Analysis and design of steel structures for buildings according to Eurocode 0, 1 and 3

Steel Design 1

H.H. Snijder

H.M.G.M. Steenbergen

Structural basics

Ernst & Sohn
A Wiley Brand

bouwen met staal

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Colophon

text	prof.ir. H.H. Snijder (chapter 1 to 7)
	ir. H.M.G.M. Steenbergen (chapter 1 and 2)
editing	ir. C.H. van Eldik / Bouwen met Staal
graphic design	Karel Ley / Fig.84-Reclamestudio

published by	Bouwen met Staal
ISBN (print)	978-90-72830-98-2
ISBN (PDF)	978-3-433-61069-5

The publication of this textbook has been made possible by:

Bauforumstahl	www.bauforumstahl.de
Bouwen met Staal	www.bouwenmetstaal.nl
Infosteel	www.infosteel.be
Stahlbau Zentrum Schweiz	www.szs.ch
Tata Steel	www.tatasteelconstruction.com
World Steel Association	www.constructsteel.org

This textbook was originally published in 2011 (and updated in 2012) by Bouwen met Staal, in Dutch, as *Krachtswerking* by the same authors. The English translation has been prepared by prof.ir. H.H. Snijder and ir. F.M.W. van den Hove (both Eindhoven University of Technology) and checked by dr. G. Couchman (The Steel Construction Institute).

The text is based on the (English) EN version of the Eurocodes using default and/or recommended values. Where a country can make a national choice – or when non-contradictory complementary information may be used – this is indicated by the following symbol: NA. Separate annexes contain the national choices for Belgium, Germany, Luxembourg, The Netherlands and Switzerland. These annexes can be downloaded free of charge from the websites of the (national) organisations as well as any errata, corrections and additions to this textbook.

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Structural basics

1

Structural safety

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1 Structural safety

The safety of structures – of buildings in which we live and work amongst others – is a fundamental need of humanity. In many countries, the government sees it as its responsibility to guarantee structural safety. Structural safety is mostly addressed by legal regulations which designate the building standards and codes to be used, in particular the Eurocodes. In this way minimum requirements for the safety of structures are assured. Buildings and bridges (or parts of these) can collapse when their structural elements do not satisfy these minimum requirements, leading to significant damage or even casualties (fig. 1.1 and 1.2). Many publications, amongst others [1], discuss structural safety and reliability.
The general principles of structural safety are presented in the basic Eurocode EN 1990. This code describes the principles of structural design and analysis and provides guidelines for inter-dependent aspects of the structural reliability.

This chapter first discusses the theory and background of EN 1990 regarding:
– probability of failure (safety);
– reliability principles (taking uncertainties into account);
– design value of resistance (strength of a structure);
– design value of actions (action types and combinations of actions);
– reliability (consequence classes and reliability index).

Finally, the content and structure of EN 1990 is discussed briefly, following the order of the chapters in the code.

1.1 Collapsed parking deck at a hotel in Tiel (The Netherlands, 2002) due to, amongst other things, insufficient stability of the edge beam.

1.2 Collapsed Saint Anthony Falls Bridge in Minneapolis (USA, 2007) due to incorrectly designed joints (gusset plates) in the truss.

1.1 Probability of failure

When designing a structure, the structural engineer needs to show that the effect of actions E on the structure is lower than the resistance R of the structure during its design working life. The term 'actions' is broad, covering not only loads but also, for example, imposed deformations, and expansion due to changing temperature and creep.
The effect of actions on a structure depends on the following basic variables:
- actions and environmental influences;
- material and product properties;
- geometrical properties of the structure and its elements.

Using applied mechanics the effect of actions can be described in terms of internal forces – such as bending moments, shear forces and normal forces – or, for example, stresses, strains or deflections. After determining the geometrical properties of the structure – including the cross-section dimensions – the cross-section properties can be determined from books of tables, and the magnitude of the actions is determined using the different parts of the Eurocode on actions, EN 1991, see *Structural basics 2* (Actions and deformations). The material properties for steel structures follow from EN 1993-1-1. Thus, using this approach, all basic variables get one specific value.
The assessment procedure for structures is in this way similar as a deterministic approach. The structural engineer should appreciate that many basic variables in reality do not have the exact same values as those applied in the analysis. This is due to the fact that all basic variables are, statistically speaking, so-called stochastic variables: actions vary in time, dimensions vary between tolerance limits and material properties show certain variability. The structural engineer should therefore show that the probability of failure of the structure is sufficiently small. By following the Eurocode approach the engineer will implicitly ensure that the probability of failure is sufficiently small.
The probability of failure of a structure is denoted as P_f. The probability of survival P_s is the probability that the structure does not fail and is complementary to the probability of failure. According to the theory of probability, the sum of the probability of failure and the probability of survival is equal to one. The probability of survival is referred to as the reliability. The reliability of the structure is then:

$$P_s = 1 - P_f \tag{1.1}$$

The result of a reliability analysis is the probability that the structure survives, which is known as its reliability. This probability of survival is generally almost equal to 1, for example in the order of $P_s = 0{,}999999$, where the number 1 is equal to 100%. This can also be written as:

$$P_s = 0{,}999999 = 1 - 0{,}000001 = 1 - 10^{-6} \tag{1.2}$$

Equation (1.2) shows that the probability of failure is $P_f = 10^{-6}$. In the interests of clarity the result of a reliability analysis, although defining reliability, is usually presented as a probability of failure.

When the determined probability of failure is larger than a previously determined target value P_0, the structure is insufficiently safe. Another measure for the reliability is the reliability index β, which can be related to the probability of failure P_f as follows:

$$P_f = \Phi(-\beta) \tag{1.3}$$

Where Φ is the cumulative distribution function of the standardized normal distribution. For the relationship between the probability of failure P_f and the reliability index β reference is made to table 1.5. The Joint Committee on Structural Safety has determined four internationally accepted levels of sophistication to be used to assess the safety of a structure.

- *Level 0* (deterministic method). At level 0, specific fixed values for the action, the resistance and the safety coefficient are used.
- *Level I* (semi-probabilistic method). At level I, characteristic values for the action and the resistance are used with partial factors related to safety. This approach is very useful for codification and is the basis of the design rules in the Eurocodes.
- *Level II* (approximate probabilistic method). At level II, certain well-defined simplifications are applied for practical reasons. However, analyses at this level are still very complicated and time consuming. This method is not practical for 'ordinary' structural analyses. One method at level II is discussed in EN 1990, namely the first order reliability method (FORM).
- *Level III* (full probabilistic method). At level III, the analysis is fully based on the theory of probability and the action and the resistance – or any material property which determines the resistance – are regarded as stochastic variables. These variables are described statistically, for example for the action by a distribution function with an average and a standard deviation.

A mathematical relationship exists between the variables 'strength' and 'resistance', used to describe the limit state being considered. Next, the safety – or the reliability – can be determined. Such an analysis is too difficult to perform for normal practice.

The current Eurocodes are mainly based on considerations at level I, which are calibrated against considerations at level II. The structural engineer must ensure that the probability of failure of the structure is sufficiently small. This is achieved by applying a characteristic value for the action and multiplying this value by a partial factor for actions and determining a characteristic value for the resistance and dividing this by a partial factor for resistance. The desired probability of failure is not exceeded if it is shown that the effect of the characteristic action multiplied by the partial factor for actions is not larger than the characteristic resistance divided by the partial factor for resistance.

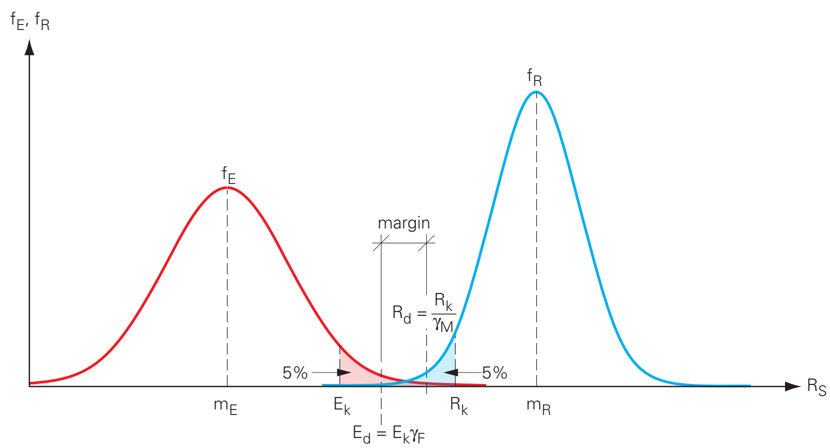
1.2 Reliability principles

EN 1990 requires that a structure is assessed at different limit states, making a distinction between ultimate and serviceability limit state. The ultimate limit state uses characteristic values of actions and resistances. Both these types of characteristic values are defined as values for which the probability that larger actions or lower resistances could occur is a specific percentage. Usually, 5% is assumed (fig. 1.3). Therefore, the characteristic value usually corresponds with the 5% fractile (the value which is only exceeded, or not reached, in 5% of all cases).
The design value of the effect of actions E_d is determined by multiplying the characteristic value of the effect of actions E_k by the partial factor for actions γ_F. The design value of the resistance R_d is determined by dividing the characteristic value of the resistance R_d by the partial factor for the resistance γ_M.
The partial factor for actions γ_F depends on the nature of the action and the limit state being considered. The following effects are taken into account by this factor:
– the possibility of unfavourable deviations of the value of the action compared to its characteristic value (for example the occurrence of extreme values and possible underestimation of the action due to a lack of sufficient statistical information);
– uncertainty in the models which are used to determine the action effects;
– uncertainty in the modelling of the actions (for example the wind action);
– inaccuracies in the combinations of actions (the actions on a building due to negative and positive wind pressure are not independent, while in the design these two actions are taken into account separately).

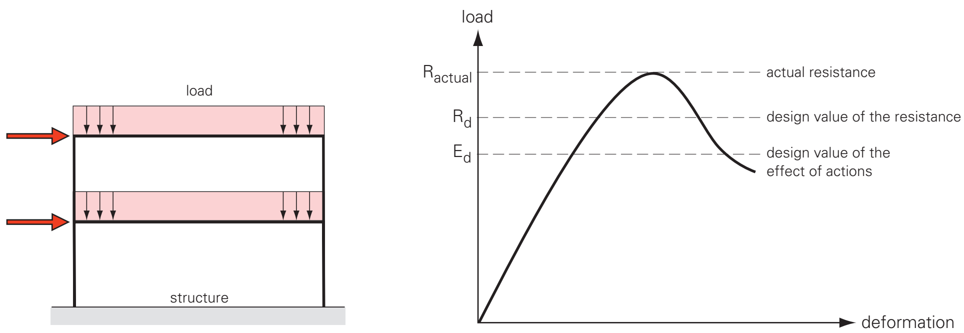
The partial factor for resistance γ_M takes into account the following effects:
– the possibility of unfavourable deviations of a material or product property compared to its characteristic value;
– uncertainty in the resistance model (also called the mechanical model) for determining the load bearing capacity (strength and stability);
– the possibility of deviations of certain geometrical dimensions, including cross-sectional dimensions.

The structure will be sufficiently safe when the following requirement is satisfied:

$$E_k \gamma_F \leq \frac{R_k}{\gamma_M} \qquad (1.4)$$



1.3 Basis of the assessment method with as a requirement $E_d \leq R_d$ (E = effect of actions; R = resistance; f = frequency).



1.4 Assessment method for the ultimate limit state.

This requirement is often presented as a unity check in the Eurocodes:

$$\frac{E_d}{R_d} \leq 1 \tag{1.5}$$

Where:
E_d design value of the effect of actions ($E_d = \gamma_F E_k$);
R_d design value of the corresponding resistance ($R_d = R_k/\gamma_M$);
E_k characteristic value of the effect of actions;
R_k characteristic value of the resistance;
γ_F partial factor for actions; $\gamma_F \geq 1$;
γ_M partial factor for resistance; $\gamma_M \geq 1$.

In this way it is possible to assess the reliability of the structure (fig. 1.4).

1.3 Design value of resistance

The design value of resistance of the structure follows from the resistance function of the design model. This model is based on a combination of theoretical considerations and the observed behaviour of structures during tests. The resistance function R of a member loaded in tension is, for example:

$$R = Af_y \tag{1.6}$$

Where:
A area of the cross-section;
f_y yield stress.

β	1,28	2,32	3,09	3,72	4,27	4,75	5,20	5,61
P_f	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}	10^{-8}

1.5 Relationship between the reliability index β and the probability of failure P_f.

The suitability of a chosen design model can be assessed by comparing results of the resistance function with test results, for example of members loaded in tension. The design model is adjusted until sufficient agreement is reached between the theoretical results and test results. When this is the case, the resistance function R is adjusted and the characteristic strength R_k can be determined, expressed in terms of the nominal value for the dimensions and the design value of the material properties. Finally, the partial factor for resistance γ_M is applied and the design value of the resistance R_d follows from:

$$R_d = \frac{R_k}{\gamma_M} \qquad (1.7)$$

The value of γ_M depends on the reliability index β, which is a measure for the probability of failure of the structure. The relationship between the value of the reliability index β and the probability of failure P_f is shown in table 1.5. EN 1990 assumes β = 3,8 for the ultimate limit state, and for a reference period of fifty years.

When the procedure described above for determining the design value of resistance is applied strictly, it will lead to a different value of γ_M for each resistance function. This is clearly inconvenient

NA in practice. Therefore, EN 1993-1-1, cl. 6.1 provides a limited number of recommended values for buildings, depending on the nature of failure:

γ_{M0} = 1,00 for cross-sections where yielding governs and therefore the yield stress is of importance in the resistance function;

γ_{M1} = 1,00 for stability of members;

γ_{M2} = 1,25 for cross-sections loaded in tension up to fracture, where fracture governs and therefore the tensile strength is of importance in the resistance function.

1.4 Design value of actions

It is important to consider not only the resistance but also the actions in order to assess the reliability of a structure. However the actions cannot be described by only a characteristic value in combination with a certain probability of exceedance. Therefore, representative values are used for the actions, see also sections 1.4.3, 1.6.4 and 1.6.6.

Not only are there several types of actions, but the action which should be taken into account depends on the location of the structural member. For a beam which supports a roof for example, snow load should be included in the combination of actions. However, for a beam which supports a floor snow load is irrelevant, and the imposed floor load should be taken into account in the combination of actions.

1.6 Example of permanent load: self-weight of the structure (columns, beams, floors and walls).

1.7 Example of variable action: snow load on a roof.

1.8 Example of an accidental action: collision by a car.

1.4.1 Action types

EN 1990 divides actions into three types, depending on their variation in time, see also *Structural basics 2* (Actions and deformations):
- permanent loads (G);
- variable actions (Q);
- accidental actions (A).

EN 1990, cl. 4.1.1 mentions seismic actions (A_E), which may be considered as accidental and/or variable actions, depending on the site location.

Permanent loads are actions which are always present, and which only vary slightly in their magnitude. The most important permanent loads are:
- self-weight of the structure (fig. 1.6) and the permanent installations (for example elevators and air-conditioning)
- pressures from soil and water (provided these pressures have a limited variation in magnitude over time, or else they have to be treated as variable action);
- pre-stress (P) due to cables or due to imposed deformations at the supports;
- imposed deformations (for example due to creep or differential settlements).

Although pre-stress is a permanent load and therefore a part of G, when considered in combinations of actions it is treated separately, designated by P. Such separation allows different partial factors to be used for actions due to pre-stress and other permanent loads.

Variable actions vary in time and in magnitude during time. An example is wind action: sometimes there is no wind, usually there is mild wind, and occasionally strong wind. Examples of variable actions are:
- snow load (fig. 1.7);
- wind action;
- thermal action due to temperature differences;
- imposed load due to people, furniture and vehicles;
- load due to storage of goods and materials;
- sometimes also water pressure.

Accidental actions have in general large and disastrous effects, although they have a small probability of occurrence. These actions occur for example due to gas explosions and collisions (fig. 1.8).

action type	indication	probability of occurrence
permanent	G	always
variable	ψQ_k, accompanying value	almost always
	Q_k, characteristic value	rarely
accidental	A	once or perhaps never

1.9 Action types and probability of occurrence.

The different types of actions each have a different nature, especially in that the probability of occurrence over time differs strongly (table 1.9). EN 1990 helps designers by answering two important questions:

- What is the characteristic value of the variable action? For example, should heavy snowfall be taken into account, or, like in The Netherlands, only a couple of centimetres of snow?
- Which actions shall be considered in the combination of actions, and to what extent? Permanent loads are always present, variable actions sometimes, and accidental actions maybe once or perhaps never.

1.4.2 Characteristic value of the variable action

It is known from both experience and specific measurements that variable actions vary in magnitude. Take for example wind action. The wind action on a building is a function of the wind speed and the shape, location and dimensions of the building. In the Netherlands for example, the average wind speed is measured at ten minute intervals at a number of meteorological stations. The probability density function is then determined based on these measurements and with this, the characteristic value of the wind action is determined. For wind, the characteristic value is set as the wind speed with an annual probability of exceedance of 0,02. This is approximately the wind speed which is on average exceeded once every fifty years in the Netherlands, which is approximately 27 m/s.

The wind speed at a certain (random) moment in time follows from the statistical distribution. This value is referred to as the accompanying value of the wind speed and is for the Netherlands approximately 11 m/s. However, during a storm the wind speed is clearly much higher than 11 m/s, so for design not only the wind speed at a certain (random) moment in time is of importance, but also the largest wind speed expected during the design working life of the structure.

1.4.3 Combinations of actions

The three types of actions – permanent loads, variable and accidental actions – occur both separately and in combination with each other. EN 1990 provides combinations of actions for the ultimate limit states (equilibrium and failure) and for the serviceability limit states (deformations and vibrations). Both are discussed here.

Permanent loads (G) are always present and are therefore included in all combinations of actions (table 1.9). The accompanying values (ψQ_k) of the variable actions are statistically speaking also always present and are therefore also included in all combinations of actions (ψ is the accompanying factor). Characteristic values of the variable actions (Q_k) and accidental actions (A) only rarely occur. The probability that two separate (independent) variable actions both reach their characteristic values at the same time ($Q_{k,1} + Q_{k,2}$) is highly unlikely. The same is true for the occurrence of an accidental action in combination with a characteristic variable action ($A + Q_k$). This means that in the design rules of EN 1990, independent characteristic values do not have to be combined.

a NA EN 1990, table A1.1 provides the recommended values for the ψ-factors for variable actions, see table 1.10. Three ψ–factors can be distinguished:

- the combination value $\psi_0 Q_k$: to be used for ultimate limit states and for irreversible serviceability limit states;
- the frequent value $\psi_1 Q_k$: to be used for ultimate limit states involving accidental actions and for reversible serviceability limit states;
- the quasi-permanent value $\psi_2 Q_k$: to be used for ultimate limit states involving accidental actions, for reversible serviceability limit states and for calculation of long-term effects.

b NA 1.10 Recommended values of ψ factors for variable actions.

action	ψ_0	ψ_1	ψ_2
imposed loads in buildings:			
– cat. A: domestic, residential areas	0,7	0,5	0,3
– cat. B: office areas	0,7	0,5	0,3
– cat. C: congregation areas	0,7	0,7	0,6
– cat. D: shopping areas	0,7	0,7	0,6
– cat. E: storage areas	1,0	0,9	0,8
– cat. F: traffic area, vehicle weight ≤ 30 kN	0,7	0,7	0,6
– cat. G: traffic area, 30 kN < vehicle weight ≤ 160 kN	0,7	0,5	0,3
– cat. H: roofs	0	0	0
snow loads:			
– Finland, Iceland, Norway, Sweden	0,70	0,50	0,20
– other CEN member states, for sites at altitude H > 1000 m	0,70	0,50	0,20
– other CEN member states, for sites at altitude H ≤1000 m	0,50	0,20	0
wind loads	0,6	0,2	0
temperature (non-fire) in buildings	0,6	0,5	0

The accompanying value ψQ_k is set as zero for some variable actions ($\psi = 0$). This means that these actions only occur in a limited number of combinations. The principle for the combination of actions is that the most unfavourable situation should be considered. Combinations with a small probability of occurrence, or with a small probability that they will be critical, may be neglected.

Ultimate limit state

The combinations of actions for persistent and transient design situations (fundamental combinations of actions) in the ultimate limit state are as follows:

$$\sum_{j\geq1}\gamma_{G,j}G_{k,j} + \gamma_P P + \gamma_{Q,1}Q_{k,1} + \sum_{i>1}\gamma_{Q,i}\psi_{0,i}Q_{k,i} \qquad (1.8)$$

Where γ_G, γ_P and γ_Q are the partial factors for the different actions. Values for γ_G and γ_Q are discussed in *Structural basics 2* (Actions and deformations), section 2.2.3. Values for γ_P are provided in the material Eurocodes, depending on the type of pre-stress.

Equation (1.8) is equation (6.10) of EN 1990. Each time one variable action ($Q_{k,1}$ for i = 1) is chosen as governing, the characteristic value of this action is used. For the other variable actions ($Q_{k,i}$ for i > 1) the combination value is used ($\psi_{0,i}Q_{k,i}$).

The less favourable of the equations (1.9) and (1.10) may be applied for the fundamental combinations of actions as an alternative to equation (1.8):

$$\sum_{j\geq 1} \gamma_{G,j} G_{k,j} + \gamma_p P + \gamma_{Q,1} \psi_{0,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \tag{1.9}$$

$$\sum_{j\geq 1} \xi_j \gamma_{G,j} G_{k,j} + \gamma_p P + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \tag{1.10}$$

a NA Equations (1.9) and (1.10) are equations (6.10a) and (6.10b) of EN 1990. Here, ξ is a reduction factor for unfavourable permanent load G with a recommended value $\xi = 0{,}85$ according to EN 1990, table A1.2(B).

In equation (1.9), only combination values for variable actions are used and therefore this equation takes less action into account ($\psi_{0,1} Q_{k,1}$ instead of $Q_{k,1}$) compared to equation (1.8). Equation (1.10) is very similar to equation (1.8), the difference between them being the lower permanent load due to the reduction factor ξ which is taken into account in equation (1.10).

b NA EN 1990, tables A1.2(A) and A1.2(B) contain the recommended values for the partial factors for actions. For the ultimate limit state concerning internal failure (or occurrence of excessive deformations) of the structure, where the resistance of the materials is governing (STR, see section 1.6.6) and pre-stress is neglected, equations (1.8) to (1.10) can be modified into:

$$1{,}35G + 1{,}5Q_{k,1} + \sum_{i>1} 1{,}5\psi_{0,i} Q_{k,i} \tag{1.11}$$

$$1{,}35G + \sum_{i\geq 1} 1{,}5\psi_{0,i} Q_{k,i} \tag{1.12}$$

$$1{,}15G + 1{,}5Q_{k,1} + \sum_{i>1} 1{,}5\psi_{0,i} Q_{k,i} \tag{1.13}$$

The values in equations (1.11) to (1.13) are partial factors for actions γ including the reduction factor ξ where appropriate. In these equations, an unfavourably acting permanent load is assumed. For favourably acting permanent loads, the factors 1,35G and 1,2G should be replaced by 0,9G.
In equations (1.11) and (1.13) a leading variable action is chosen for which the characteristic value is used. In equation (1.12) the permanent load is combined with the combination value of all variable actions. In equation (1.13) the permanent load has a lower partial factor due to the reduction factor ξ and is combined with the combination values of the variable actions. However, the combination value of one variable action is successively replaced by its characteristic value.
The combinations of actions at the ultimate limit state for accidental design situations – for example fires, explosions, impact actions and emergency repair after these incidents – are as follows:

c NA

$$\sum_{j\geq 1} G_{k,j} + P + A + (\psi_{1,1} \text{ or } \psi_{2,1}) Q_{k,1} + \sum_{i>1} \psi_{2,i} Q_{k,i} \tag{1.14}$$

This is equation (6.11b) of EN 1990. In this combination of actions, permanent loads and variable actions (G, P and Q) are combined with one accidental action (A). The choice between $\psi_{1,1}Q_{k,1}$ and $\psi_{2,1}Q_{k,1}$ depends on the accidental design situation being considered, where the factor $\psi_{1,1}Q_{k,1}$ is only of importance for wind action on the main load bearing structure. All partial factors have a value of 1,0. Neglecting pre-stress and if $\psi_{2,1}Q_{k,1}$ is used, the following combination of actions applies:

$$1{,}0G + 1{,}0A + \sum_{i \geq 1} 1{,}0\psi_{2,i}Q_{k,i} \tag{1.15}$$

For loading due to fire reference is made to EN 1991-1-2. Chapter 6 of EN 1990 also includes combinations of actions for seismic design situations. These combinations are not discussed here.

Serviceability limit state

According to EN 1990, cl. 6.5.3(2), the characteristic combination of actions (equation (6.14b) of EN 1990) should be applied for irreversible serviceability limit states:

$$\sum_{j \geq 1} G_{k,j} + P + Q_{k,1} + \sum_{i > 1} \psi_{0,1}Q_{k,1} \tag{1.16}$$

This is actually equation (1.8) with the partial factors for actions γ set to 1,0. When pre-stress (P) is neglected, this leads to:

$$1{,}0G + 1{,}0Q_{k,1} + \sum_{i > 1} 1{,}0\psi_{0,i}Q_{k,i} \tag{1.17}$$

An irreversible serviceability limit state occurs if the yield stress is exceeded somewhere in the structure so permanent deformation remains when the load is removed.

NA

Frequent combinations of actions should be applied for reversible serviceability limit states, see equation (6.15b) of EN 1990:

$$\sum_{j \geq 1} G_{k,j} + P + \psi_{1,1}Q_{k,1} + \sum_{i > 1} \psi_{2,i}Q_{k,i} \tag{1.18}$$

With partial factors equal to 1,0, and when pre-stress is neglected, this leads to:

$$1{,}0G + 1{,}0\psi_{1,1}Q_{k,1} + \sum_{i > 1} 1{,}0\psi_{2,i}Q_{k,i} \tag{1.19}$$

A reversible serviceability limit state occurs if the yield stress is not exceeded during loading so the structure recovers elastically, without any permanent deformation, after unloading.

It is not necessary to show that stresses remain below yield for the characteristic combinations of actions, because these combinations are applied to irreversible serviceability limit states.

However for the frequent combinations of actions checking for yielding should be carried out, because these combinations are valid for reversible serviceability limit states. Because of the magnitude of the ψ-factors this stress requirement is usually met, and yielding will not occur for the frequent combinations of actions.

Quasi-permanent combinations of actions also exist alongside characteristic and frequent combinations. They should be considered for the assessment of long-term effects (shrinkage and creep) and when the aesthetics of the structure are of importance:

$$\sum_{j\geq 1} G_{k,j} + P + \sum_{i\geq 1} \psi_{2,1} Q_{k,i} \qquad (1.20)$$

With partial factors equal to 1,0 and neglecting pre-stress, this leads to:

$$1,0G + \sum_{i\geq 1} 1,0\,\psi_{2,1} Q_{k,i} \qquad (1.21)$$

Shrinkage and creep are not important for steel structures at ambient temperatures.

1.4.4 Partial factors for actions

Different partial factors for actions γ are incorporated in the combinations of actions given above, see for example equations (1.11) to (1.13) and (1.15).
It is possible to determine these partial factors when the actions, the resistance, the partial factors for resistance and the reliability index β of a structure are known. The partial factors for actions have an average and a standard deviation which both depend on the action type. For practical reasons, EN 1990 only distinguishes between partial factors for permanent loads and variable actions. No distinction is made between the various variable actions.
EN 1990 uses the term 'design working life' to determine the magnitude of the actions which should be taken into account, see *Structural basics* 2 (Actions and deformations), section 2.2.4. The characteristic values of the actions only occur once during the design working life. A short design working life therefore leads to a relatively low value of an action and a long design working life leads to a relatively high value, depending on the statistical distribution of the action.

1.5 Reliability

From an economical point of view it is not sensible to design all buildings to have the same structural reliability. Society requires a much larger reliability for e.g. a nuclear power plant than for a greenhouse. To assess the term 'reliability' it is not enough to consider just the probability of failure. It is better to assess reliability in terms of the probability of failure multiplied by the damage, which occurs as a consequence of failure. This combination is defined as risk, including the risk to humans. In other words: besides the probability of failure also the consequences of failure are of importance in an assessment. For this reason, reliability is differentiated based on several consequence classes.

1.5.1 Consequence class

[a] NA It is not practical – and almost impossible – to determine the desired structural reliability for each different type of building. Therefore, EN 1990, table B1 provides a classification putting all types of buildings and parts of buildings into three consequence classes, see table 1.11. The consequence classes are identified by CC followed by a number. CC refers to consequence class. For a higher number, the consequence class is higher and it is always safe (but may not be economic) to classify a structure in a higher consequence class as provided by table 1.11.

1.5.2 Effect on combinations of actions

The consequence classes CC3, CC2 and CC1 correspond with the reliability indexes RC3, RC2 and RC1 (RC = reliability class), which determine the required reliability index β. When the reliability class (and therefore also the consequence class) is higher, the reliability index is larger.

[b] NA 1.11 Definition of consequence classes.

consequence class	description	examples of buildings and civil engineering works
CC3	• *high* consequences for loss of human life, or • *very great* economic, social or environmental consequences	• grandstands • public buildings where consequences of failure are high (e.g. a concert hall)
CC2	• *medium* consequences for loss of human life, or • *considerable* economic, social or environmental consequences	• residential buildings • office buildings • public buildings where consequences of failure are medium (e.g. an office building)
CC1	• *low* consequences for loss of human life, and • *small or negligible* economic, social or environmental consequences	• agricultural buildings where people do not normally enter (e.g. storage buildings) • greenhouses

consequence class	reliability class	reliability index β (reference period 50 year)	factor K_{FI}
CC3	RC3	4,3	1,1
CC2	RC2	3,8	1,0
CC1	RC1	3,3	0,9

1.12 Reliability index, depending on the consequence class

The classification into consequence classes and reliability classes also has implications for the magnitude of the partial factors for actions. These factors are larger when the class is higher. The partial factors for actions are multiplied by a factor K_{FI} (table 1.12).

The factor K_{FI} is only valid for the ultimate limit states concerning failure (STR, see section 1.6.6) of the structure. For an office building (CC2, RC2, $\beta = 3{,}8$ and $K_{FI} = 1{,}0$), the fundamental combinations of actions according to equations (1.11) to (1.13) are valid. These equations change for a grandstand (CC3, RC3, $\beta = 4{,}3$ and $K_{FI} = 1{,}1$) into:

a NA

$$1{,}1\cdot\left(1{,}35G + 1{,}5Q_{k,1} + \sum_{k>1} 1{,}5\psi_{0,1}Q_{k,1}\right) = 1{,}5G + 1{,}65Q_{k,1} + \sum_{k>1} 1{,}65\psi_{0,1}Q_{k,1} \quad (1.22)$$

$$1{,}1\cdot\left(1{,}35G + \sum_{k\geq 1} 1{,}5\psi_{0,1}Q_{k,1}\right) = 1{,}5G + \sum_{k\geq 1} 1{,}65\psi_{0,1}Q_{k,1} \quad (1.23)$$

$$1{,}1\cdot\left(1{,}15G + 1{,}5Q_{k,1} + \sum_{k>1} 1{,}5\psi_{0,1}Q_{k,1}\right) = 1{,}25G + 1{,}65Q_{k,1} + \sum_{k>1} 1{,}65\psi_{0,1}Q_{k,1} \quad (1.24)$$

For an industrial building with one or two floors (CC1, RC1, $\beta = 3{,}3$ and $K_{FI} = 0{,}9$) the fundamental combinations of actions become as follows:

$$0{,}9\cdot\left(1{,}35G + 1{,}5Q_{k,1} + \sum_{k>1} 1{,}5\psi_{0,1}Q_{k,1}\right) = 1{,}2G + 1{,}35Q_{k,1} + \sum_{k>1} 1{,}35\psi_{0,1}Q_{k,1} \quad (1.25)$$

$$0{,}9\cdot\left(1{,}35G + \sum_{k\geq 1} 1{,}5\psi_{0,1}Q_{k,1}\right) = 1{,}2G + \sum_{k\geq 1} 1{,}35\psi_{0,1}Q_{k,1} \quad (1.26)$$

$$0{,}9\cdot\left(1{,}15G + 1{,}5Q_{k,1} + \sum_{k>1} 1{,}5\psi_{0,1}Q_{k,1}\right) = 1{,}0G + 1{,}35Q_{k,1} + \sum_{k>1} 1{,}35\psi_{0,1}Q_{k,1} \quad (1.27)$$

b NA

1.5.3 Assessment of existing buildings

The discussions above concern the structural reliability of 'new-built' buildings. The Eurocodes do not yet cover the structural reliability of existing buildings in case of renovation, retrofitting or re-use. The assessment of existing structures often requires the application of refined methods that are beyond the scope of design codes for new structures. Methodologies appropriate for existing structures have evolved over the last 20 years in many countries, and are applied on a national level.

New European design rules are being developed to cover the assessment, re-use and retrofitting of existing structures, staying within the principles and fundamental requirements of the Eurocodes[2, 3]. These rules will bring together the different national approaches into a broadly accepted, coherent and harmonised set of rules for existing structures complementing those for the design of new structures. Currently, national legislation normally provides additional requirements for the structural safety of existing buildings.

1.6 EN 1990

EN 1990 provides the general principles and the basis of structural design. The content of EN 1990 is discussed below, following the order of its table of contents:
– general (chapter 1);
– requirements (chapter 2);
– principles of limit states design (chapter 3);
– basic variables (chapter 4);
– structural analysis and design assisted by testing (chapter 5);
– verification by the partial factor method (chapter 6);
– annexes (annex A1, A2, B, C and D).

EN 1990 should always be applied in combination with the appropriate National Annex.

1.6.1 General (chapter 1)

The scope of the code is discussed in chapter 1 of EN 1990. The code presents the general principles of structural design and states the principles and requirements for the safety, serviceability and durability of structures. It also describes the basis for design and verification of structures, and gives guidelines for structural reliability.
EN 1990 should be used in conjunction with EN 1991 to EN 1999, and can also be used for the assessment of existing structures. Chapter 1 also contains the usual Eurocode clauses concerning normative references, assumptions, distinction between principles and application rules, terms and definitions and symbols.

1.6.2 Requirements (chapter 2)

EN 1990 chapter 2 first states the basic requirements for building structures:
– a structure shall be designed and executed in such a way that it will, during its intended design working life, with an appropriate degree of reliability and in an economical way, sustain all actions and influences likely to occur during execution and use, and meet the specified serviceability requirements;
– a structure shall be designed to have adequate structural resistance, serviceability and durability;

category	design working life (years)	examples
1	10	temporary structures[a]
2	10-25	replaceable structural parts, e.g. gantry girders, bearings
3	15-30	agricultural and similar structures
4	50	building structures and other common structures
5	100	monumental building structures, bridges and other civil engineering works

a. Structures or parts of structures that can be dismantled with a view to being re-used should not be considered as temporary.

1.13 Indicative design working life.

– in case of fire, the structural resistance should be adequate for the required period of time (the so-called fire resistance, see [8]);
– potential damage shall be avoided or limited.

The reliability of structures is then discussed. The required reliability shall be achieved by design in accordance with EN 1990 to EN 1999, alongside appropriate execution and quality management measures. Different levels of reliability may be adopted for the structural resistance and the serviceability. The level of reliability that should be applied for a particular structure may be specified by classification, where the consequences of failure determine the classes (see section 1.5.1 and section 1.6.7 under annex B).

NA Chapter 2 of EN 1990 also discusses the design working life during which the structure should have the required reliability. EN 1990 distinguishes five different categories, see table 1.13. The final part of chapter 2 discusses durability and quality management.

1.6.3 Principles of limit states design (chapter 3)

EN 1990 distinguishes between ultimate and serviceability limit states in chapter 3.

• *Ultimate limit states* are defined as limit states concerning collapse or similar structural failure. They concern the safety of people and/or the safety of the structure. The following ultimate limit states should be assessed:
– loss of equilibrium of the structure considered as a rigid body;
– failure by excessive deformations, by transformation of the structure or any part of it into a mechanism, by rupture and by loss of stability;
– failure caused by fatigue or other time-dependent effects.

• *Serviceability limit states* are defined as limit states where prescribed serviceability requirements, of a structure or any part of it, are exceeded. They concern the functioning of the structure under normal use, the comfort of people and the appearance of the building. The code distinguishes between reversible and irreversible serviceability limit states, depending on whether or not the effects of acti-

ons remain when the action is removed. The following serviceability limit states should be assessed:
- deformations that affect the appearance, the comfort of users or the functioning of the structure, or deformations that cause damage to finishes or non-structural members;
- vibrations that cause discomfort to people, or that limit the functional effectiveness of the structure;
- damage that is likely to adversely affect the appearance, the durability or the functioning of the structure.

The limit states should be related to the following design situations of a structure: persistent, transient, accidental and seismic.
Persistent design situations refer to the conditions of normal use of the structure and transient design situations refer to temporary conditions such as during execution or repair. Accidental design situations refer to exceptional conditions applicable to the structure such as fire, explosion or impact. Finally, seismic design situations refer to the conditions applicable to the structure when subjected to seismic events.

A design for limit states shall always be based on the use of structural and action models for the relevant limit states. It shall be ensured that the limit states are not exceeded for all relevant design situations and load cases. Usually the method of partial factors is applied, the semi-probabilistic method (see sections 1.1 and 1.6.6). This is because it is not practical to apply either the approximate or full probabilistic methods.

1.6.4 Basic variables (chapter 4)

Chapter 4 of EN 1990 describes the basic variables to be considered for the assessment of structures, namely the actions and the environmental influences, the material and product properties, and the geometrical data.

Actions and environmental influences

Actions should be classified according to their variation in time as follows:
- permanent actions (G);
- variable actions (Q);
- accidental actions (A).

Actions should also be classified by their:
- origin: direct or indirect;
- spatial variation: fixed or free;
- nature and/or structural response: static or dynamic.

Actions should be described by one scalar quantity, which can have several representative values. The most important representative value is the characteristic value F_k of an action. This can be the mean value, an upper or lower value, or a nominal value. The value which is applied for the characteristic action depends on the action type.

- For permanent actions the variability can be considered as small and only one single value G_k may be used for the characteristic permanent action. This value should be determined based on the nominal dimensions and the mean densities. Pre-stressing (P) should be classified as a permanent action caused by either controlled forces and/or controlled deformations imposed on a structure. For the ultimate limit states, an average pre-stress P_k can be assumed as a characteristic value.
- For variable actions the characteristic value Q_k shall in general be an upper bound value with an intended probability of not being exceeded during a specific reference period.
- For accidental actions, the design value A_d should be specified for each individual project.
- For seismic actions, the design value A_{Ed} should be determined from the characteristic value A_{Ek}, or specified for each individual project.

Other representative values of a variable action are:

– the combination value $\psi_0 Q_k$: used for ultimate limit states and for irreversible serviceability limit states;
– the frequent value $\psi_1 Q_k$: used for ultimate limit states involving accidental actions and for reversible serviceability limit states;
– the quasi-permanent value $\psi_2 Q_k$: used for ultimate limit states involving accidental actions, for reversible serviceability limit states and for calculation of long-term effects.

Chapter 4 of EN 1990 also discusses fatigue actions, dynamic actions, geotechnical actions and environmental influences. These aspects are not discussed here.

Material and product properties

Properties of materials or products should be represented by characteristic values. In general, a low value of a material or product property is unfavourable so the characteristic value should be defined as the 5% fractile value. As an alternative – which is especially of importance for steel as a material – nominal values may be applied as characteristic values.
The structural stiffness parameters – Young's modulus, Poisson's ratio – and thermal expansion coefficients should be represented by a mean value. These values are given in EN 1992 to EN 1999, notably for steel in EN 1993.

Geometrical data

Geometrical data of a structure – such as length and cross-section dimensions – shall be represented by their characteristic values or directly by their design values. The dimensions specified in the design may be taken as characteristic values. Imperfections that should be taken into account are discussed in EN 1992 to EN 1999, notably for steel structures in EN 1993.