Methodology of probabilistic analysis for building structures of nuclear facilities

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Abstract

The probabilistic analysis of building structures has not been generally incorporated in the PSA (probabilistic safety analysis) of nuclear facilities so far. The PSA normally considers the systems and components related to the safety of nuclear facilities. In order to introduce structural failure into the PSA it is necessary to assess the probability of structural failure. This probability depends on the failure modes that have occurred. The Eurocode EN 1990 "Basis of Structural Design" allows designing special construction works (e.g. nuclear installations, dams, bridges, etc.) based on probabilistic methods if the relevant authority gives permission. A methodology for implementing a probabilistic approach for the design and assessment of NPP (nuclear power plant) structures has been developed. Particular aspects have been implicitly considered and addressed. These are, for example, the higher reliability class of nuclear facility buildings in comparison with the conventional ones or specific requirements on ultimate resistance and serviceability properties.

Keywords: Reliability, Structures, Nuclear, Failure; Design.

1. Introduction

For the safety assessment of components of nuclear power plants the probabilistic safety analysis (PSA) has been elaborated and widely used during the last decades. This method is based on an event tree analysis taking into account the failure probability of each component. Generally, the buildings or building structures of nuclear power plants are not considered in this analysis. This has already been recognised [1-2]. The only attempt to take into account the structural reliability in a simple manner was undertaken in the seismic PSA, for example in [3]. It was proposed to use the fragility curve, as it is done for systems, structures and components (SSC) of NPP. The probability of building structure failure was introduced in the event tree analysis with some value, independent of the structural failure mode.

2. Development of Structural Reliability and Building Codes

1.1 Short Review

Already in the 1920s, Meyer proposed to use the probabilistic concept for structural safety [4]. In civil engineering, the design concept, which considers the structural reliability, was developed mostly in the 1970s and 1980s (e.g. [5-10]). This development of structural reliability led to the evolution of building standards in Europe. As a result, a new homogenous standard "Eurocode" (EN 1990 – EN 1999) was elaborated. This standard now replaces the "old" national standards in all EU countries.

The Eurocode proposes the target reliability of building structures. To achieve this reliability, the concept of partial safety factors has been realised. This is a so-called semi-probabilistic approach. To determine these partial safety factors for different design situations a lot of pre-normative research has been performed during last decades. Most of this research uses the well-developed probabilistic methods with deep mathematical background (e.g. [11-12]).

In parallel, the Joint Committee on Structural Safety (JCSS) has initiated the development of the full probabilistic code. The first part of the "JCSS Probabilistic Model Code" [13] was ready in 2000. The updating process on this code is continuing.

1.2 EN 1990 "Basis of Structural Design"

In 1975, the Commission of the European Community initiated the Eurocode programme in the field of construction. The objective of the programme was the harmonisation of technical specifications. Within this programme, the Commission took the initiative to establish a set of harmonised technical rules for the design of construction works which, in a first stage, would serve as an alternative to the national rules in the Member States and, ultimately, would replace them.

EN 1990 "Basis of Structural Design" [14] describes the principles and requirements for the safety, serviceability and durability of structures. It is based on the limit state concept used in conjunction with a partial factor method. For the design of new structures, EN 1990 is intended to be used, for direct application, together with Eurocodes EN 1991 to 1999. EN 1990 also gives guidelines for the aspects of structural reliability relating to safety, serviceability and durability for designs not covered by EN 1991 to EN 1999. EN 1990 may be used, when relevant, as a guidance document for the design of structures outside the scope of the Eurocodes, for assessing other actions and their combinations, for modelling other material and structural behaviour, and for assessing numerical values of the reliability format. In [14] it is mentioned, that for the design of special construction works (*e.g.* nuclear installations, dams, etc.), other provisions than those in EN 1990 to EN 1999 might be necessary.

The very important particularity of EN 1990 is the given reliability management. According to [14] the choice of the levels of reliability for a particular structure should take account of the relevant factors, including:

- the possible cause and /or mode of attaining a limit state;
- the possible consequences of failure in terms of risk to life, injury, potential economic losses;
- public aversion to failure;
- the expense and procedures necessary to reduce the risk of failure.

For the purpose of reliability differentiation, consequences classes (CC) are given in Annex B "Management of Structural Reliability for Construction Works", Table B1 by considering the consequences of failure or malfunction of the structure (see Figure 1.).

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

Figure 1. Definition of consequences classes from [14].

Depending on the structural form and decisions made during design, particular members of the structure may be designated in the same, higher or lower consequences class than for the entire structure.

The reliability classes (RC) are defined by the reliability index β concept. Three reliability classes RC1, RC2 and RC3 can be associated with the three consequences classes CC1, CC2 and CC3. Table B2 of Annex B gives recommended minimum values for the reliability index associated with reliability classes (see Figure 2.) for the ultimate limit state.

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

Figure 2. Recommended minimum values for reliability index β (ultimate limit state) from [14].

Table C2 of Annex C "Basis for Partial Factor Design and Reliability Analysis" [14] gives the target values for the reliability index β for various design situations, and for reference periods of 1 year and 50 years. These values correspond to levels of safety for reliability class RC2 (see Figure 3.).

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.

Figure 3. Target values for reliability index β for RC2 class from [14].

The semi-probabilistic concept is based on the partial factor method. By using this method it shall be verified that, in all relevant design situations, no relevant limit state is exceeded when design values for actions or effects of actions and resistances are used in the design models. Design values are to be obtained by using the characteristic or other representative values, in combination with partial and other factors as defined in [14] and EN 1991 to EN 1999. These factors, however, are defined for the reliability class RC2.

If the structure belongs to another reliability class, the partial factors should be redefined. This can be done by the methods given in Annex C of EN 1990. This annex provides information on the structural reliability methods, on the application of the reliability-based method to determine by calibration design values and/or partial factors and on the design verification formats in the Eurocodes.

The semi-probabilistic concept (partial factors method) corresponds to the Level I of reliability methods. The probabilistic calibration procedures for partial factors are subdivided into two main classes: full probabilistic methods (Level III) and first order reliability methods (FORM) (Level II). In both the Level II and the Level III methods, the measure of reliability is 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 is to be considered to be unsafe.

In the Level II procedures, the EN 1990 uses an alternative measure of reliability, the so-called reliability index β, which is related to P_f (probability if failure) by eq. (1):

$$P_f = \Phi\ (-\beta) \tag{1}$$

where Φ is the cumulative distribution function of the standardised normal distribution. The relation between P_f and β is given in Figure 4.

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

Figure 4. Relation between β and P_f from [17], Table C1.

To define the probability of failure the performance function g (also called limit state function) is introduced. If R is the resistance and E the effect of actions, the limit state function g is given by eq. (2):

$$g = R - E \tag{2}$$

where R, E and g are random variables.

The limit state is reached if R is equal to E. The probability of failure (probability of exceeding of the limit state) can be then represented by eq. (3):

$$P_f = P(g \leq 0) = P(E \geq R) \tag{3}$$

Accordingly the probability of survival (limit state is not exceeded) can be expressed by eq. (4):

$$P_S = P(g \geq 0) = P(R \geq E) \tag{4}$$

The procedures of Level II and Level III can be used instead of the partial factors method (semi-probabilistic approach) if the full probabilistic approach is used for the design.

EN 1990 makes a distinction between ultimate limit states and serviceability limit states. The limit states that concern the safety of people, and/or the safety of the structure are to be classified as ultimate limit states. The following ultimate limit states shall be verified where they are relevant:

- loss of equilibrium of the structure or any part of it, considered as a rigid body;
- failure by excessive deformation, transformation of the structure or any part of it into a mechanism, rupture, loss of stability of the structure or any part of it, including supports and foundations;
- failure caused by fatigue or other time-dependent effects.

The limit states that concern the functioning of the structure or structural members under normal use, the comfort of people or the appearance of the construction works are to be classified as serviceability limit states. The verification of serviceability limit states should be based on criteria concerning deformations, vibrations and damage that affect the appearance, the comfort of users, the durability or the functioning of the structure.

Limit states are related to design situations. Design situations are classified as persistent, transient, accidental or seismic. Actions are classified as permanent, variable or accidental. The main representative value of action is the characteristic one. The characteristic value multiplied by the partial safety factor leads to the design value. For resistance (e.g. material properties) design values are defined from the appropriate characteristic values divided by the partial safety factor for resistance. At Level II procedures, the design values of actions and resistance are determined directly as results of FORM calculations.

3. Structural Reliability Concept for Nuclear Facility Buildings

The above-mentioned reliability concept of EN 1990 can be adopted for the design and assessment of the nuclear facility structures. This objective can be achieved if some specific design aspects of NPPs are taken into account.

3.1 Differentiation of Reliability Index β (or Probability of Failure)

EN 1990 distinguishes three reliability classes (RC) which are associated with the three consequences classes (CC). The partial safety factors in the Eurocodes are given only for reliability class RC2. Building structures of NPPs that are important for safety may belong to higher reliability class e.g. RC3. In this case, all partial safety factors at Level I procedures have to be recalculated because the reliability index β associated with RC3 (see Figure 2.) is higher. For this purpose the analysis by means of probabilistic methods at Level II or at Level III (see above) has to be performed. The nuclear facility structures can even belong to a reliability class higher than RC3. In this case the reliability index β should be firstly determined or alternatively should be set by the authorities.

3.2 Specific Requirements on the Limit States

According to EN 1990 it shall be verified that no limit state is exceeded with a given probability (see above, Figure 2.). The requirements on the exceeding of relevant limit state (i.e. probability of failure) for nuclear structures can deviate from the ones for conventional (i.e. non-nuclear) structures. This is due to the fact that the structures of NPPs that are important for safety belong to a higher consequences class than the conventional ones. Two examples can be mentioned here.

Normally the limitation of the induced vibrations and the limitation of the crack width in the reinforced concrete structures are referred to serviceability limit state. By the NPP structures that are important for safety these requirements are to be attributed to the ultimate limit state. In this case the structural bearing capacity will not be exhausted if the relevant limit state is reached. Nevertheless, the appropriate probability of failure will be exceeded for the specific failure scenario, as for example the failure of nuclear systems and components important for safety due to induced vibrations or radioactive release due to a large crack. The attempt to take this problem into account was already made in [15]. The crack width limitation in the pre-stressed concrete containment was proposed to be attributed to serviceability limit state. However, at the same time the probability of exceeding this limit state was set at about 10^{-8} to 10^{-9}. These values are considerably smaller than the values of EN 1990 [14] (see Figure 3.) for serviceability limit states. Correspondingly, the values of safety factors in [15] were about 1, 7. This is higher than in [14]. This example shows that the need for more stringent requirements on exceeding the serviceability limit states for nuclear structures was recognised already in the past. Therefore, in the reliability analysis of nuclear structures, all possible failure scenarios should be considered regarding the associated failure consequences. Afterwards, the appropriate probability of failure should be adjusted for all relevant limit states. This adjustment would be based on the severity of failure consequences, which are specific for nuclear facilities structures. The reliability values from conventional building design cannot be directly used in NPP design.

3.3 Additional Actions

The EN 1991 [16] provides information on different actions which can affect the structures. On the other hand EN 1990 [14] is applicable to the design of structures where other materials or other actions outside the scope of the Eurocodes are involved. This means that for the actions which are not described in the Eurocodes, the approach of EN 1990 can be used. For example, characteristic and design values, combination factors (at Level I approach), or probability distributions (at Level III approach) can be defined in this way. If this estimation is not possible due to a lack of statistical information, the nominal values (e.g. deterministic) for actions can be used for the design and assessment of nuclear facility structures. The following actions, which are not provided in Eurocode, are normally considered in the design of nuclear facilities:

- tornado;
- tsunami;
- aircraft crash;
- blast wave.

These actions belong to the group of accidental actions. For these actions the probability of occurrence or occurrence rate should be determined for the reliability analysis. For tornados and tsunamis, such determination can be performed on the basis of climate data. For aircraft crashes and blast waves, such estimation can be performed on the basis of statistical data of accidents.

3.4 Particular Failure Modes

Due to the consideration of specific actions (see above) it is necessary to investigate, whether the additional failure modes that are not provided in the Eurocodes can oocur. For example such failure modes as spalling or scabbing (spalling of concrete on the inner side of a concrete structure due to an impact load) can occur as the result of an aircraft crash. Normally these failure modes are not taken into account by conventional design. This particularity shall be implicitly considered by the specification of ultimate and serviceability limit states.

For each action, appropriate to the nuclear facility design (especially accidental actions, see chapter 3.3), all possible failure modes for the relevant structure should be identified and estimated. For example, for the structural analysis of a massive reinforced concrete structure of an NPP under the action of an aircraft crash, the following failure modes should be taken into account: loss of equilibrium, flexure failure (global or local), punching, perforation or penetration, scabbing, spalling, induced vibrations. Some of these failure modes are not considered in the conventional design and not given in the Eurocodes.

3.5 Particular Failure Scenarios and Consequence Scenarios

The failure scenarios shall be defined for all nuclear facility buildings (or structures) and evaluated regarding the possible consequences (or damage cases). Only then can the probability of exceeding of the relevant limit state be determined. For example, the crash of a heavy crane in the reactor building due to an earthquake or aircraft crash can be considered as a failure scenario specific to NPPs. The associated consequences should be evaluated for the relevant situation.

The failure consequences of the NPP structures are normally different from the ones of conventional buildings, for example due to a radiological release. This should be implicitly considered when determining the reliability of the nuclear facility structures. This procedure is provided in the new European Norm EN 1990 [14].

3.6 Stochastic Characteristics of Design Parameters

Due to specific requirements on the nuclear facility buildings the design parameters can deviate from the ones of conventional construction works regarding their stochastic background. This effect has an influence on the reliability level of structures.

The NPP building structures are usually very massive (e.g. reactor building). Therefore, one of the most important design parameters is the compressive strength of concrete. Due to the large volume of concrete needed for the construction of NPP buildings and special requirements on building materials, the statistical properties of concrete for NPP structures may differ from the ones for conventional construction work. The concrete can be delivered from different ready-mixed concrete plants. Therefore, the dispersion of the compressive strength can increase due to different types of concrete components which the ready-mixed concrete plant uses (e.g. Portland cement, fly ash, aggregate and chemical admixtures). Due to specific procedures on the construction site e.g. pouring, hardening, curing and geometrical difference of structures (e.g. beams, plates, membranes, columns, walls, floors), the dispersion of compressive strength can increase even more. As this effect has an influence on the reliability of NPP buildings, the partial factor for concrete (at Level I approach) should be recalculated. This safety factor can be higher than the value of

Eurocode. At Level II or Level III methods, the appropriate statistical concrete data for NPP structures may be directly used in the reliability analysis.

The control of the compressive strength of the concrete should give assurance of the concrete quality by means of conformity control and identity testing according to European Norm EN 206 [17]. The code EN 1990 [14] gives a possibility to perform the design based on a combination of tests and calculations (see Annex D “Design assisted by testing”). If the results of these tests for the concrete used in European NPPs were available, it would be possible to investigate the variability of the compressive strength of the concrete with respect to structural reliability.

4. Aircraft Crash as Example of Particular Action on NPP

4.1 Short Historic Review

The aircraft crash is considered as the impact action which belongs to the group of accidental actions. Only accidental aircraft crash is mentioned here. The terroristic aircraft crash impact is out of scope of these investigations.

The aircraft impact of a commercial airliner on the structures of an NPP was first considered in [18]. Afterwards the impact of a military aircraft on an NPP was investigated in Germany due to high crash rate of the NATO jets [19-20]. The German approach was confirmed by the large-scale experiment with the Phantom F-4 military jet in USA / Japan [21].

4.2 Impact Parameters

On the basis of statistical data of accidents with military jets in Germany, the impact velocity was determined to be 215 m/s. The impact angle can vary between 0^o and 90^o to the structure surface. In [18] the commercial airplane Being B707 was investigated with the impact velocity of 100 m/s. As the most unfavorable impact angle for the structure would be normal to the surface, only this case was considered.

4.3 Structural Behaviour under Impact

Due to the massive structures of NPP buildings the target structure is normally considered to be a stiff one. In contrast the aircraft is presented as a soft body. However the aircraft consists additionally of some hard parts, for example flap tracks, engine shafts or the components of landing gears. The action of the whole aircraft can be attributed to the soft impact. This means that the aircraft would be deformed and collapsed on the surface of the building. The action can be represented by means of a load-time-function, proposed in [18], see eq. (5)-(6):

$$P[t] = P_b[x(t)] + \mu[x(t)] \cdot v^2[x(t)] \tag{5}$$

$$dV[x(t)] = -P_b[x(t)] / M_r[x(t)] \tag{6}$$

where:

- P is the impact load,
- t is time from the beginning of the impact,
- $x(t)$ is the distance from the nose of the aircraft,
- P_b is load necessary to crush or deform the fuselage (crushing force),
- $\mu(x)$ is the mass of the aircraft per unit length,
- v is the velocity of the uncrushed portion the aircraft at time t,
- M_r is the mass of the uncrushed portion the aircraft at time t,
- v_0 is the initial impact velocity.

Based on this approach, the load-time-function was determined for the military jet in Germany and introduced in the safety standard [20] for the design of building structures and safety systems of NPPs (see Figure 5.).

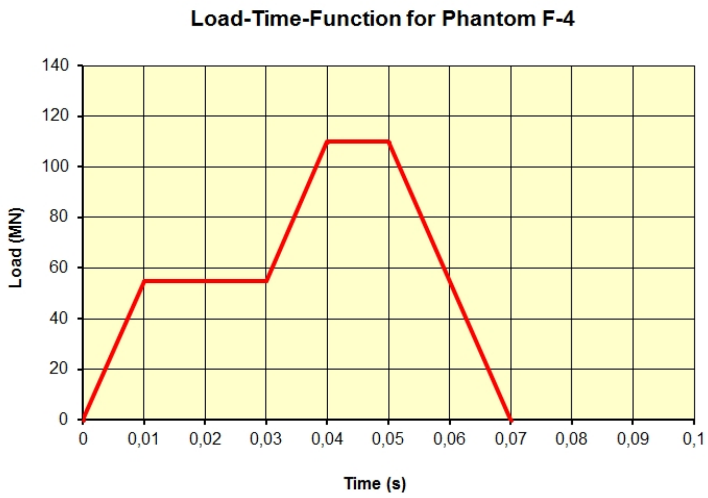

Figure 5. Load-time-function for military jet from [23].

4.4 Structural Failure Modes for Aircraft Impact

As mentioned in Chapter 3.4, the following failure modes can occur due to the action of an aircraft crash:

- loss of equilibrium,
- flexure failure (global or local),
- punching,
- perforation or penetration,
- scabbing, spalling,
- induced vibrations.

The loss of equilibrium, flexure failure, punching and induced vibrations are caused by the effect of global loading due to the whole aircraft. For the verification of appropriate limit states, the load-time-function method (see above) may be used. The perforation or penetration, scabbing/spalling or local punching are caused by the effect of local loading, mostly due to the compact parts of the aircraft (see above).

The failure mode "perforation" is considered here as an example. The probability of exceeding the appropriate limit state is calculated by means of stochastic simulations. As a projectile, a steel cylinder is considered, whose weight is 500 kg and diameter 0.2 m. This cylinder impacts the reinforced concrete wall. The CEA-EDF (French Atomic Energy und Électricité de France) approach defines a perforation thickness (minimal concrete thickness to prevent the perforation of the projectile through the wall) by means of eq. (7):

$$t_p = 0.82(f_c)^{-3/8}(\rho_c)^{-1/8}(W/d)^{0.5}V^{3/4} \tag{7}$$

where:

- W is the mass of the projectile,
- d is the diameter of the projectile,
- V is the impact velocity of the projectile (is set at 170 m/s),
- f_c is the compressive strength of the concrete,
- ρ_c is the density of reinforced concrete.

The compressive strength of the concrete f_c follows the lognormal distribution. The density of the reinforced concrete ρ_c follows the normal distribution. The concrete class C30/37 is taken for the investigation. According to [22] the characteristic value (5%-quantile) of compressive strength of the concrete f_{ck} = 30 MPa and the mean value f_{cm} = 38 MPa. The log-normal distribution of variable x with the mean value m_x, standard deviation σ_x and coefficient of variation V_x has the following form:

$$P(x) = \Phi\left[\frac{\ln x - A}{B}\right] \tag{8}$$

where:

- $Ø$ is the function of the standardised normal distribution,
- A, B are the parameters of the log-normal distribution.

$$A = \ln(m_x) - \frac{B^2}{2} \tag{9}$$

$$B = \sqrt{\ln(1 + V_x^2} \tag{10}$$

The inverse function of the log-normal distribution is:

$$x(P) = \exp\left[A + B\Phi^{-1}(P)\right] \tag{11}$$

The parameters A and B are determined under the condition that quantile $x(P)$ is equal to the characteristic value and that the probability P is equal to 0.05. The mean value and the standard deviation of the reinforced concrete density have been set at m_ρ = 2500 kg/m^3 and σ_ρ = 120 kg/m^3.

Therefore, the coefficient of variation is V_ρ = 0,048. The normal distribution has the following form:

$$P(x) = \Phi\left[\frac{x - m_\rho}{\sigma_\rho}\right] \tag{12}$$

The inverse function of the normal distribution is:

$$x(P) = \left[m_\rho + \sigma_\rho \Phi^{-1}(P)\right] \tag{13}$$

The probability P was computed by random number generator. The compressive strength of the concrete f_c and the density of the reinforced concrete ρ_c were defined by eq. (11, 13). Afterwards, the perforation thickness was computed by eq. (7) and compared with the given wall thickness t. 100.000 stochastic simulations were performed. The number of stochastic events, in which the perforation thickness t_p is smaller than a given wall thickness t was counted as N_s. The ratio N_s / N represents the probability of an event in which the projectile does not perforate the wall: $P_s = N_s / N$. Accordingly $P_f = 1 - N_s / N$ represents the probability of event, in which the projectile perforates the wall (probability of failure). This procedure was repeated for different wall thicknesses from 0.7 m to 1.5 m. The results are shown in Figure 6.

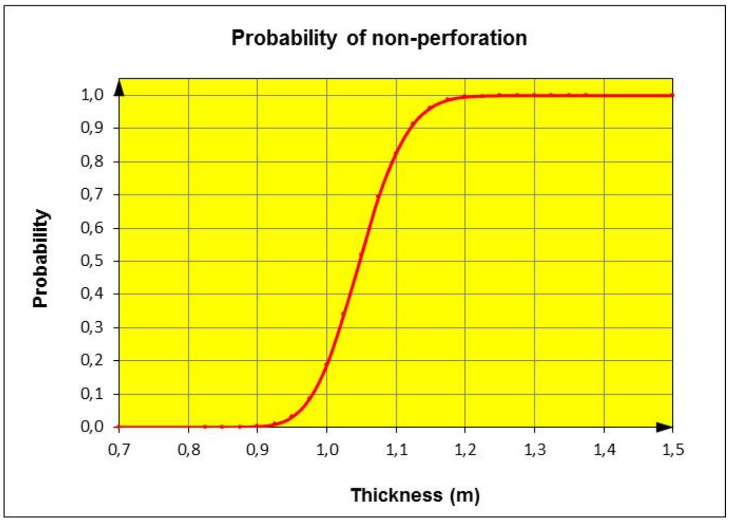

Figure 6. Probability of non-perforation of the concrete wall.

5. Summary

The probabilistic analysis of building structures can be taken into account in the PSA of nuclear facilities. This is possible following the provisions of EN 1990. Both the semi-probabilistic approach and full probabilistic one can be adopted in the design of NPP structures. The attempt is made to develop a methodology for this implementation. The particularities of NPPs which are to be implicitly considered are explained. The example of the action "aircraft crash" and the specific failure mode "perforation" are given.

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