenance but without major repair being necessary

expected working life [years]

≥ 150	dams of water reservoirs
≥ 80	bridges
≥ 60	residential and business buildings
30-40	industrial buildings
≈ 10	temporary buildings




Design working life

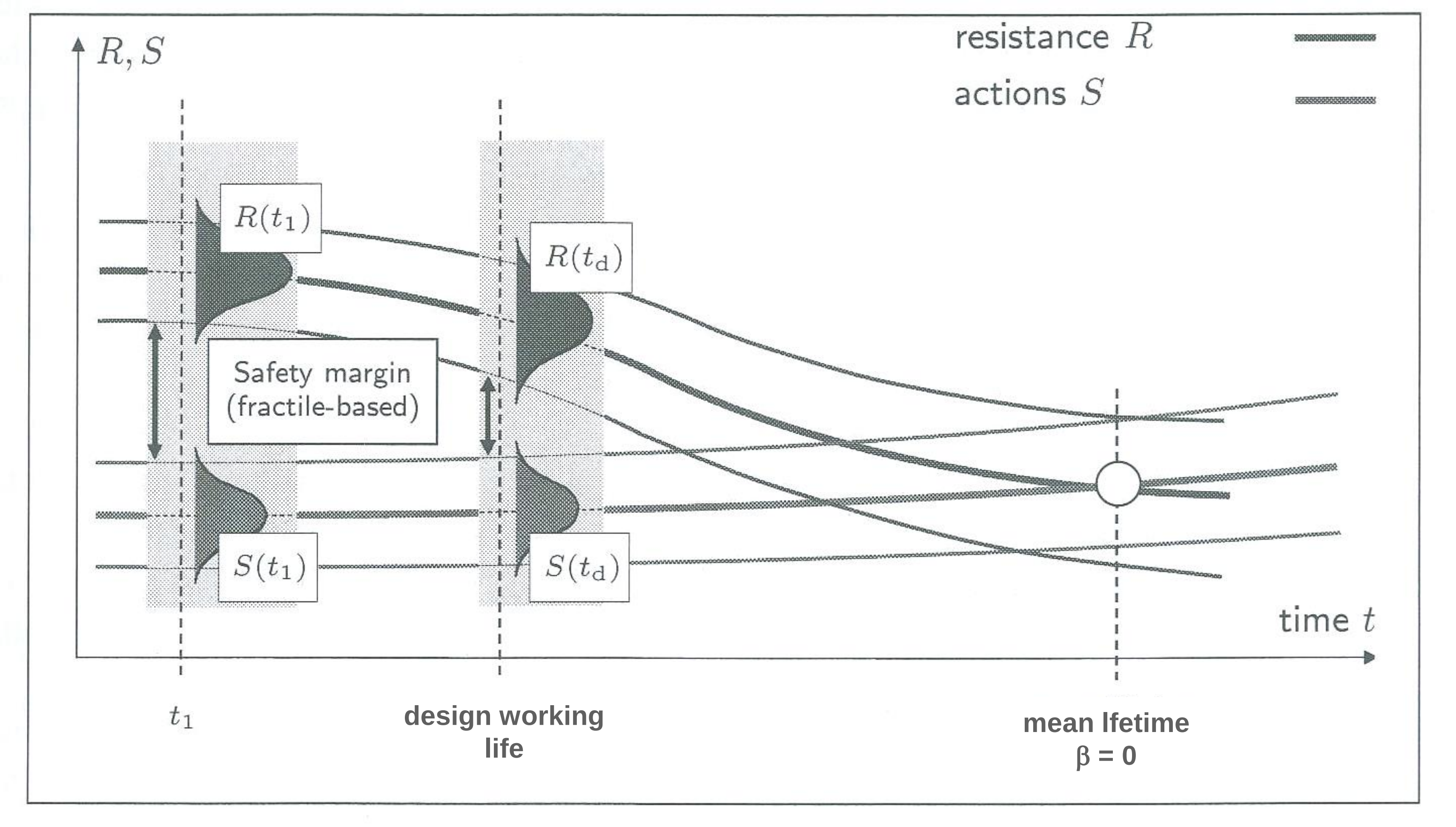
(1) The design working life should be specified.

NOTE Indicative categories are given in Table 2.1. The values given in Table 2.1 may also be used for determining time-dependent performance (*e.g.* fatigue-related calculations). See also Annex A.

Table 2.1 - Indicative design working life

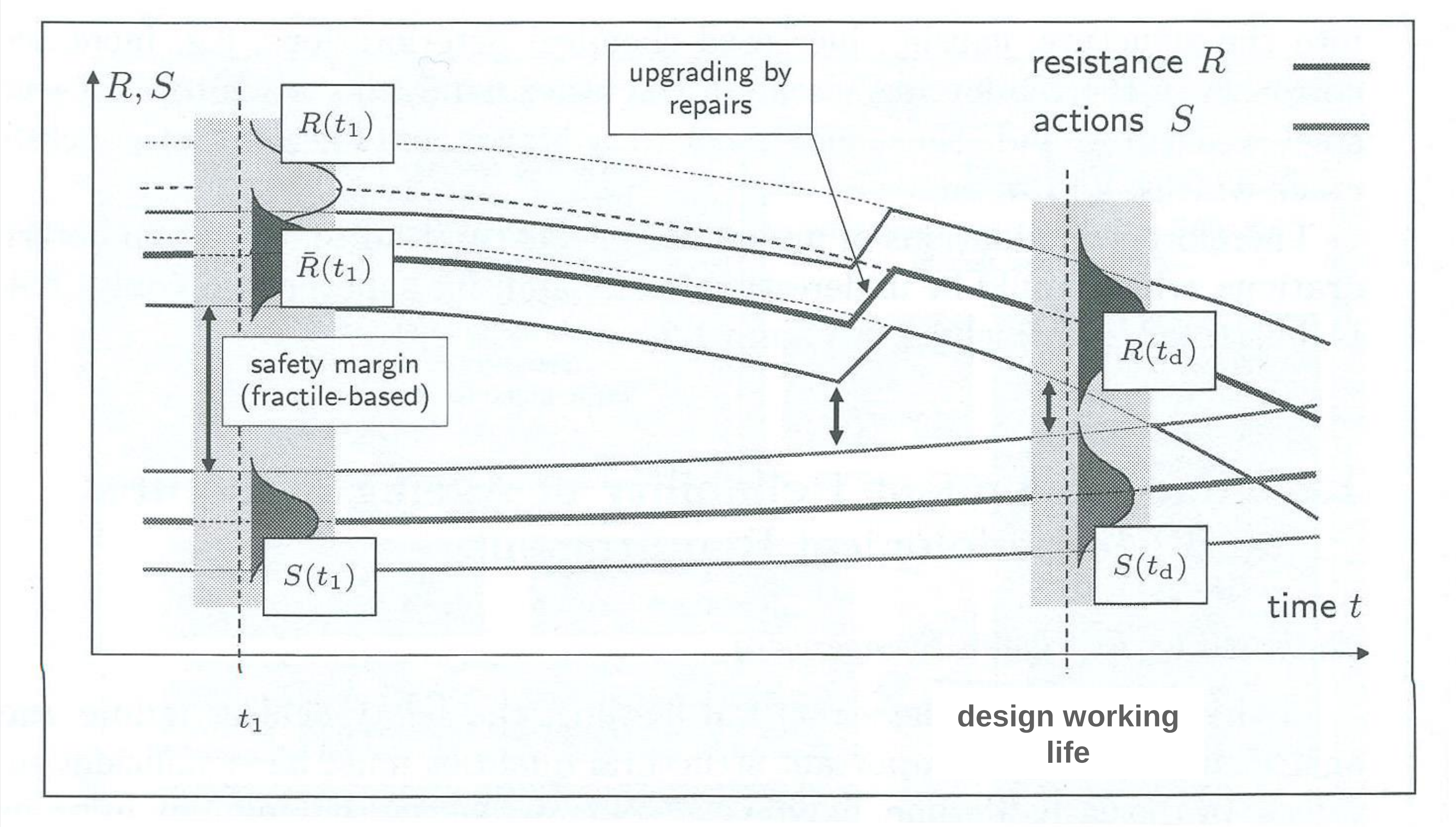
Design working life category	Indicative design working life (years)	Examples
1	10	Temporary structures (1)
2	10 to 25	Replaceable structural parts, e.g. gantry girders, bearings
3	15 to 30	Agricultural and similar structures
4	50	Building structures and other common structures
5	100	Monumental building structures, bridges, and other civil engineering structures
(1) Structures or parts of structures that can be dismantled with a view to being re-used should not be considered as temporary.		

Design working life

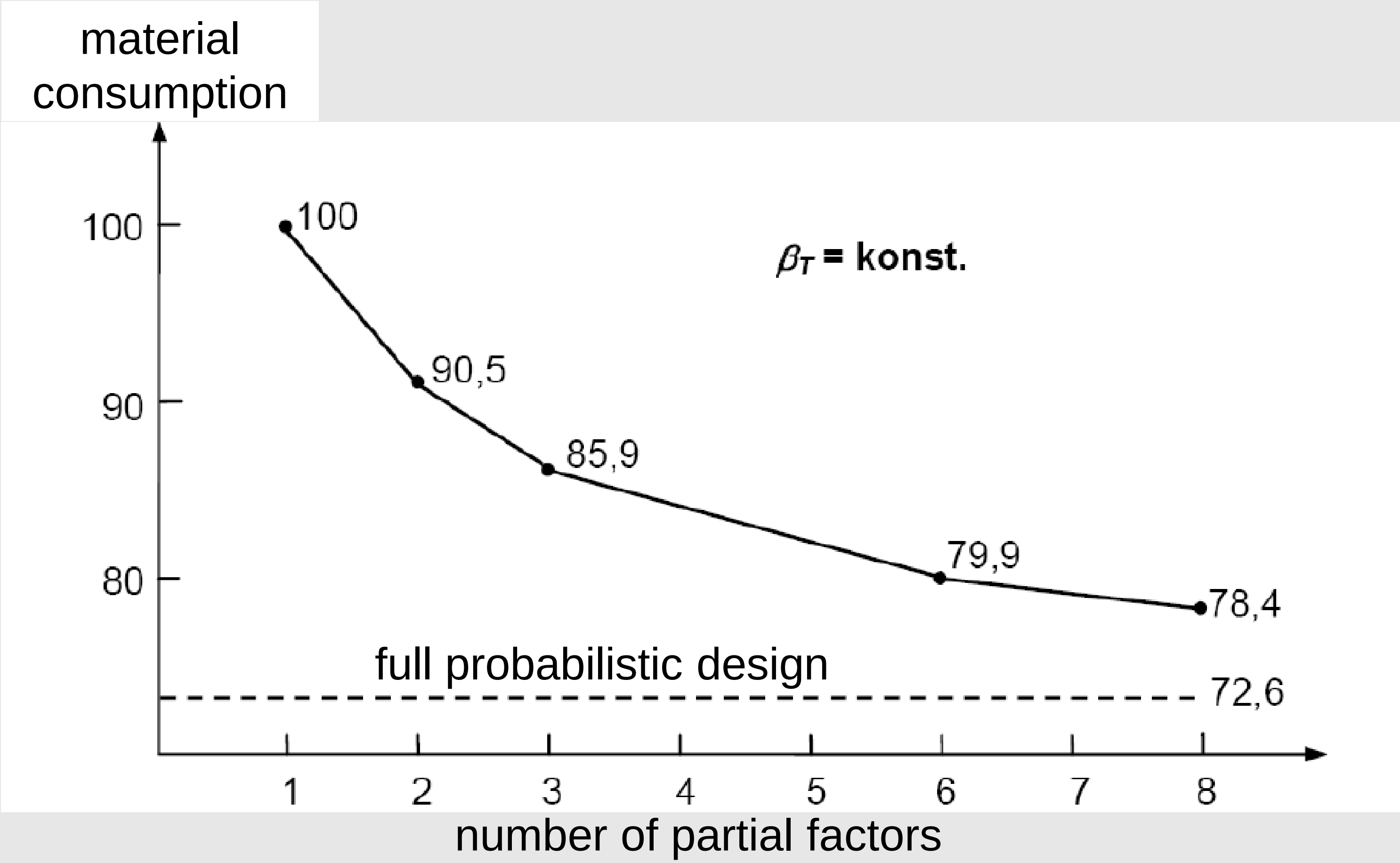


Design working life

Extension of design working life by upgrading



Conclusion

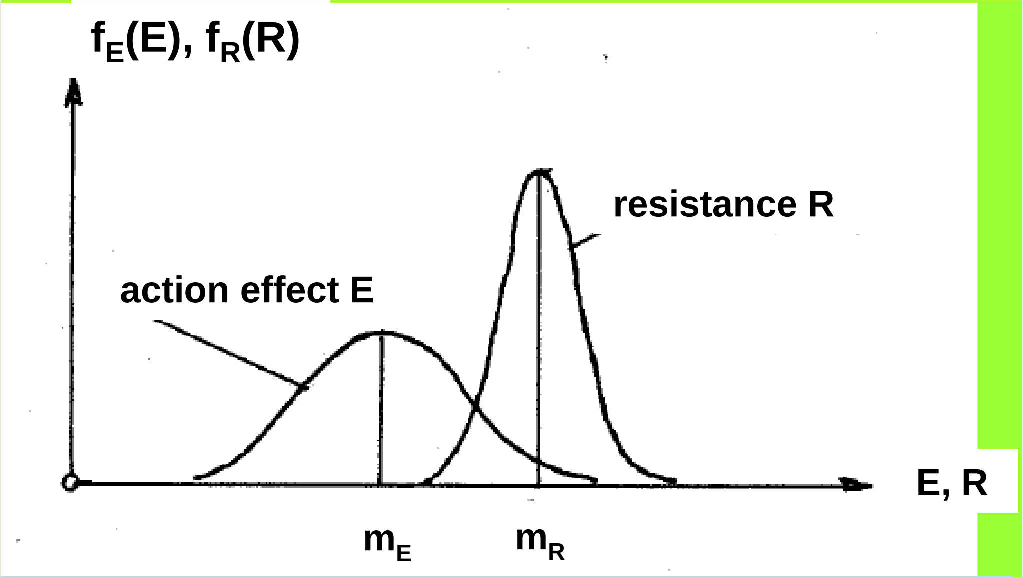


Gerhard Spaethe (1983)

The calculated structural material consumption is reduced by extending the flexibility of the reliability format.

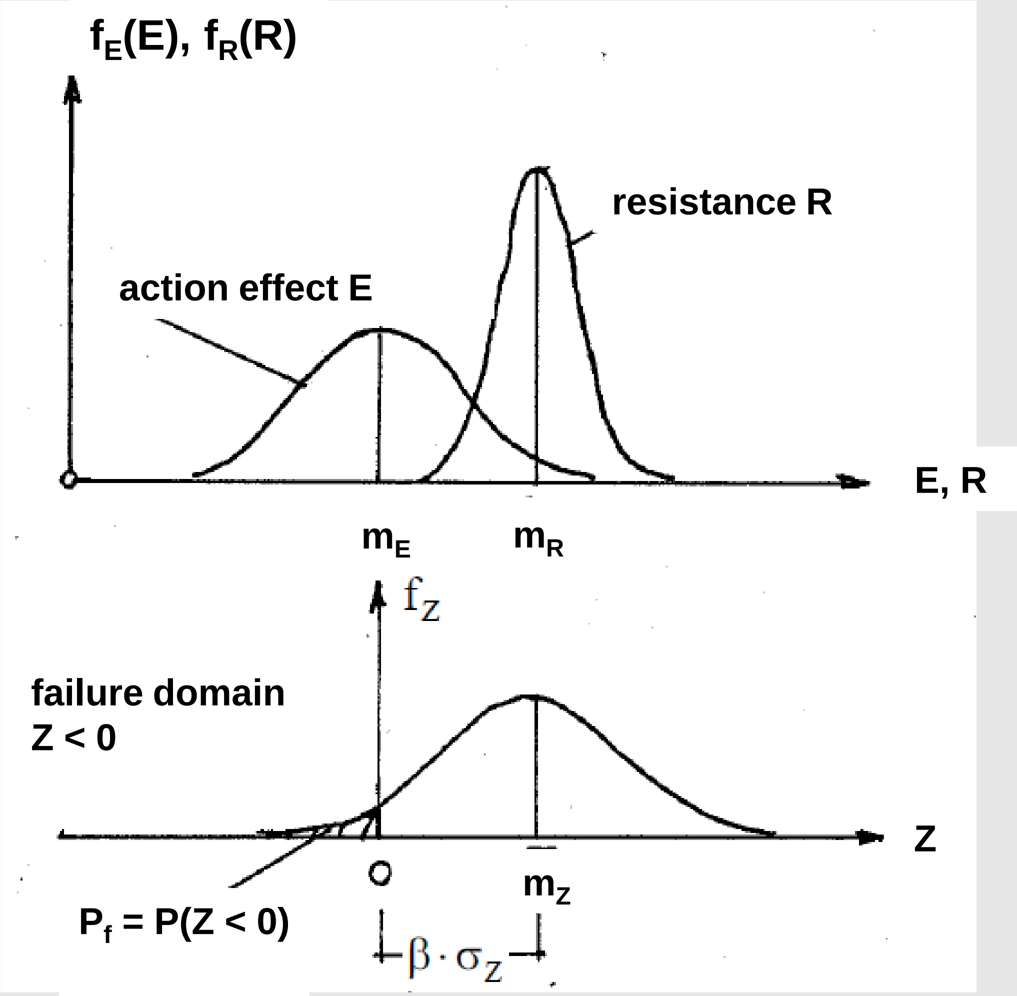
Measures of Reliability in a probabilistic concept

E - action effect with mean m_E, standard deviation σ_E;
R – resistance, load bearing capacity with mean m_R, standard deviation σ_R



Probability densities of the action effect, f_E and of the resistance, f_R

Measures of Reliability



E - action effect
with mean m_E,
standard deviation σ_E;
R – resistance, load bearing capacity
with mean m_R,
standard deviation σ_R

safety zone: $Z = R – E$
with mean m_Z, stand. dev. σ_Z

(Z is named „performance function g“ in EN 1990)

Partial Factor Concept in EN 1990

Probabilistic reliability verification results in the design values E_d and R_d:

$$R_d = m_R - \beta\alpha_R \cdot \sigma_R \geq E_d = m_E - \beta\alpha_E \cdot \sigma_E$$

The values of the weight factors are fundamental in the derivation of the design values. In ***EN 1990,*** the values are:

resistance weight factor $\alpha_R = 0{,}8$
load weight factor $\alpha_E = -0{,}7$

The probabilistic design values are then

$$R_d = m_R - 0{,}8\ \beta \cdot \sigma_R$$
$$E_d = m_E + 0{,}7\ \beta \cdot \sigma_E$$

EN 1990 does not apply directly the ***design values*** but utilises the ***partial factor design procedure.*** In it, the design values of action effects and resistance are calculated as

$$E_d = E\{\gamma_F \cdot F_k;\ \gamma_F \cdot \psi \cdot F_k\}$$
$$R_d = R\{X_k/\gamma_m\}$$

The probabilistic design values are used to calibrate the partial factors.

Eurocode 1: Actions on Structures

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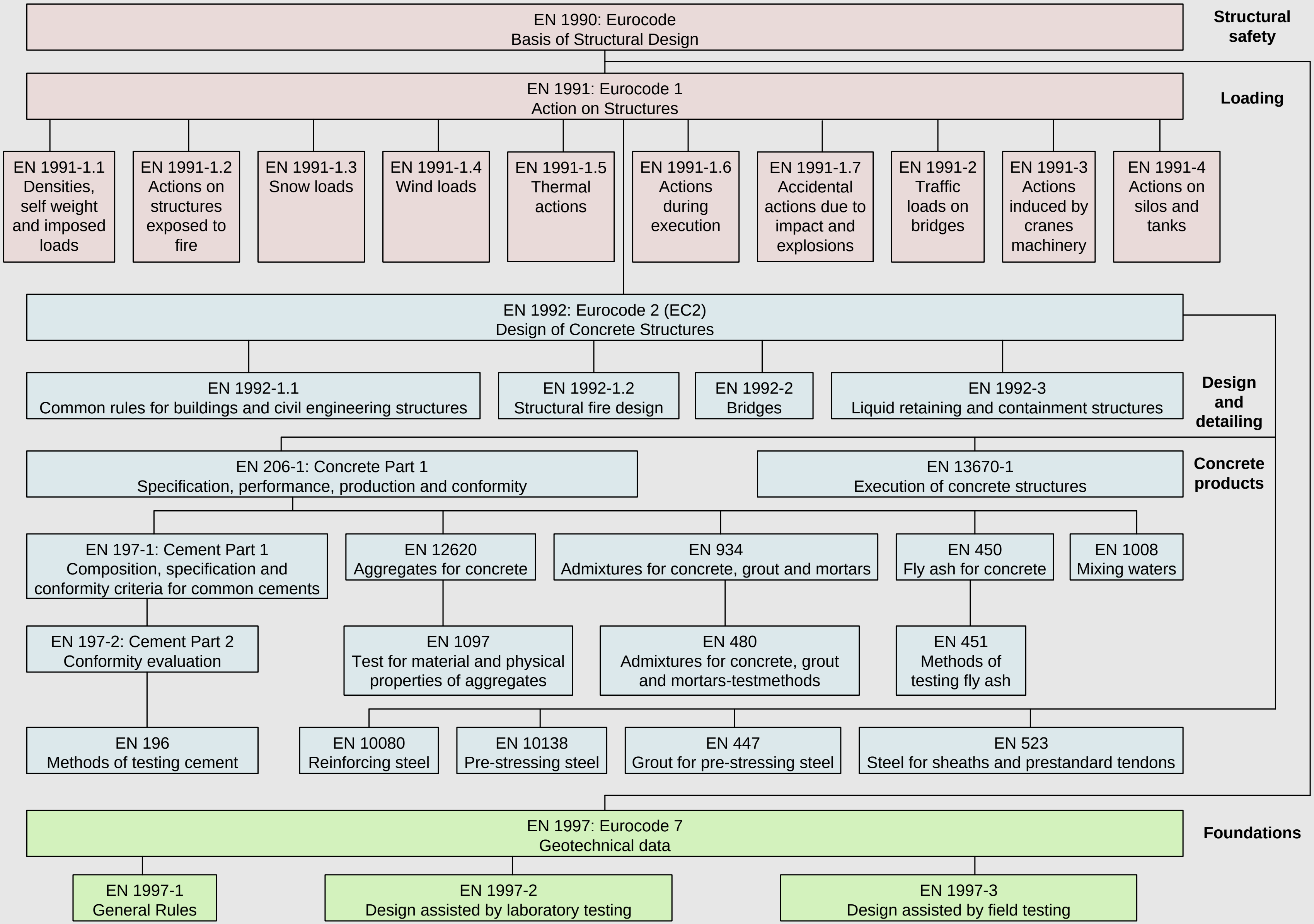
EN 1991-2	Traffic loads on Bridges : 2004
EN 1991-2	Traffic loads on Bridges : 2004

Structural Eurocodes



F_k – characteristic value of an action
E(F) – action effect

NOTE In so far as a characteristic value can be fixed on statistical bases, it is chosen so as to correspond to a prescribed probability of not being exceeded on the unfavourable side during a "reference period" taking into account the design working life of the structure and the duration of the design situation.

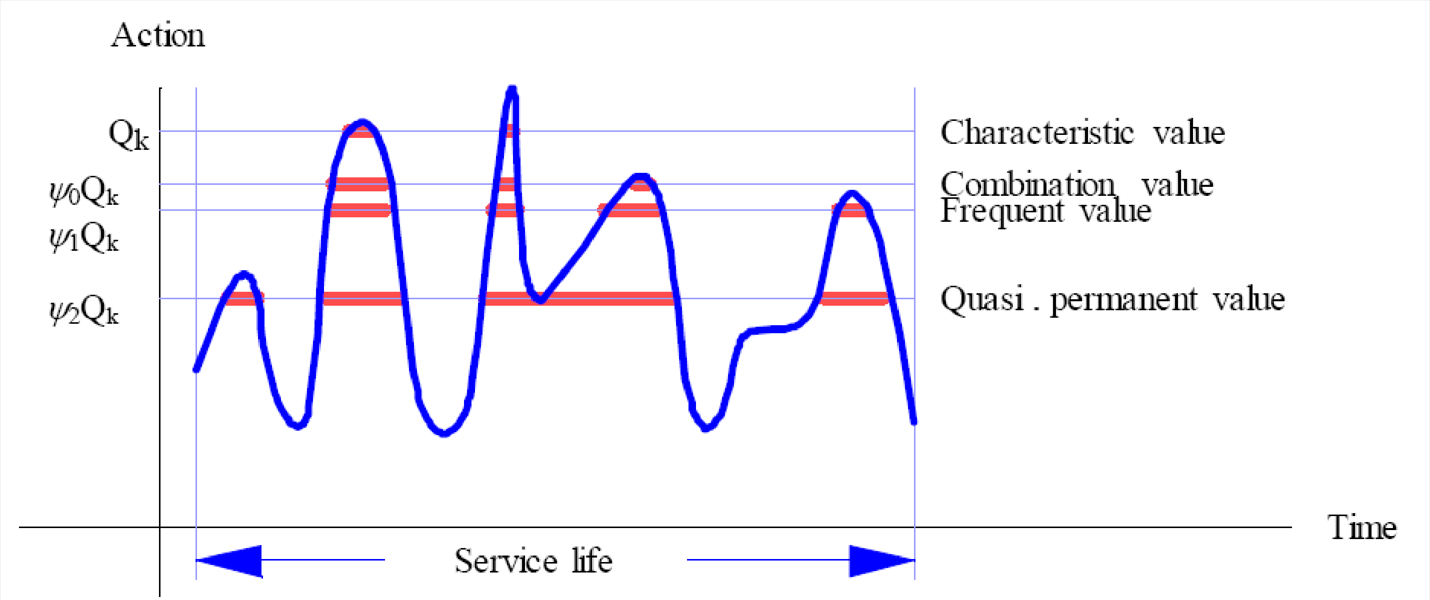
reference period
chosen period of time that is used as a basis for assessing statistically variable actions,

typically 1yr

R_k – characteristic value of resistance
X_k – characteristic value of a material property

value of a material or product property having a prescribed probability of not being attained in a hypothetical unlimited test series. This value generally corresponds to a specified fractile of the assumed statistical distribution of the particular property of the material or product. A nominal value is used as the characteristic value in some circumstances

Representative Values



The accompanying value of a variable action ($\psi\, Q_k$) is the value of a variable action that accompanies the leading action in a combination. The accompanying value of a variable action may be the combination value, the frequent value or the quasi-permanent value.

design working life

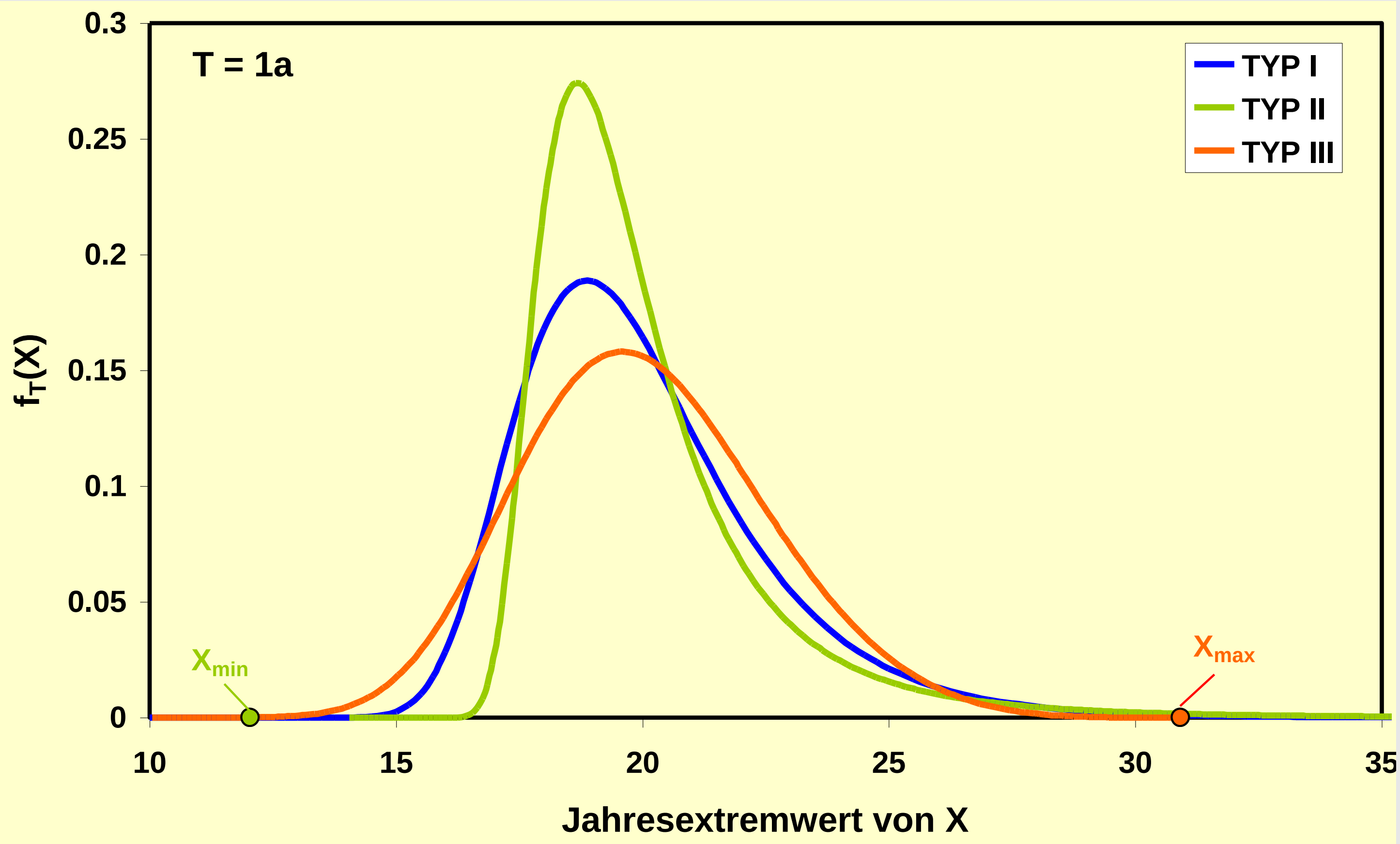
assumed period for which a structure or part of it is to be used for its intended purpose with anticipated maintenance but without major repair being necessary

expected working life [years]

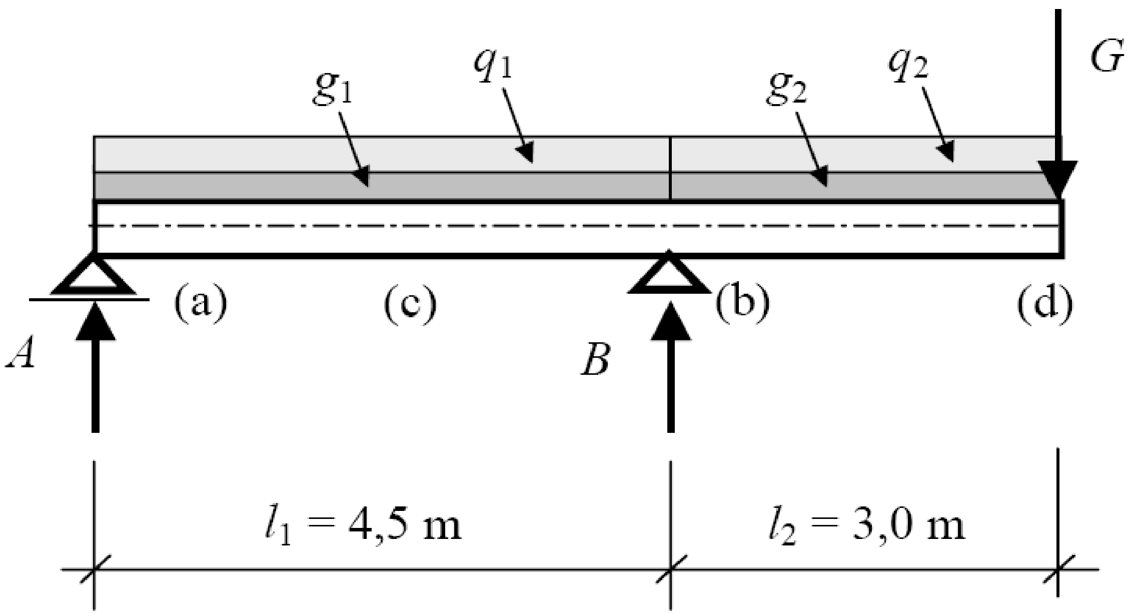
- **≥ 150** — **dams of water reservoirs**
- **≥ 80** — **bridges**
- **≥ 60** — **residential and business buildings**
- **30-40** — **industrial buildings**
- **≈ 10** — **temporary buildings**

Extreme Value Probability Distributions after Gumbel

probability density



Cantileverd Beam with Loadings

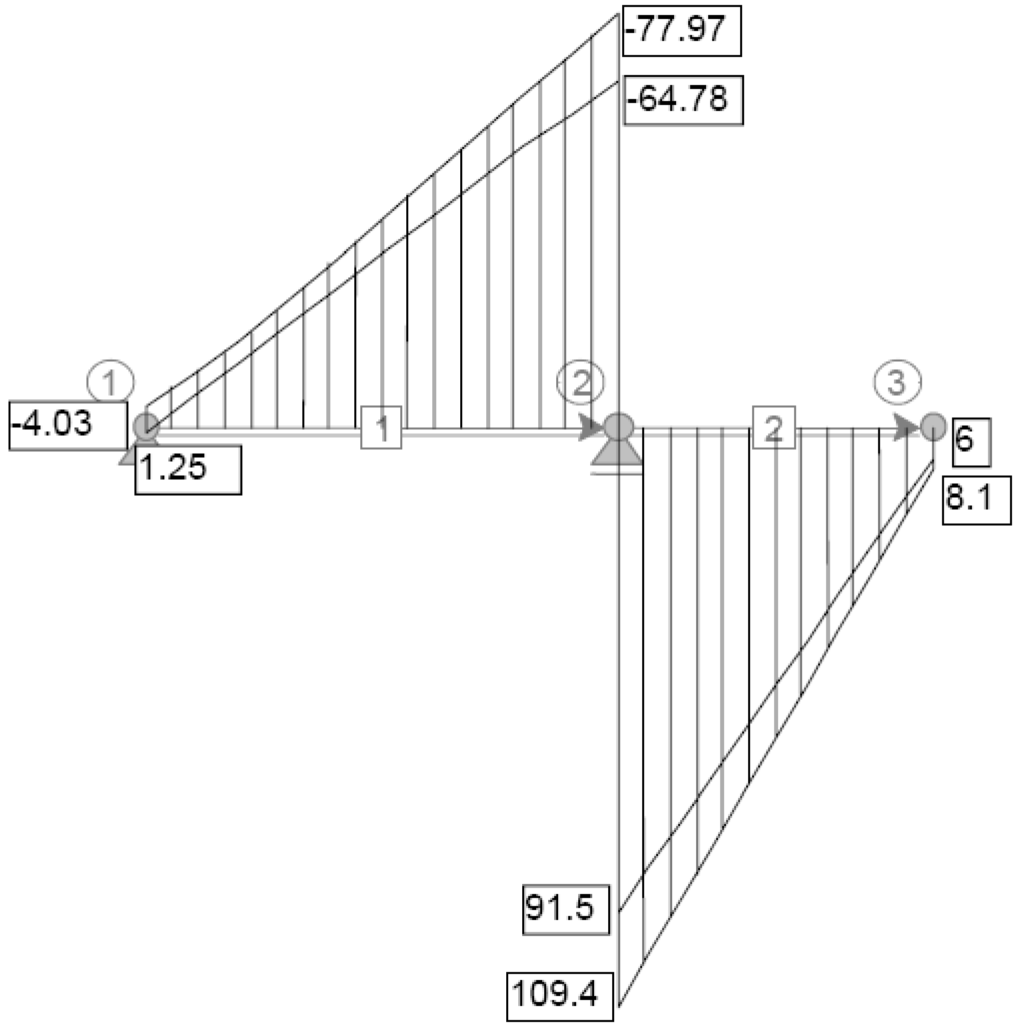


$g_1 = 15$ kN/m
$g_2 = 15$ kN/m
$q_1 = 9$ kN/m
$q_2 = 9$ kN/m
$G = 6$ kN/m

Table 1. Load cases and factors γ_G, γ_Q, $\gamma_Q \times \psi$ or $\xi \times \gamma_G$ corresponding to relevant expressions in EN 1990 [1] indicated in brackets, if g_1 and g_2 are actions from one source then factors in brackets should be applied.

Load case	Bending moment in *)	Limit state	Factors γ_G, γ_Q, $\gamma_Q \times \psi$ or $\xi \times \gamma_G$ assuming $\gamma_G = 1,35$, $\gamma_Q = 1,50$, $\psi = 0,70$ and $\xi = 0,85$ for actions g_1	g_2	q_1	q_2	G
1	-	Equilibrium, exp. (6.7), (6.10)	0,90	1,10	0	1,50	1,10
2	-	Equilibrium, exp. (6.7), (6.10)	1,15	1,35	0	1,50	1,35
3	-	Equilibrium, exp. (6.7), (6.10)	1,00	1,00	0	1,50	1,00
4	(c)	Ultimate, exp. (6.10)	1,35	1,00 (1,35)	1,50	0	1,00
5	(b)	Ultimate, exp. (6.10)	1,00 (1,35)	1,35	0	1,50	1,35
6	(c)	Ultimate, exp. (6.10a)	1,35	1,00 (1,35)	1,50×0,7	0	1,00
7	(c)	Ultimate, exp. (6.10b)	1,15	1,00 (1,15)	1,50	0	1,00
8	(b)	Ultimate, exp. (6.10a)	1,00 (1,35)	1,35	0	1,50×0,7	1,35
9	(b)	Ultimate, exp. (6.10b)	1,00 (1,15)	1,15	0	1,50	1,15
10	(c)	Ultimate, exp. ($6.10a_{mod}$)	1,35	1,00 (1,35)	0	0	1,00
11	(b)	Ultimate, exp. ($6.10a_{mod}$)	1,00 (1,35)	1,35	0	0	1,35
12	-	Serviceability, exp. (6.14)	1,00	1,00	1,00	0	1,00
13	-	Serviceability, exp. (6.14)	1,00	1,00	0	1,00	1,00
14	-	Serviceability, exp. (6.15)	1,00	1,00	1,00×0,5	0	1,00
15	-	Serviceability, exp. (6.15)	1,00	1,00	0	1,00×0,5	1,00
16	-	Serviceability, exp. (6.16)	1,00	1,00	1,00×0,3	0	1,00

EQU – Static Equilibrium



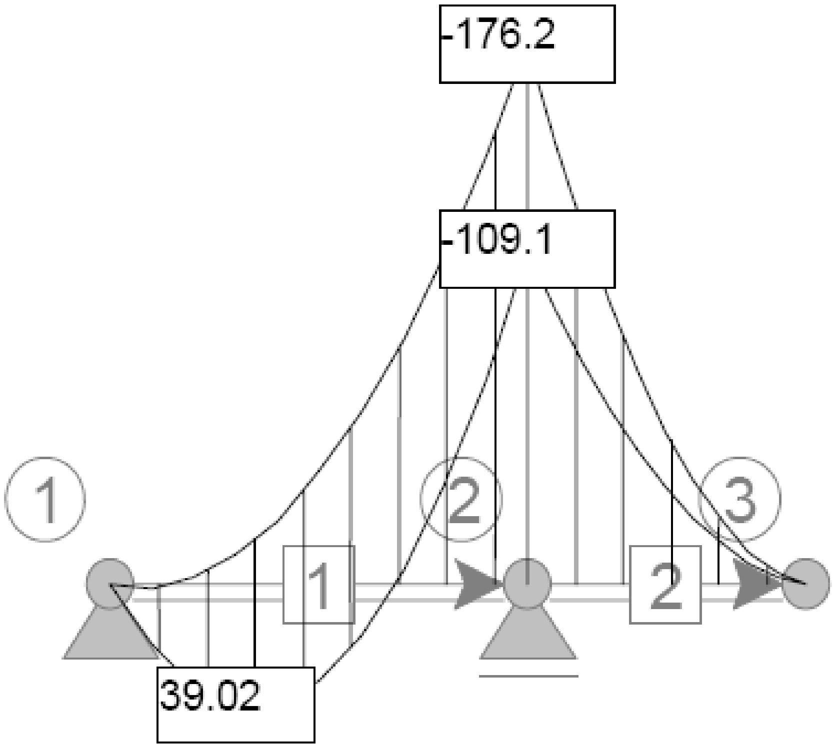


Figure 3a. Bending moment envelopes [kNm] according to expression (6.10) assuming g_1, g_2 being from one source.

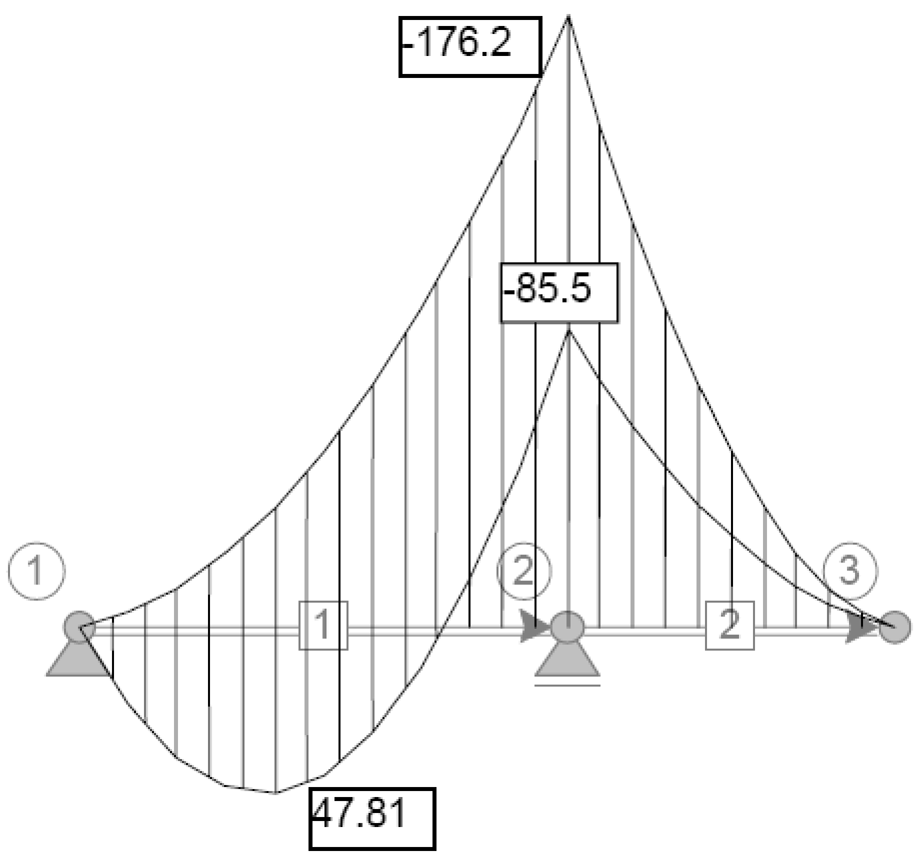


Figure 3b. Bending moment envelopes [kNm] according to expression (6.10) assuming g_1, g_2 independent.

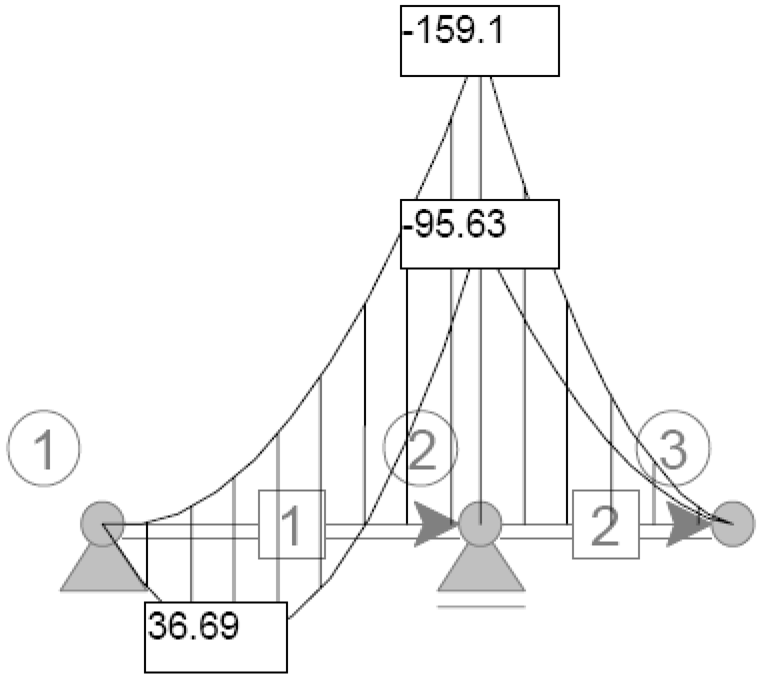


Figure 4a. Bending moment envelopes [kNm] according to exp. (6.10a), (6.10b) and (6.10a$_{mod}$), (6.10b) assuming g_1, g_2 being from one source.

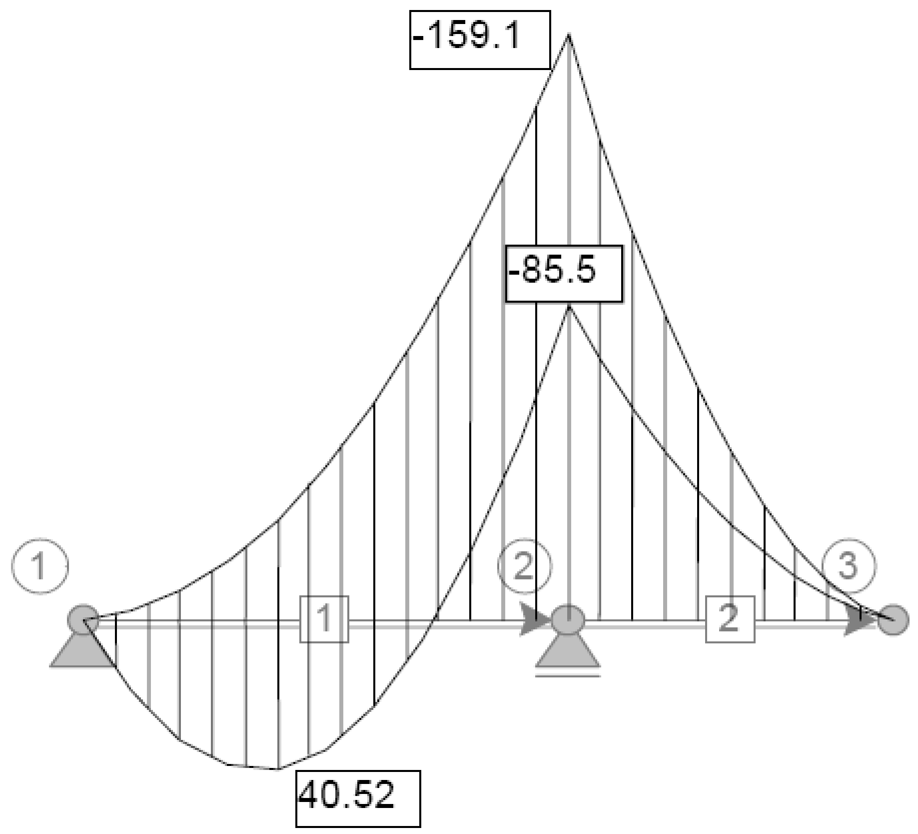


Figure 4b. Bending moment envelopes [kNm] according to exp. (6.10a), (6.10b) and (6.10a$_{mod}$), (6.10b) assuming g_1, g_2 independent.

The combination value of a variable action ($\psi_0 Q_k$)

Represented as a product of the characteristic value multiplied by the coefficient ψ_0 ($\psi_0 \leq 1$). It is used for the verification of ultimate limit states and irreversible serviceability limit states; is the value chosen - in so far as it can be fixed on statistical bases - so that the probability that the effects caused by the combination will be exceeded is approximately the same as by the characteristic value of an individual action.

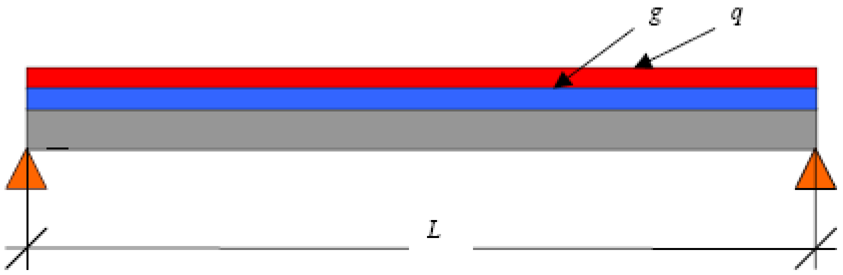
The frequent value of a variable action ($\psi_1 Q_k$)

Represented as a product $\psi_1 Q_k$, used for the verification of ultimate limit states involving accidental actions and for verifications of reversible serviceability limit states; is the value determined –also if it can be fixed on statistical bases - so that either the total time, within the reference period, during which it is exceeded is only a small given part of the reference period, or the frequency of it being exceeded is limited to a given value. For buildings, for example, the frequent value is chosen so that the time it is exceeded is 0,01 of the reference period; for road traffic loads on bridges, the frequent value is assessed on the basis of a return period of one week. It may be expressed as a determined part of the characteristic value by using a factor $\psi_1 \leq 1$.

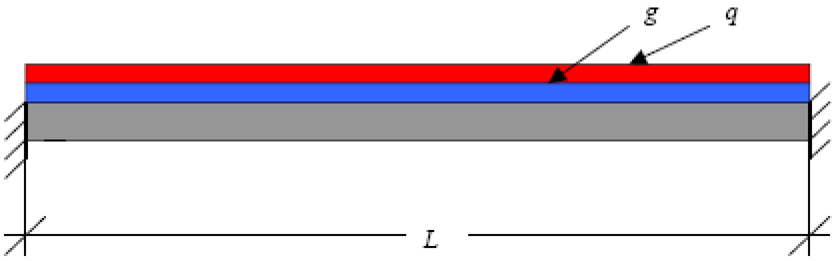
The quasi-permanent value of a variable action ($\psi_2 Q_k$)

Represented as a product $\psi_2 Q_k$, used for the verification of ultimate limit states involving accidental actions and for the verification of reversible serviceability limit states. Quasi-permanent values are also used for the calculation of long-term effects; is the value determined so that the total period of time for which it will be exceeded is a large fraction of the reference period. It may be expressed as a determined part of the characteristic value by using a factor $\psi_2 \leq 1$.

a) Simply supported beam: IPE 240 S235

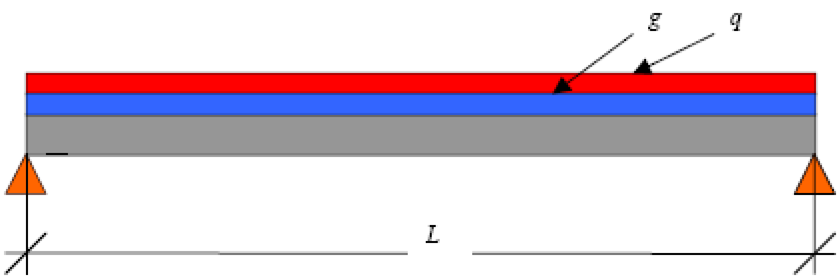


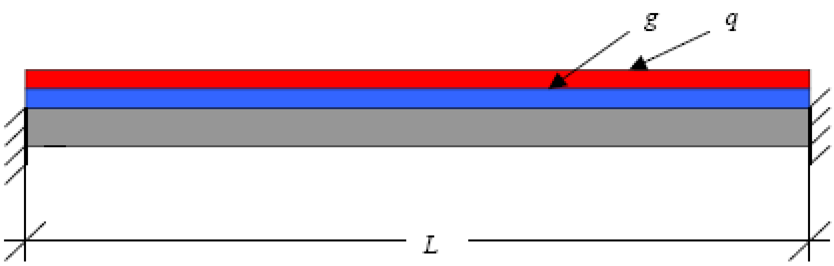
b) Double fixed beam IPE 220 S235



Span	$L = 6{,}0$ m
Cross section area:	$A = 39{,}12 \cdot 10^{-4}$ m^2
Moment of inertia	$I_y = 3\,892 \cdot 10^{-8}$ m^4
Yield stress	$f_y = 235$ MPa
Elastic modulus	$E = 210\,000$ MPa
Thermal expansion coef.:	$\alpha = 12 \cdot 10^{-6}$ / °C

Span	$L = 6{,}0$ m
Cross section area:	$A = 33{,}37 \cdot 10^{-4}$ m^2
Moment of inertia	$I_y = 2\,772 \cdot 10^{-8}$ m^4
Yield stress	$f_y = 235$ MPa
Elastic modulus	$E = 210\,000$ MPa
Thermal expansion coef.:	$\alpha = 12 \cdot 10^{-6}$ / °C





Actions, characteristic value:

Direct:

Permanent load: $g_k = 7{,}0$ kN/m

Variable load: $q_k = 3{,}0$ kN/m

Indirect:

Uniform temperature increase: $\Delta T = 20°C$

Settlement at one support: $\delta = 12$mm

Actions, *characteristic value:*

Direct:

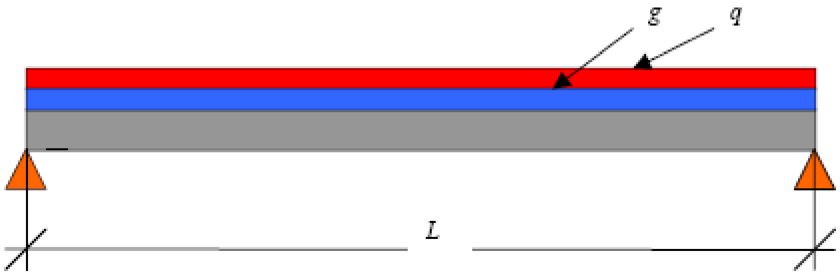
Permanent load: $g_k = 7{,}0$ kN/m

Variable load: $q_k = 3{,}0$ kN/m

Indirect:

Settlement at one support: $\delta = 12$mm

Uniform temperature increase: $\Delta T = 20°C$



Effects of actions, characteristic value:

Permanent loads:

Mid span moment

$1/8\ g_k L^2 = 31{,}5$ kNm

Variable loads

Mid span moment

$1/8\ q_k L^2 = 13{,}5$ *kNm*

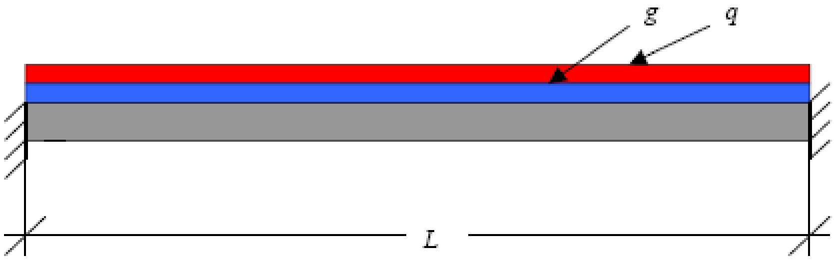
Indirect:

Settlement at one support: $\delta = 12$ mm

No effects

Uniform temperature increase: $\Delta T = 20°C$

no effects*



Effects of actions, characteristic value:

Permanent loads:

Mid span moment

$1/24\ g_k L^2 = 10{,}5$ kNm

Moment at supports

$-1/12\ g_k L^2 = -91{,}0$ kNm

Variable loads

Mid span moment

$1/24\ q_k L^2 = 4{,}5$ kNm

Moment at supports

$-1/12\ q_k L^2 = -9{,}0$ kNm

Indirect:

Settlement at one support: $\delta = 12$ mm

Mid span moment 0 kNm

Moment at supports

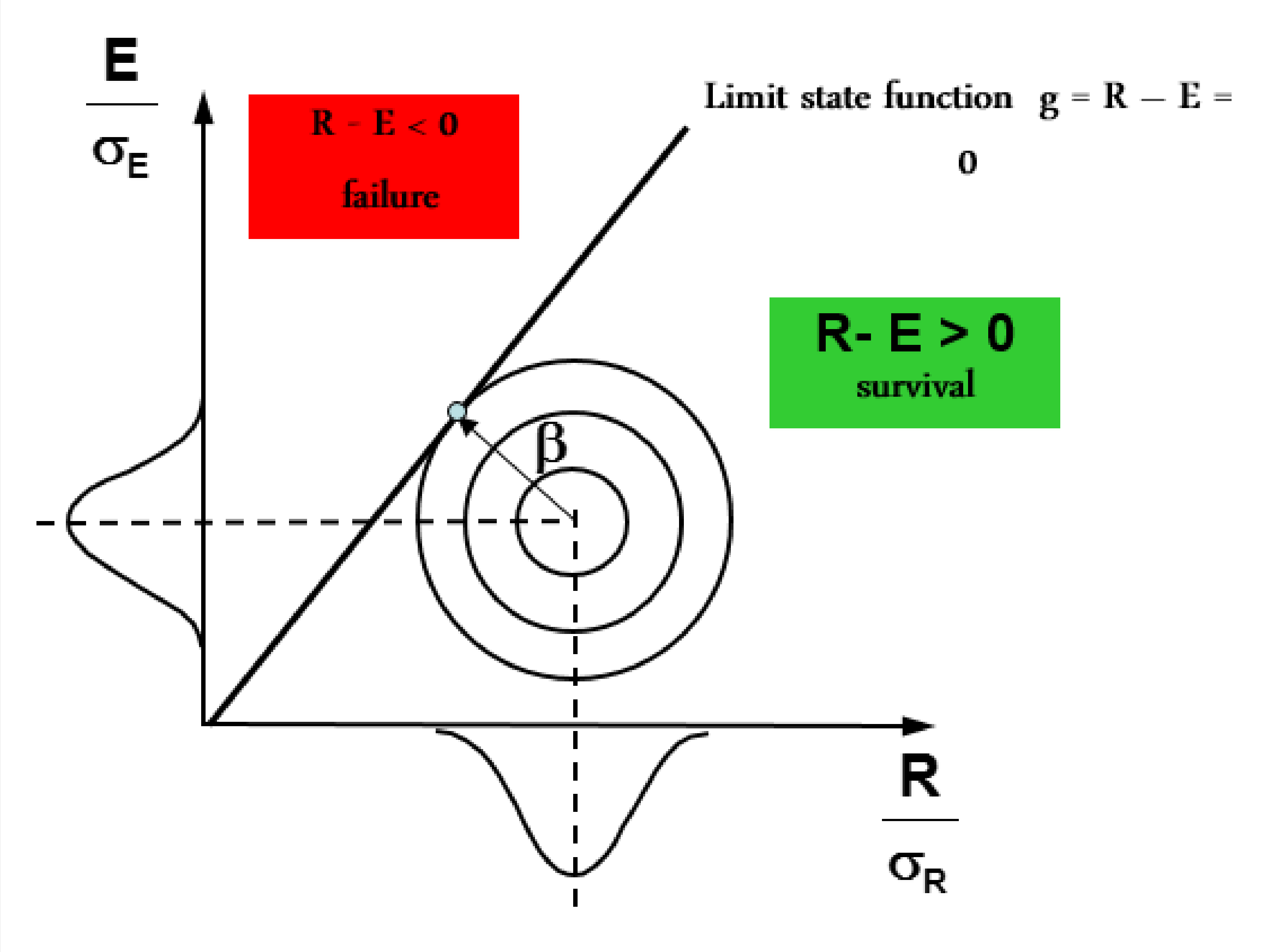
$\pm\delta\ 6\ EI / L^2 = \pm\ 11{,}64$ kNm

Uniform temperature increase: ΔT

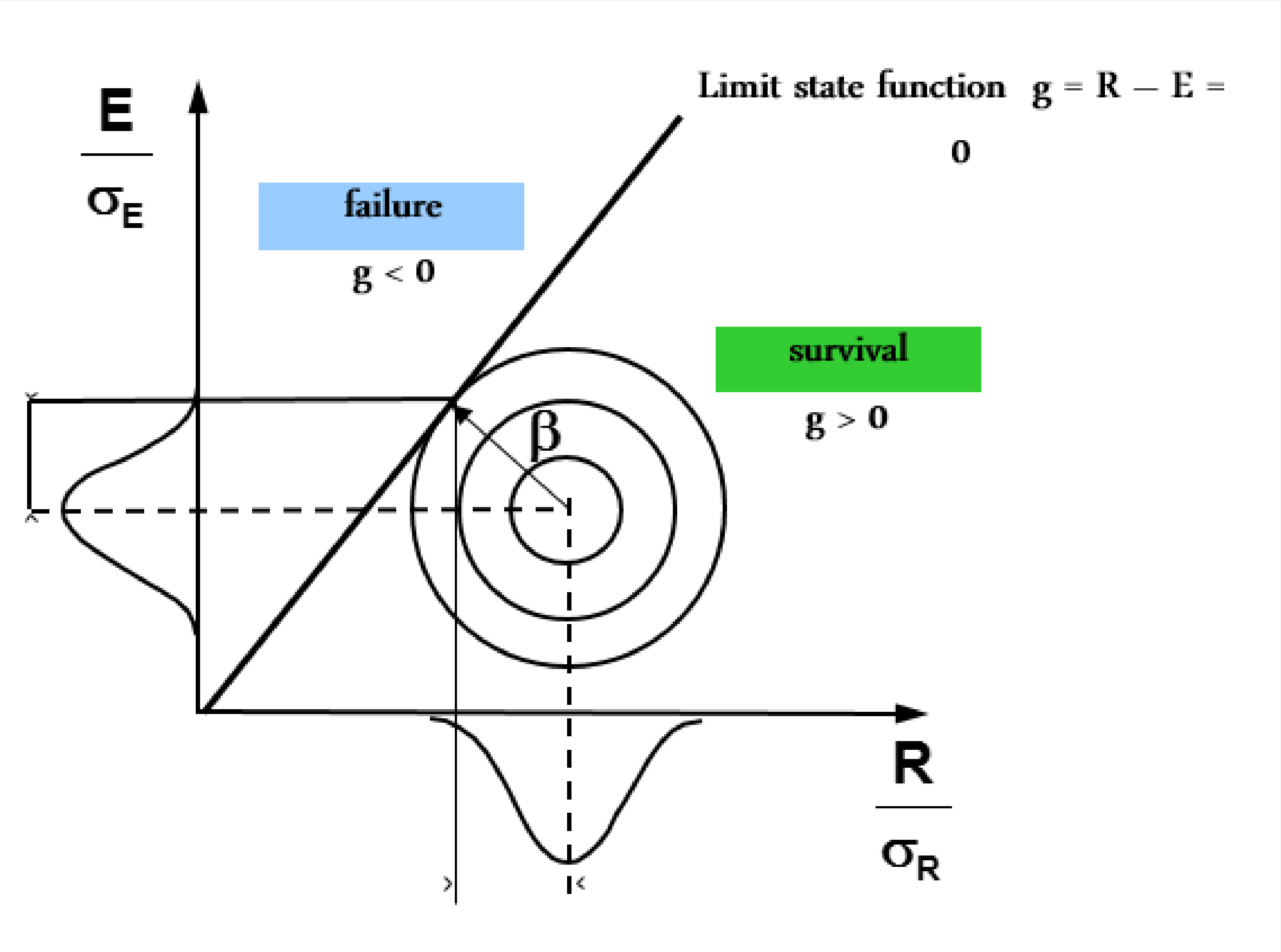
Uniform compression stress*

$\sigma = \alpha\ E\ \Delta T = 50{,}4$ Mpa

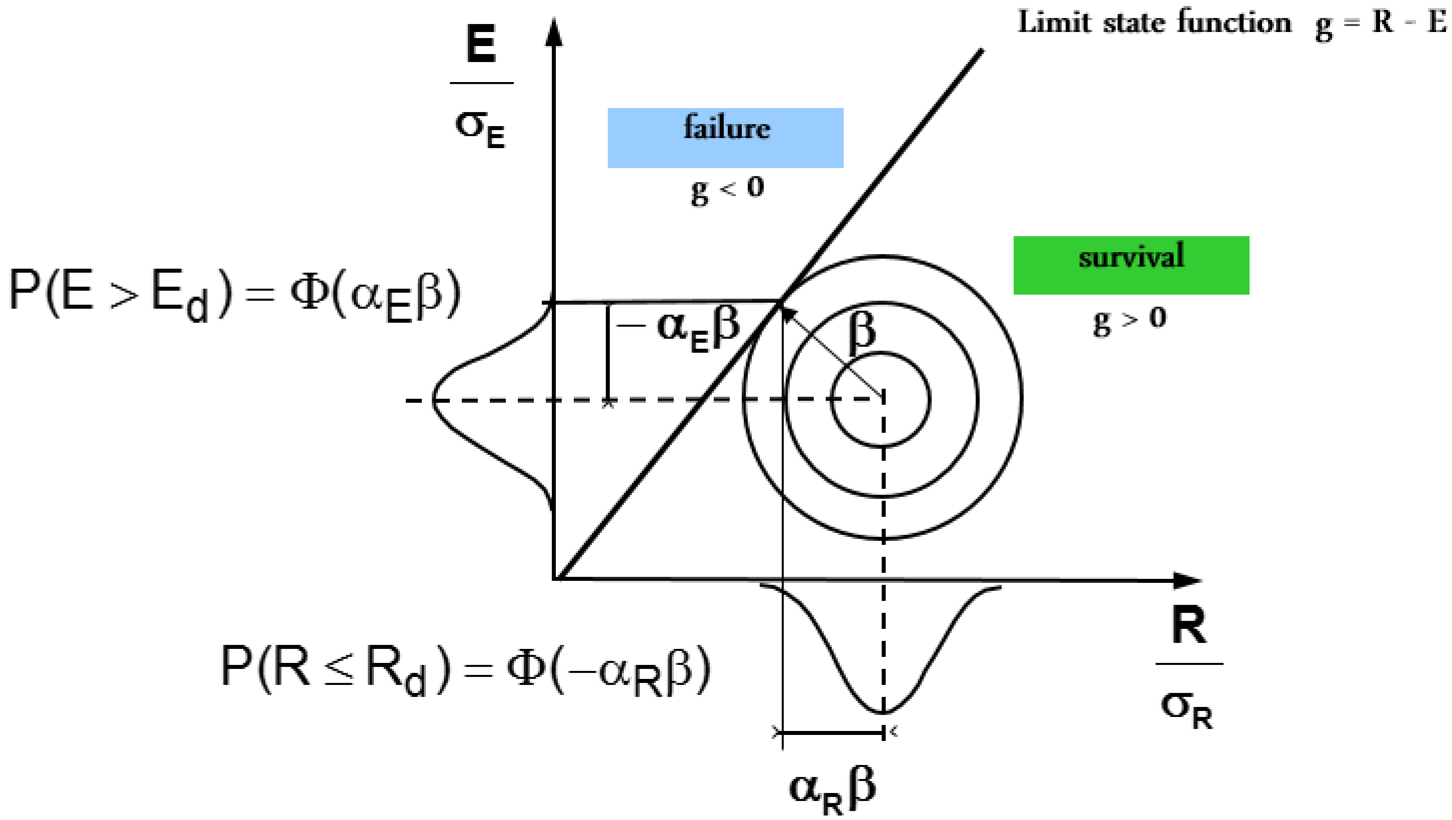
Joint probability density of the load effect E and resistance R



Joint probability density of the load effect E and resistance R



Joint probability density of the load effect E and resistance R



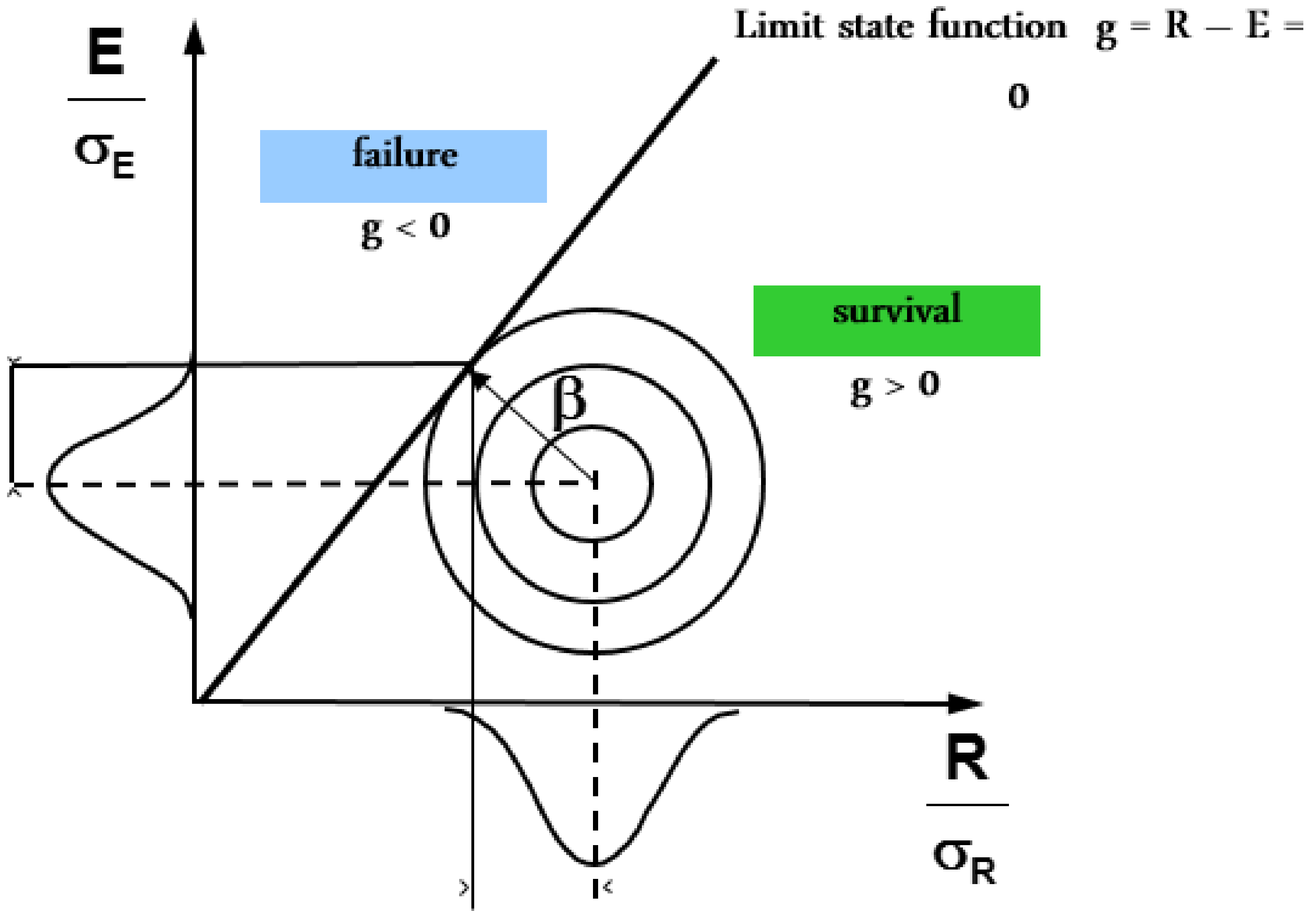
$$E_d \leq R_d$$

design values for the Gaussian distribution

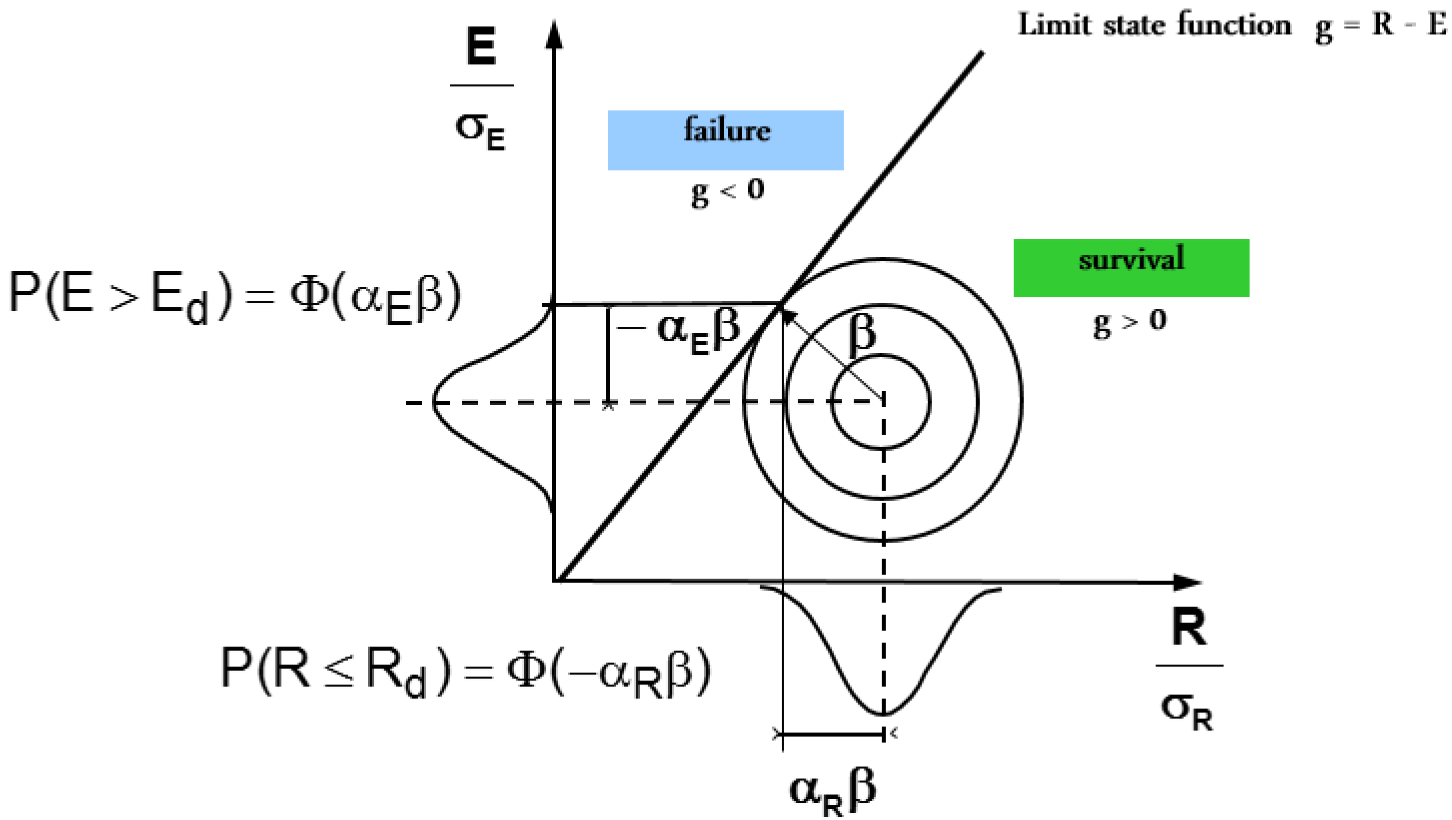
$$E_d = \mu_E - \alpha_E \beta \sigma_E \qquad \alpha_E < 0$$

$$R_d = \mu_R - \alpha_R \beta \sigma_R \qquad \alpha_R > 0$$

Joint probability density of the load effect E and resistance R



Joint probability density of the load effect E and resistance R



$$E_d \leq R_d$$

design values for the Gaussian distribution

$$E_d = \mu_E - \alpha_E \beta \sigma_E \qquad \alpha_E < 0$$

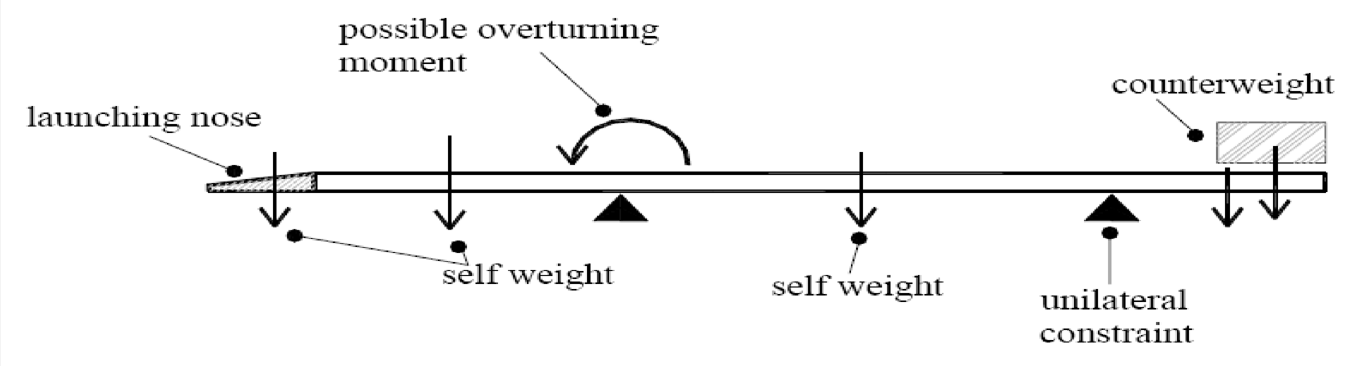
$$R_d = \mu_R - \alpha_R \beta \sigma_R \qquad \alpha_R > 0$$

Ultimate Limit States

Loss of Equilibrium ***EQU***
Structural Failure ***STR***
Failure of the Soil ***GEO***
Failure due to Fatigue ***FAT***

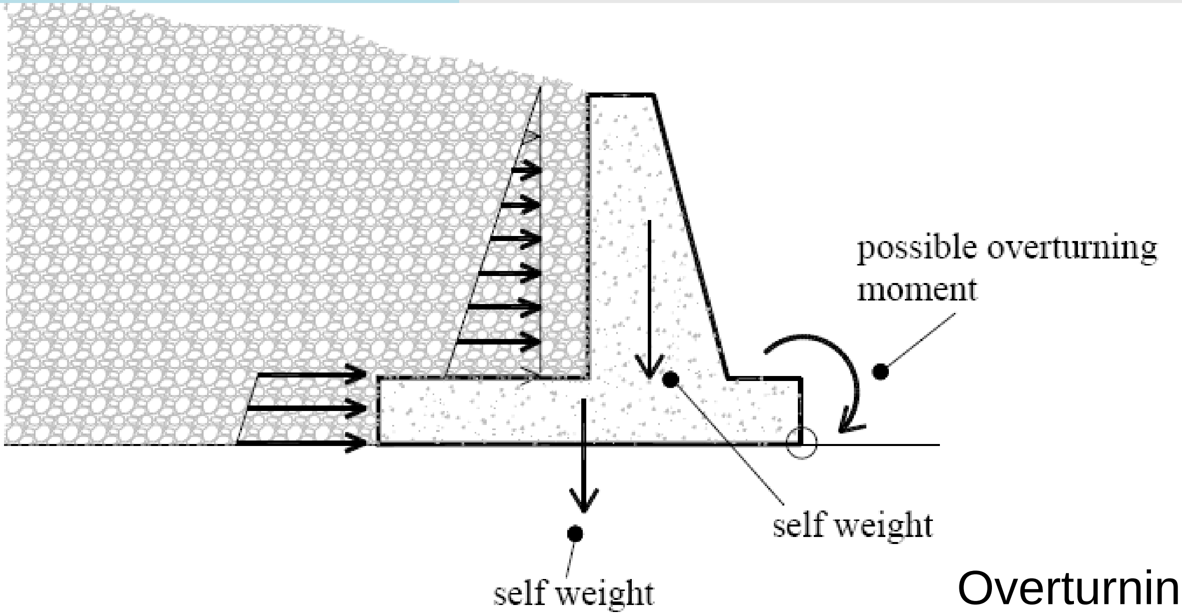
EQU Limit States

These involve the loss of static equilibrium in the considered structure, either as whole rigid body or in any one of its parts. In such situations, the mechanical and resistance properties of the materials are not generally determining factors, while even modest geometric variations in the distribution of actions or their points of application may be crucial. Going beyond such limit conditions generally causes collapse of the structure, and their inclusion amongst the ultimate limit states thus seems obvious.



Bridge Construction with Launching Nose

EQU Limit States



Overturning of a Retaining Wall

Destabilising actions (unfavourable actions) must be taken into account by adopting higher design values, while assuming lower design values accounts for stabilising actions (which have a favourable effect on the structure's equilibrium). With regard to stabilising effects, only those actions that can reasonably be expected to occur in the structure should be included in the combination (for instance, when considering a specific stage of construction, the effective presence of finishing accessories or other equipment must be accounted for). It is moreover necessary to bear in mind the possibility that non-structural members can be replaced or removed.

EQU Limit States

Verifications of static equilibrium

$$E_{d,dst} \leq E_{d,stb} \quad (6.7)$$

where :

$E_{d,dst}$ is the design value of the effect of destabilising actions ;

$E_{d,stb}$ is the design value of the effect of stabilising actions.

6.4.3 Combination of actions

$$E_d = E\{\gamma_{G,j} G_{k,j} \,;\, \gamma_P P \,;\, \gamma_{Q,1} Q_{k,1} \,;\, \gamma_{Q,i} \psi_{0,i} Q_{k,i}\} \quad j \geq 1 \,;\, i > 1 \quad (6.9b)$$

G_k, Q_k characteristic values of permanent and variable actions
γ partial factors
ψ combination factors

$$\sum_{j\geq 1} \gamma_{G,j} G_{k,j} \text{"+"} \gamma_P P \text{"+"} \gamma_{Q,1} Q_{k,1} \text{"+"} \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \quad (6.10)$$

Distinction between Principles and Application Rules

EN 1990, 1.4, constitutes

Principles comprise
- general statements and definitions without alternative;
- requirements and analytical models for which no alternative is permitted.

Application Rules are in accordance with the State of the Art and comply with the Principles

It is permissible to use ***alternative design rules*** different from the Application Rules, provided it is shown that the alternative rules accord with the relevant Principles and are equivalent with regard to the structural safety, serviceability and durability.