

Application of Probabilistic Methods to the Structural Integrity Analysis of RBMK Reactor Critical Structures

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1. Introduction

One of the main requirements for building structures and components of nuclear power plants is that during extreme internal and/or external loading the structural integrity of the buildings and components installed in the building should be retained. Analytical solutions and sophisticated numerical models are used to evaluate the structural integrity of these structures. In case of dynamic loading, impact energy is transformed into the energy of plastic deformation of the structures. The transient behaviour of the structures during transient loading is a complex phenomenon due to various factors such as inertia effects, large deformations, and inelastic behaviour. It is, thus, not possible to obtain analytical solutions for general cases; so sophisticated numerical models are necessary for analysis. The finite element method (FEM) has been used extensively to simulate many applications in structural dynamics (Nagel, 2005; Aljawi, 2002; M. Yamashita, 2005, Olabi, 2008, Lee, 2004). Finite element codes are able to accurately model the plastic deformation via bending, compression or full collapse of the structures. After the terrorist attacks in New York and Washington D. C. using commercial airliners, the structural integrity assessment of civil airplane crashes into civil structures has become very important. Since these attacks, researchers from many countries have simulated aircraft crash into building structures. The FEM methodology mainly was used for the aircraft crash analysis (Bossak, 2003, Brinkmann, 2006, Dundulis, 2007a, Muragishi, 2001, Ramalingam, 2002).

Due to the tendency for increased nuclear safety, the analysis of dynamic loading shall demand multidisciplinary optimization of the methods used. However, simulation-based multidisciplinary optimization generates deterministic optimum design, which are frequently pushed to the limits of design constraint boundaries, leaving little or no room for tolerances (uncertainty) in modelling, simulation uncertainties, and manufacturing imperfections. Consequently, deterministic optimum designs that are obtained without consideration of uncertainty may result in unreliable designs, indicating the need for Reliability-Based Design Optimization (Youn, 2004).

The selection of the numerical simulation methods, preparation of models for analysis, evaluation of the material properties and loads are very important for structural integrity analysis of critical nuclear power plant structures subjected to either internal or external

extreme loading events. It is known that every material parameter is uncertain. The uncertainty of strength parameters is especially characteristic for concrete material. In order to ensure that buildings are reliable and safe in case of accidental transient loading, it is very important to evaluate uncertainty associated with loads, material properties, geometrical parameters, boundaries and other parameters. Therefore, it is necessary to account for the uncertainty in these quantities when performing a structural integrity evaluation (Podula, 2006). This can be accomplished through probabilistic analyses to see if a combination of values of relevant parameters could lead to failure and to determine the probability of failure (Alzbutas, 2003, Lyle, 2007). Therefore, a probability-based structural integrity analysis was performed as the integration of deterministic and probabilistic methods using existing state-of-the-art software for both the whipping pipe event and the aircraft crash event. The methodology of the probability-based structural integrity analysis that integrates deterministic and probabilistic methods is explained in this chapter. The application of this methodology to two Ignalina Nuclear power plants (NPP) postulated accidents is presented. Ignalina NPP that have a RBMK type reactor are quite different in comparison to power plants with PWR or BWR types reactors. Several events have been identified for these plants that can compromise the integrity of critical structural components. The Chernobyl RBMK reactor accident is the most serious accident in the history of the nuclear industry. Typically, RBMK reactors do not possess the conventional containment structure. Only the Ignalina NPP contains two RBMK 1500 reactors, which is the most advanced version of the RBMK reactor, and has a pressure suppression type confinement, which is referred to as the Accident Localization System (ALS) (Almenas, 1998). However the ALS encloses only about 65% of the entire cooling circuit. It does not enclose the sections of piping most vulnerable to rupture in case of the dangerous loss-of-coolant accident. Ruptured piping can lead to a whipping pipe impacting onto the inside of a critical exterior wall.

The finite element method is used for deterministic strength analysis of building and piping. The deterministic finite element software NEPTUNE (Kulak, 1988) was used here for structural integrity analysis. This software can analyze the transient structural response of the concrete and steel structures, which undergo large displacements and nonlinear material response during transient loading, including object impact onto the structures.

The Monte Carlo Simulation, First Order Reliability Method, and the combined Monte Carlo Simulation and Response Surface method were used for the probabilistic analyses of failure of the structures in case of the transient loading. The ProFES (Cesare, 1999) software was used for the probabilistic analysis of structural failure. ProFES is a probabilistic analysis system that allows performing probabilistic finite element analysis in a 3D environment that is similar to modern deterministic finite element analysis.

The developed probability-based approach (Kulak et al., 2003, Alzbutas et al., 2004) was used to analyze failures of RBMK reactor critical structures of the Ignalina NPP. This methodology was applied to the analysis of a group-distribution-header pipe whip, which results from a guillotine break, and subsequent impact to the adjacent walls (Alzbutas et al., 2003, Dundulis et al., 2005). This is a postulated accident for the Ignalina NPP RBMK-1500 reactors. Also, the probabilistic analysis of building failure was performed for an external event induced accident, that is, impact by a commercial aircraft (Dundulis et al., 2007c).

2. Probabilistic Analysis Methods

Probabilistic methods provide a means to assess the affect of design uncertainties and manufacturing/construction tolerances on the reliability and performance of structures. The proposed probability-based approach for integrated analysis of structure failure includes the use of several methods, which are combined to study the problem. In this chapter, the Monte Carlo Simulation, First Order Reliability Method, and the combined Monte Carlo Simulation and Response Surface method were introduced for the probabilistic analyses of failure of the structures subjected to severe transient loading.

2.1 Monte Carlo Simulation Method

The Monte Carlo Simulation (MCS) is a method, which is suitable for simulations that execute relatively quickly (e.g. simple linear finite element models or closed form expressions), because it typically requires a large number of response variables and limit state function evaluations. The precision of the evaluations depends on the number of evaluations and the chosen value of the failure probability. MCS method is mainly used to study the initial estimates and sensitivity of the response variables as well as the effect of uncertainties of system properties and uncertain model parameters to the probability of limit states (e.g. some failure states). Uncertainties in numerical values are modelled as random variables with specified characteristics. Confidence ranges and subjective probability distribution describe the state of knowledge about all uncertain parameters.

Initially using the MCS method, sets of random variables (i.e., different combinations of these random variables) are generated, and then each set is used as input for separate deterministic runs. The values of the random variables are based on their individual probability density functions. Estimates of probabilities are determined by a simple statistical analysis of the simulation results. Therefore, the most important random variables and limit states for further analysis may be selected according to the results of MCS. The aim of such initial analysis is to identify and at least roughly quantify the response of all potentially important uncertain parameters. Least important parameters are then discarded in subsequent analysis and this provides a substantial computational cost savings.

2.2 First Order Reliability Method

The First Order Reliability Method (FORM) is an approximation method that estimates the probability of an event under consideration—typically named a “failure”. This method redefines the limit state in terms of the input variables and searches the point for the combination of values that is most likely to cause failure. This point is often referred to as the design point or the most probable point. The method then fits a linear surface at the most probable point and uses this surface (along with transformations for any non-normal random variables) to compute probabilities.

FORM is characterized by the iterative, linear (hence the term ‘first-order’) approximation to the performance function. It uses analytical schemes to approximate the failure probability and is reported to be computationally very efficient compared to MCS, especially for scenarios corresponding to low probabilities of failure (Bjæger 1990). Also, FORM is the preferred method for evaluating small probabilities because for the same precision as MCS it often requires the least number of deterministic (finite element) model runs, which is very important when large FE models are used. With FORM, the computational effort is

proportional to the number of random variables and limit states. FORM is more efficient than MCS in terms of computing time and accurate results can be obtained even when the failure probability is low. All the random parameters must, however, be continuous and large errors can also result if there are local minima in the limit state or high non-linearity. Despite these limitations, FORM is the most popular reliability analysis method; it can be easily extended to non-linear limit states; and it has a reasonable balance between ease of use and accuracy.

2.3 Response Surface/Monte Carlo Simulation Method

The hybrid Response Surface/Monte Carlo Simulation (RS/MCS) method is used to determine a polynomial function, which expresses the failure probability. The Response Surface (RS) method is similar to the FORM described above. However, while the FORM deals directly with the performance function, the RS method involves approximating the original, complicated system performance function with a simpler, more computationally tractable model. This approximation typically takes the form of a first or second order polynomial:

$$Y(x) = Y(x_1, \dots, x_n) \sim a_0 + a_1 x_1 + \dots + a_n x_n + a_{n+1} x_1^2 + \dots + a_{2n} x_n^2 + a_{2n+1} x_1 x_2 + \dots \quad (1)$$

Determination of the constants is accomplished through a linear regression about some nominal value, typically the mean. Using this performance function the probability of failure can be estimated using MCS. Thus, this fitted response-surface is used as a replacement or proxy for the deterministic model with initial parameters, and all inferences related to probabilistic analysis for the original model are derived from this fitted model (Khuri, 1987). The hybrid RS/MCS method is proposed to be used in order to study the relation between failure probability and parameters of initial model.

3. Finite Element Modelling of the Building Structures and Components

Sophisticated analysis of complex structures is possible using powerful numerical techniques on modern computer workstations. This may aid in a better understanding of the behaviour of the structures under transient loads. The most general approach for numerical analysis is the finite element method. The finite element method is a numerical analysis technique used by engineers, scientists, and mathematicians to obtain solutions to the differential equations that describe, or approximately describe a wide variety of physical (and non-physical) problems. Physical problems range in diversity from solid, fluid and soil mechanics, to electromagnetism or dynamics.

The underlying premise of the method states that a complicated domain can be sub-divided into a series of smaller regions in which the differential equations are approximately solved. By assembling the set of equations for each region, the behaviour over the entire problem domain is determined. Each region is referred to as an element and the process of subdividing a domain into a finite number of elements is referred to as discretization. Elements are connected at specific points, called nodes, and the assembly process requires that the solution be continuous along common boundaries of adjacent elements.

The finite element method is used for deterministic strength analysis of building structures and components. In this work, the deterministic finite element software NEPTUNE (Kulak, 1988) was used for structural integrity analysis.

The NEPTUNE code is based upon the central difference explicit integrator. Thus, the code does not employ stiffness or flexibility matrices but is based upon a nonlinear internal nodal force vector. This approach is ideal for transient, nonlinear analyses in which metals are deforming in an elastoplastic mode, concrete is cracking/crushing and contact impact is taking place. When individual elements reach a failed state, their contributions to the internal nodal force vector is reduced to zero and there is no change required to the solution algorithm.

The central difference integrator is used to solve the equations of motions. This integrator is well suited for the treatment of transient (short duration) problems in which the variation of element eigenvalues over the mesh is not large. The acceleration, velocity, and displacement are obtained from central difference formulas:

$$\ddot{u}_{ii}(n) = (f_{ii}^{ext}(n) - f_{ii}^{int}(n)) / m_{ii}(n), \text{ no sum} \tag{2}$$

$$\dot{u}_{ii}(n+1/2) = \dot{u}_{ii}(n-1/2) + \Delta t \ddot{u}_{ii}(n) \tag{3}$$

$$u_{ii}(n+1) = u_{ii}(n) + \Delta t \dot{u}_{ii}(n+1/2) \tag{4}$$

where "i" refers to the coordinate direction (x, y, z), "I" refers to the node number, and "n" is the time step number.

The analyses examples of the building structures and piping systems are presented in this section. The following section describes the nonlinear finite elements used to model building and piping components.

3.1 Finite elements for modeling reinforced concrete

The walls of the building structures are composed of several structural components: concrete, reinforcing bars and embedded steel-frame columns. Several types of elements are needed to properly model this complex wall structure. The wall of the building was modelled using an enhanced version of the four-node quadrilateral plate element developed by Belytschko, et al. (1984) for metallic structures. The formulation of this element is based upon the Mindlin theory of plates and uses a velocity strain formulation. The element was further developed by Kulak and Fiala (1989) and Kulak et al. (1997) by incorporating the features to represent concrete and reinforcing steel (Fig. 1). Subsequently, additional failure criteria were added, and this enabled the modified element to model concrete cracking, reinforcing bar failure and gross transverse failure.

The concrete failure model used is the Hsieh-Ting-Chen four-parameter model. The model uses the following four-parameter criterion involving the stress invariants I_1 , J_2 and the maximum principal stress σ_1 (Kulak, 1997):

$$f(I_1, J_2, \sigma_1) = a \frac{J_2}{f_c^2} + b \frac{\sqrt{J_2}}{f_c} + c \frac{\sigma_1}{f_c} + d \frac{I_1}{f_c} - 1 = 0 \tag{5}$$

The four failure parameters (a , b , c , d) are determined so that they represent the following four failure states:

1. Uniaxial tensile strength, $f_t=0.1 f_c$
2. Uniaxial compressive strength, f_c ;

3. Equal biaxial compressive strength, $f_{bc}=1.15 f_c$;
4. Combined triaxial compression, $f_{pc}=0.8 f_c$, $f_{cc}=4.2 f_c$.

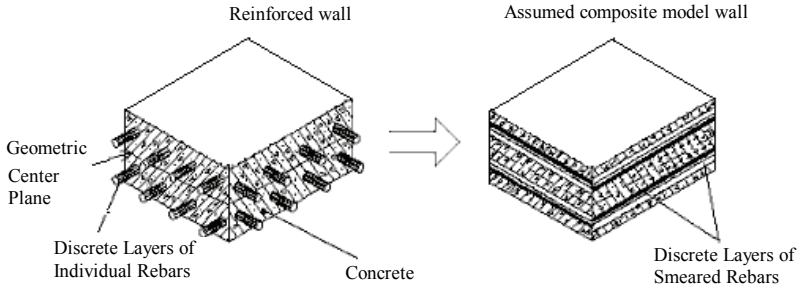


Fig. 1. Section through wall/slab and equivalent finite element model with distinct layers of smeared reinforcing bars

With Eq. (5) the strength capacity of the concrete in a multiaxial stress space can be characterized by the Hsieh-Ting-Chen four-parameter model failure surface (Kulak, 1997). A von Mises type loading function is used to determine elastic-plastic response. If the von Mises function is used alone, the same behaviour is implied in both tensile and compressive region, which is not true for concrete. Therefore, the von Mises function is used with Hsieh-Ting-Chen failure criterion that would produce different tensile and compressive responses. The failure surface of concrete in 2D space is schematically presented in Fig. 2.

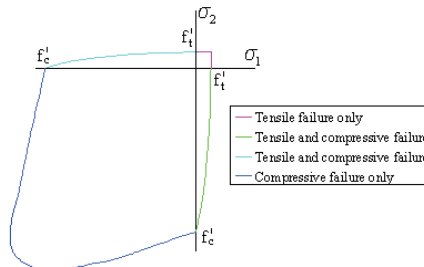


Fig. 2. Schematic representation of the failure surface

The transverse shear failure of a reinforced concrete slab is considered by an empirical formula (Kulak, 1997):

$$\tau_u = (0.05(pf_y - \sigma_y) + 0.5)\sqrt{f_c}; \dots \text{where} \dots 0.5\sqrt{f_c} \leq \tau_u \leq 4.5\sqrt{f_c} \tag{6}$$

Terms of equation (6) are the following: σ_y is the normal stress, f_y is the yield strength of reinforcement, p is the reinforcement ratio, f_c is the compressive strength of concrete. A typical uniaxial compression - tension behaviour of concrete is shown in Fig. 3. It is known that the cracked concrete of a reinforced concrete element can still carry some tensile

and compressive stress in the direction normal to the crack. The stress reduces to zero in tension for a strain e_t and stress reduces to zero in compression for a strain e_c .

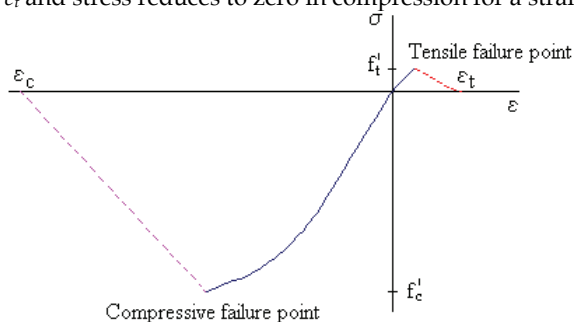


Fig. 3. Compression - tension behavior of concrete

The strain e_t at which the stress reduces to zero in tension and the strain e_c at which the stress reduces to zero in compression are used in this analysis. The value e_t is 0.000768 (4 times the tensile failure strain) and the value e_c is 0.0141 (3 times the compressive failure strain).

NEPTUNE calculates stresses at the centre of the concrete plate element at five integration points through the thickness and in each rebar layer. In the concrete plate element, layers of individual reinforcing bars within the concrete walls are represented by smeared uniformly distributed layers of steel. The thickness of these layers is determined by assuming that the cross-sectional areas of the reinforcing bars are spread uniformly along the respective pitch of the layers. The direction of reinforcement is specified in the concrete plate element. Transverse wall reinforcement and liner are neglected in the analytical model.

The steel-frame columns embedded in the ALS walls were modelled using a three-dimensional beam element (Belytschko et al., 1977). This beam element transmits moments, torque and axial forces and is a general six (6) degree of freedom element (i.e., three global translation and rotational components at each end of the beam). This element has a uniform cross-section and is capable of undergoing large deformations and can model elasto-plastic material response. The cross-section change of shape is not accounted for in any analysis.

3.2 Finite elements for modelling pipes

Pipelines were modelled using three-dimensional pipe elements. For the global solution of a pipe whip event, the use of a pipe element capable of undergoing large displacements in three-dimensional space was required. The pipe element in the NEPTUNE code was an enhanced version of a beam/pipe element developed by Belytschko and Schwer (Belytschko, 1977). The material model used can handle elasto-plastic behaviour. Validation of the use of the NEPTUNE code for pipe whip and impact problems was reported by Kulak and Narvydas (Kulak, 2001).

3.3 Finite elements for modelling contacts

For the two impact scenarios, the node-to-line contact element (Kulak, 1985 and Kulak, 1989) was employed. The node-to-line contact element, which is a three-node element, was

deployed in the following way. One node is attached to an appropriate finite element on the mesh representing the ruptured pipe or other structural component and the two other nodes are attached to an appropriate finite element on the mesh for the neighbouring structural component (pipe or wall) in which the contact occurs. This contact element is used for problems with simple geometry and when the contact-impact location is approximately known *a priori*. Since the connectivity is known *a priori*, no contact search algorithm needs to be used. Because of the simplicity of this approach, it is computationally very fast.

4. Coupling of Probabilistic and Deterministic Software

Because of the complexity needed to treat the physics that occurs during failure of NPP structures and consequence phenomena, analysts must rely on using sophisticated computer codes to model the accidents and their effects. The majority of these codes are deterministic codes in the sense that all the physical parameters used to define geometry, material properties, loadings, etc. as well as the computational parameters used in the analyses have fixed values. These values usually are chosen by the analysts/engineer as best estimate values. Thus, the results from these analyses do not take into account the range of variation of these parameters or the effect that different combinations of the parameters uncertainty have on the conclusions drawn from the analyses. With this approach, the probabilistic aspect of the parameters is neglected.

On other hand, in this work the probabilistic nature of the important parameters are taken into account to provide a more realistic assessment of safety issues. With this approach, a probabilistic analysis engine is coupled to a deterministic finite element engine to provide for integrated deterministic and probabilistic analysis of failure probabilities. Thus, two stand alone software packages are combined to provide probabilistic analysis capability for critical NPP structures.

In general, deterministic software (DS) is used for the deterministic analysis of system failure, accidents and/or consequences. In order to perform a probabilistic simulation, many deterministic analyses are performed using different values of the random variables defined by the probabilistic software (PS). The PS collects the results from the DS and performs a statistical analysis. Once all the deterministic computations are performed and transferred to the PS, the probabilistic software can determine probability estimate of system failure and/or consequences.

The initial part of the work is to develop the coupling translator, which consists of pre-processor and postprocessor used for data flow between deterministic and probabilistic software. The translator can be developed, using almost any advanced programming language (e.g. C++, Visual Basic or Perl). The language used should be suited to extract information from one text file and generate another as the main work is usually related to the input file preparation and output file data extraction. If the PS and/or DS are interactive software tools, the analyst works from a graphical user interface (GUI) and the translator also should be adapted to it.

There are five main steps to perform integrated probabilistic analysis:

1. Import a deterministic model (finite element model);
2. Define the random variables (initial set can be screened using sensitivity analysis);
3. Describe the failure criterion (based on deterministic criteria);

4. Run deterministic analysis to obtain the response for each set of random variables;
5. Analyse and review the results (probabilistic estimates).

The integration process starts when the DS input file, which represents the deterministic model, is processed (imported) by the translator. A pre-processor is used to provide the PS with information as to which parameters are to be used in the current deterministic model as random variables. This is necessary especially if the user can browse and select properties via the GUI provided by PS.

Most of the time when the translator is running, it is modifying the values of the random variables in the DS input file according to the values determined by the PS, and retrieving the values of the response variables from the DS output file for the probabilistic analysis by PS. At the end of computations, PS prepares an analysis report giving the results of the probabilistic analysis. Fig. 4 graphically shows the interactions among the user, PS, DS and the translator, which consist of pre-processor and post-processor.

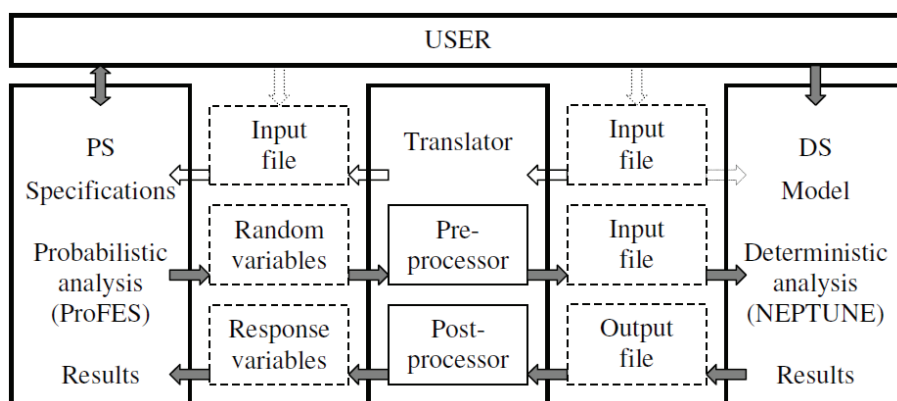


Fig. 4. The integration of models and different probabilistic methods

The first step in using PS with an integration of DS is to start PS. Then, the analyst imports the DS model into PS using the import interface. Next, the probabilistic model must be setup. This is done by selecting random variables, specifying distributions, setting correlations, designating dependent variables, defining limit states, etc. This part depends on the type of probabilistic analysis. In general, random variables and specific distributions with their parameters are specified. The list of considered variables depends on quantities from DS, which are available in input files and are proposed to be treated as changeable or random variables. The distribution functions and distribution parameters for each random variable are specified. Then, prior to each model call (i.e., DS calculation) corresponding values from the DS input file are modified by translator according to the values selected by PS. The list of response variable is also identified in initial stage. The response variables correspond to the requested output quantities (e.g. displacements, stresses, strain, etc.) according to which the failure parameters (e.g. failure rate, probability of failure) are calculated in the final stage.

Practically, probabilistic methods provide a means to assess the effects of uncertainties associated with material properties, component geometry data and loads in predicting structure reliability and performance. In order to perform a probabilistic analysis of an

building or component failure due to transient loading, the NEPTUNE and ProfES software were coupled. The probabilistic software ProfES was developed by the Computational Mechanics Group of ARA's Southeast Division, located in Raleigh NC (Cesare, 1999). ProfES allows quick development of probabilistic models from model executables, analytical formulations, or finite element models. It is possible to use ProfES independently to perform probabilistic simulations using functions internal to ProfES or functions that can be manually typed in. ProfES can be used as an add-on to the modelled executables, so it is possible to perform probabilistic studies using the deterministic models.

Here the NEPTUNE (Kulak and Fiala, 1998) software was used to perform deterministic transient analysis of an building and component during the dynamic loading on the values for the random variables selected by ProfES. By default, NEPTUNE input/output cannot be imported directly into ProfES. Thus, for this purpose, a special ProfES/NEPTUNE coupling code, *pnglue*, was developed at Argonne National Laboratory (Kulak, 2003). The code was programmed in Perl because it is well suited to extracting information from one text file and generating another. Since it is an interactive software tool, the analyst works from a graphical user interface. Initially a NEPTUNE input file is processed (imported) by *pnglue* to provide ProfES with information as to which values are available in the current model. This is necessary so that the user can browse and select properties via the graphical user interface provided by ProfES. Most of the time when *pnglue* is running, it is modifying the identified random variables in the NEPTUNE input file according to the values determined by ProfES and to retrieve the response variables from the NEPTUNE output file for probabilistic analysis by ProfES. At the end of computations, ProfES prepares an analysis report giving the results of the probabilistic analysis.

5. Structural Integrity Analysis Using Probabilistic Methods Applications to Ignalina NPP

The above methodology of the probability-based structural integrity analysis that integrates deterministic and probabilistic methods was applied to safety analysis of the Ignalina NPP. The application of this methodology to two postulated accidents, i.e. the pipe whip impact and aircraft impact, is presented in this chapter.

The preparation of models for analysis, evaluation of the material properties and loads are very important for structural integrity analysis of nuclear power plant building and components. The material parameter is uncertain. In order to ensure structural integrity analysis results of the buildings and component are reliable it is very important to evaluate uncertainty associated with loads, material properties, geometrical parameters, boundaries and other parameters. Therefore, it is necessary to account for the uncertainty in these quantities when performing a probabilistic structural integrity evaluation.

5.1 Probabilistic Analysis of Pipe Whip Impact

5.1.1 Model for the Analysis of Damage to the Adjacent Wall

The subject of this investigation is the collision between the group distribution header (GDH) and the adjacent wall that would result from a guillotine break at the blind end of a group distribution header. Structural integrity of the group distribution header support-wall is also important. Therefore, the group distribution header, the impacted wall and the

support wall (items 3, 1, 2, respectively, in Fig. 5 (a)) are included in the model of a whipping group distribution header analysis.

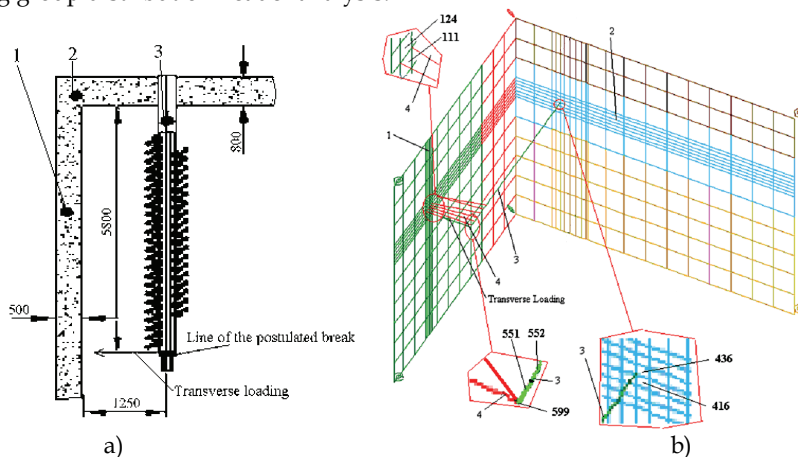


Fig. 5. Top view of the group distribution header's in the compartment (a) and combined schematic models of the GDH pipes and concrete walls (b) for investigation of impact to reinforced concrete wall: 1 - adjacent impacted-wall, 2 - GDH's support-wall, 3- broken GDH, 4 - contact element between GDH pipe and wall, 111 - number of rebar element adjacent to nodes of GDH and wall contact, 124 - number of concrete element adjacent to nodes of GDH and wall contact, 416 - number of rebar element adjacent to node of the whipping GDH pipe fixity, 436 - number of concrete element adjacent to node of the whipping GDH pipe fixity, 599 number of end node of the whipping GDH, 551, 552 - number of the whipping GDH pipe elements.

GDH is a horizontal cylinder (pipe) with outside diameter of 0.325 m, wall thickness of 0.015 m, and is about 6 m long. The distance from the centreline of the outboard group distribution header to the inside surface of the adjacent wall is 1.25 m (Fig. 5). The thickness of the impacted wall is 0.5 m and has the following reinforcement: 28 mm diameter reinforcement bars with a pitch of 200 mm and 16 mm diameter reinforcement bars with a pitch of 200 mm. The thickness of the supporting wall is 0.8 m with the following reinforcement: 28 mm diameter reinforcement bars with a pitch of 200 mm and 16 mm diameter reinforcement bars with a pitch of 200 mm.

The compartment walls were modelled using the four-node quadrilateral plate element (see section 3.1). The group distribution header's were modelled using three-dimensional pipe elements (see section 3.2). The FE model prepared for the investigation of a whipping GDH impact onto reinforced concrete wall is presented in Fig. 5 (b). After a guillotine break, the GDH can impact the nearest wall of the compartment.

Regarding material properties, the model analysed has two basic parts: the group distribution header that is made from 08X18H10T steel and the compartment walls that are made from M300 reinforced heavy concrete. More detailed descriptions of the geometry of the group distribution header, adjacent piping and surrounding walls and material properties are presented in (Dundulis et al., 2007a).

The transient analysis of a guillotine pipe break was reported in (Dundulis et al., 2007a). It was conservatively assumed that the transverse load applied to the end of the group distribution header was equal to the axial load. This load was also applied here in the probabilistic analysis. This essentially is an upper bound load. The load was not treated as a random variable in the analyses based on Monte Carlo Simulation and First Order Reliability method. However, the load of the guillotine break is uncertain. The Response Surface /Monte Carlo Simulation method was used to express failure probability as function of the loading and to investigate the dependence between impact load and failure probability.

5.1.2 Probabilistic Analysis Results

The aim of the uncertainty analysis is to identify and quantify all important uncertainty parameters. Ranges and subjective probability distribution describe the state of knowledge about all uncertain parameters. In probabilistic analysis, uncertainties in numerical values are modelled as random variables. The following mechanical properties and geometrical parameters important to strength of structures were simulated as random variables:

- Uncertain mechanical properties:
 - Concrete - Poisson ratio, Young's modulus, uniaxial tensile strength of pipe support-wall (walls No. 1 and 2 in Fig. 5);
 - Rebar - Yield stress (wall No. 1 and 2 in Fig. 5);
 - Pipe - Poisson ratio, Young modulus, Yield stress (pipes No. 3, Fig. 5);
 - Contact modulus (contact element between group distribution header and impacted-wall).
- Uncertain geometry data:
 - Reinforced concrete - Rebar area (wall No. 1 and 2 in Fig. 5);
 - Pipe - thickness and mid-surface radius of the pipe (pipe No. 3 in Fig. 5).

Values for the coefficient of variation were adopted from the following two paper: Hsin, 2001 and Braverman, 2001. In this analysis, the logarithmic normal distribution was used for mechanical properties and geometry parameters. The selected random variables, distributions and coefficients of variation are presented in Table 1 and Table 2.

Material	Distribution	Parameter	Mean	Coeff. of Variation	Comment
Reinforced Concrete					
Concrete (1, 2 walls - Fig. 1)	Log. Normal	Poisson's ratio	0.2	0.1	
	Log. Normal	Young's modulus	2.7e+10	0.1	Unit: Pa
	Log. Normal	Tensile strength	1.5 e+6	0.1	Unit: Pa
Reinforcement bars (1, 2 walls)	Log. Normal	Yield stress	3.916e+8	0.03	Unit: Pa
Group Distribution Header Pipe					
Austenitic Steel	Log. Normal	Young's modulus	1.8e+11	0.03	Unit: Pa
	Log. Normal	Poisson's ratio	0.3	0.03	
	Log. Normal	Yield stress	1.77e+08	0.03	Unit: Pa
Contact of Pipe	Normal	Contact modulus	1.8e+11	0.1	Unit: Pa

Table 1. The material properties and parameters expressed as random variables

Material	Distribution	Parameter	Mean	Coeff. of Variation	Comment
Reinforced Concrete					
Reinforcement bars	Log. Normal	Reinforcement layer thickness (data for 1 rebar as example).	0.00491	0.05	Unit: m
Group Distribution Header Pipe					
Austenitic Steel	Log. Normal	Wall thickness	0.015	0.05	Unit: m
	Log. Normal	Mid-surf. radius of pipe	0.155	0.05	Unit: m

Table 2. The geometry parameters of random variable

The aim of the transient analysis was to evaluate:

- Structural integrity of adjacent wall after impact;
- Structural integrity of the group distribution header support-wall.

The following limit states were assumed for the case of the group distribution header impact on the adjacent wall:

1. Limit State 1 - Contact between the broken group distribution header and the adjacent wall occurs.
2. Limit States 2, 3, 4, 5 and 6 - The concrete element adjacent to node of the group distribution header and wall contact reaches the ultimate strength in tension and the crack in concrete starts to open. NEPTUNE calculates stresses of the concrete element at five integration points. Therefore, the same limit states at all five integration points through wall thickness were checked.
3. Limit States 7, 8, 9, 10 and 11 - The concrete adjacent to the group distribution header fixity in the support-wall reaches the ultimate strength in compression and loses resistance to further loading. The same limit states at all five integration points through wall thickness were checked.
4. Limit State 12, 13, 14, 15 - The strength limit of the first layer of rebars in the concrete wall at the location of impact is reached and the rebars can fail. The same limit states at all four layers were checked in analysis using Monte Carlo Simulation method. In case of First Order Reliability method, the computational effort is proportional to the number of random variables and limit states. The probabilities of the failure of all rebars layers were received very small and similar in Monte Carlo Simulation analysis. Because of this, the Limit State 12 (the strength limit of the first layer of rebars) was used in analysis using First Order Reliability method.
5. Limit State 16, 17, 18, 19 - The strength limit of the first layer of rebars in the concrete support wall at the location of the group distribution header fixity is reached and the rebars can fail. The same limit states at all four layers were checked in analysis using Monte Carlo Simulation method. In case of First Order Reliability Method, the computational effort is proportional to the number of random variables and limit states. The probabilities of the failure of all rebars layers were received very small and similar in Monte Carlo Simulation analysis. According this the Limit State 16 (the strength limit of the first layer of rebars) was used in analysis using First Order Reliability method.

It is important to calculate the probability of concrete failure in the same run at all five integration points. Therefore, the following two system events were used in the probability analysis:

- System event 1 – Limit state 2, limit state 3, limit state 4, limit state 5 and limit state 6. This system event is evaluated as true if all the limit states are true. This system event evaluates the probability of crack opening in concrete at all integration points of the impacted wall, i.e., a complete crack through the wall.
- System event 2 – Limit state 7, limit state 8, limit state 9, limit state 10 and limit state 11. This system event is evaluated as true if all the limit states are true. This system event evaluates the probability of concrete failure (in compression) at all integration points at the location of the group distribution header fixity in support wall.

5.1.2.1 Probabilistic Analysis Results Using Monte Carlo Simulation Method

The Monte Carlo Simulation method was used to study the sensitivity of the response variables and the effect of uncertainties of material properties and geometry parameters to the probability of limit states. Twenty-nine random variables were screened; however, only the significant ones are discussed here. The screening of insignificant random variables from the large number of input random variables was performed using 95% confidence limits for sensitivity measures (acceptance limits for correspondent random variables). In order to have the possibility to compare different values, the sensitivity measures and 95% confidence limits were normalized.

The absolute value of a sensitivity measure is proportional to the correspondent random variable significance. The input random variable is considered insignificant when the correspondent sensitivity measure is close to zero. The sensitivity measures are likely to be within the acceptance limits if the random variable is insignificant. The response sensitivity measure (dY/dmu) is expressed as the derivative of the mean of the response variable with respect to the mean of the input random variable. The response sensitivity measures with acceptance limits are presented in Fig. 6. The “Input Random Variables” numbers are presented along the x - axis.

The following input random variables are the most significant random variables for the impacted wall at the location of concrete element number 124 (Fig. 5 (b)):

- Poisson’s ratio of the impacted-wall concrete (1 – Fig. 5) – input random variable 1;
- Young’s modulus of the impacted-wall concrete (1 – Fig. 5) – input random variable 2;
- Tensile Strength of the impacted-wall concrete (1 – Fig. 5) – input random variable 3;
- Yield Stress of the impacted-wall rebars (1 – Fig. 5) – input random variable 8;
- Young’s modulus of the whipping group distribution header (3 – Fig. 5) – input random variable 25;
- Wall thickness of the whipping group distribution header (3 – Fig. 5) – input random variable 28;
- Mid-surface radius of the whipping group distribution header (3 – Fig. 5) – input random variable 29.

These input random variables have the greatest positive (1, 2, 3, 25) or negative (8, 28, 29) influence on all integration points of concrete element 124.

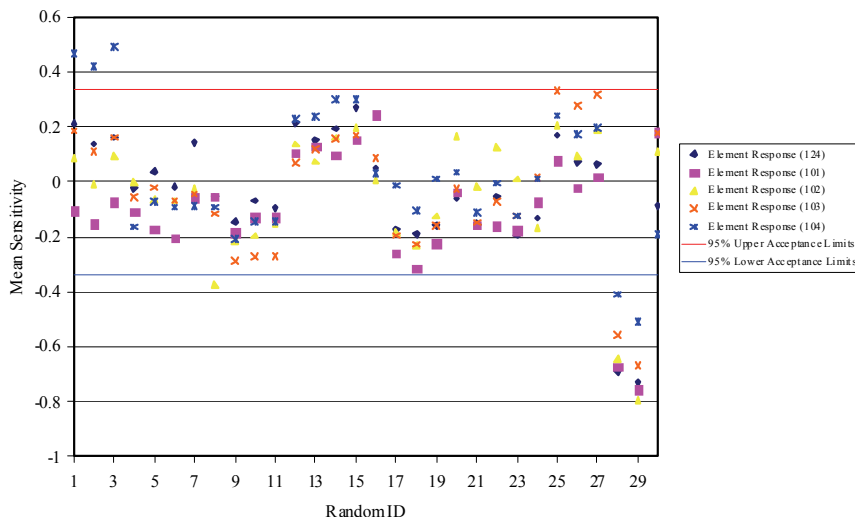


Fig. 6. Significant Random Variables for Element Response (124, 101, 102, 103, 104) Stress Equivalent.

According to the results of the sensitivity analysis related to response variables, materials properties, geometry data and limit states, the following additional items have the greatest influence on the probability of failure for the support-wall:

- Poisson’s ratio of the support-wall concrete (2 - Fig. 5) - input random variable 9;
- Young’s modulus of the support-wall concrete (2 - Fig. 5) - input random variable 10;
- Tensile Strength of the support-wall concrete (2 - Fig. 5) - input random variable 11.

All the previously listed random variables were used in the First Order Reliability method analysis as input random variables.

5.1.2.2 Probabilistic Analysis Results Using First Order Reliability Method

The FORM was used to study the probability of failure of the impacted-wall and the support-wall. The FORM is a preferred method for evaluating a small number of random variables and limit states (failure of concrete, reinforcement bars and group distribution header). The reason for this is that for the same precision as MCS, it often requires the least number of finite element model runs. With FORM, the computational effort is proportional to the number of random variables and limit states. Therefore the MCS sensitivity analysis was used to choose mechanical properties and geometrical parameters important to strength of structures for random variables in FORM. These random variables were presented in above subsection. The same limit states were used in the FORM analysis as the limit states used in MCS analysis. The number of simulations was 1419. The logarithmic normal

distribution of material properties and geometry data also was used for this analysis. The results of probabilistic analysis for limit states are presented in the Table 3 and Table 4.

Limit State	Definition	Probability	Beta
1	Nod. Response (599) Displ. Dir. 2 (Y) > 1.0875	0.981858	2.09373
2 - 6	El. Response (124) Stress Equivalent > 1.5e+6	~0.500~0.502	0.001~0.005
7 - 11	El. Response (436) Stress Equivalent < -1.7e+7	0.391 ~0.499	-0.002~-0.28
12 -13	El. Response (111) Stress Equivalent > 5.9e+8	0.326 ~ 0.11	-0.448~-1.22

Table 3. Results related to each Limit State

Name	Probability	Beta
Limit State 2 & Limit State 3 & Limit State 4 & Limit State 5 & Limit State 6	0.013	-2.22695
Limit State 7 & Limit State 8 & Limit State 9 & Limit State 10 & Limit State 11	0.0126	-2.23846

Table 4. Data of the System Event

The calculated probability of 'Limit State 1' is 0.982. This limit state probability indicates that contact between the whipping group distribution header and the adjacent wall will occur with probability of 0.982.

For the adjacent impacted-wall (element 124), the calculated probability of 'Limit States 2, 3, 4, 5 and 6' is from 0.500 to 0.502 (Table 3, 2-6). This indicates that the ultimate tensile strength of concrete will be reached at the five integration points and cracking in these layers may occur. The probability for a through crack in the concrete wall was calculated using System Event 1, which determines, for the same computer run, if cracking occurs in all the layers of the concrete element. The calculated probability of 'System Event 1' is 0.013 (Table 4). Thus, the probability for a through crack to develop is 0.013.

For the support-wall (concrete element number 436, which is the concrete element adjacent to the node at which the GDH is attached to the concrete wall), the calculated probability for 'Limit states 7, 8, 9, 10 and 11' is from 0.391 to 0.499 (Table 3, 7-11). These limit states indicate that the ultimate compressive strength of concrete will be reached at the five integration points and failure may occur. The system event was used for the analysis of probability of failure during the same computer run at all integration points of the concrete element. The calculated probability of 'System Event 2' is 0.0126 (Table 4). Thus, the ultimate compressive strength of concrete will be reached with a probability 0.0126, and the support-wall may fail at the location where the group distribution header is attached with a probability 0.0126.

For the impacted wall (element 111), the probability for 'Limit State 12' to be reached was 0.327. This limit state indicates that the ultimate stress of the rebars in the first rebar layer will be reached and the bars may fail. For the support-wall (element 416), the probability for 'Limit State 13' to be reached was 0.11. This limit state indicates that the ultimate stress of the rebars in first rebar layer will be reached with a probability 0.11 and this layer may fail.

5.1.2.3 Probabilistic Analysis Results Using Response Surface/Monte Carlo Simulation Method

The load of the guillotine break is uncertain, and it is widely accepted that to determine the loading from a guillotine break experiment is very difficult. Therefore, it is important to estimate the probability of failure of the impacted neighbouring wall due to the magnitude of the transverse load applied to the group distribution header. The RS/MCS method was used to express failure probability as a function of the loading and to investigate the dependence between impact load and failure probability.

In the first part of the RS/MCS analysis, the RS method was used to obtain dependence functions between the response variables and the input random variables. The number of RS simulations performed was 100. In the second part of the RS/MCS analysis, the MCS method was used to determine the probability of failure based upon these dependence functions. The number of MCS performed was 1,000,000.

The deterministic transient analysis of the whipping group distribution header was performed using the loading presented in paper Dundulis et al., 2007a. This load was also applied in the probabilistic analysis with the MCS and FORM. In fact this load is an upper bound load, and, in the MCS and FORM studies, it was not applied as a random variable but was considered to be deterministic. For the RS part of the RS/MCS analysis, a different loading was used than the one used in the First Order Reliability method analysis. The RS/MCS method in ProFES could not handle all the random variables related to the critical loading points (Dundulis et al., 2007a). So the mean value for the loading was defined to be a constant value of 338 kN in the range from 0.00 s up to 0.012 s and zero thereafter. The load value 338 kN is a half of the maximum load value (677 kN). The uniform distribution was used for loadings in RS part of the RS/MCS analysis. The distribution range of loading was from 0 N to the maximum loading of 677 kN.

In the RS/MCS analysis the same mechanical properties and geometrical parameters identified above as being important for the strength of structures were selected as random variables. The logarithmic normal distribution of material properties and geometry parameters were used for this analysis. The same limit states were also used in the RS/MCS analysis as those limit states used in the MCS and the FORM analysis.

Using the RS method, the dependence functions between response variables and input random variables were calculated. In the second part of the RS/MCS analysis, which is the MCS method, these functions were used to determine the failure probability.

The probability to reach the ultimate strength for compression of concrete in the support-wall as a function of the applied loads is presented here. As an example, the following equation was obtained from the RS analysis for the determination of the failure probability in relation to Limit State 7 (Table 3) - "Element Response (436 is the element number (first integration point)) Stress Equivalent > -1.7e+7":

$$y = -1.04735e+007 + -7.15074*L1 + -1.48883*L1 + 2.04335e+006*P1 + 0.00034832*Y1 + 9.06781e+007*u1 + -0.0222667*r1 + 1.85264e+006*P4 + 0.000158399*Y4 + -7.98442e+006*u4 + -2.10484e-007*Y7 + 1.02299e+009*t7 -1.53013e+007*m7 \quad (7)$$

where the response variable y is used in limit state: $y > -1.7e+7$; $L1$ - LoadUnit 1-1 and LoadUnit 1-3; $P1$ - Poisson's ratio of wall 1 (Fig. 5), $Y1$ - Young's modulus of wall 1, $u1$ - Uniaxial tensile strength of wall 1, $r1$ - Yield stress of reinforcement bar in wall 1, $P4$ -

Poisson's ratio of wall 4, $Y4$ - Young's modulus of wall 4, $u4$ - Uniaxial tensile strength of wall 1, $Y7$ - Young's modulus of pipe 7, $t7$ - thickness of pipe 7, $m7$ - mid-surface radius of pipe 7.

L1 - LoadUnit 1-1 and Load 1-3 are loading points at different times, i.e. LoadUnit 1-1 at 0 second, LoadUnit 1-3 - at the time when the whipping group distribution header moves outside of the diameter of the group distribution header end cap (0.012 second). The random variables included in this equation are explained in the section 5.1.2.1.

The dependence function, Eq. (7), which was obtained using the RS method, was applied as an internal response functions in the MCS analysis. The number of MCS simulations was 1,000,000. In Eq. (7) the loads L1 and L3 were assumed equal. They were changed step-by-step while the probability of the limit state has been changing from 0 to 1. The normal distribution with Coefficient of Variation equal 0.1 (10%) for loading and the logarithmic normal distribution of material properties and geometry parameters was used in this analysis. In Eq. (7), the nominal values of material properties and geometry parameters were the same as used in other analysis.

The analysis results are presented in Fig. 7. According to these result the relation between the probability of the 'Limit State 7' and the applied loads was determined. The compressive strength limit of concrete element 436 is first reached at a loading approximately equal to 550 kN, and the concrete failure probability reaches 1 at a load of approximately 950 kN. Note, the probability of failure at a load of 677 kN is about 0.4, which is good agreement with the results from the First Order Reliability method analysis.

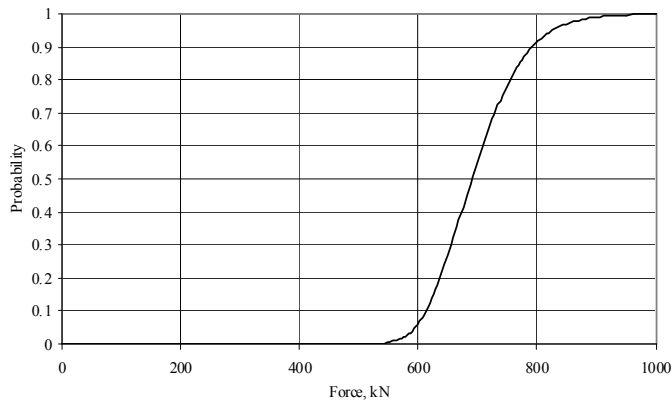


Fig. 7. The failure probabilities of concrete element adjacent to node of the group distribution header pipe fixity (node 436) to compression at dynamic loading due to guillotine rupture

5.2 Probabilistic Analysis of an Aircraft Crash

5.2.1 Model for the Analysis of Failure of the Building

The subject of this investigation is the integrated analysis of building failure due to impact by a commercial aircraft. The model of the Ignalina NPP reactor building that was used for the deterministic analysis of aircraft impact was reported by Dundulis et al., 2007b. One run

of that Ignalina building model using the NEPTUNE code takes approximately one hour. This duration is extremely long for performing the large number of runs needed for a probabilistic analysis. Therefore, a modification of the original FE model used in the deterministic analysis of the Ignalina NPP building is used for the probabilistic analysis (Dundulis et al., 2007c). The impacted wall and the adjacent walls and ceilings are included in the modified FE model of the Ignalina NPP building. The modified finite element model is presented in Fig. 8. One crash/impact location was considered. Arrows depict the assumed impact area of the aircraft. The impact direction is assumed to be perpendicular to the selected wall of the building.

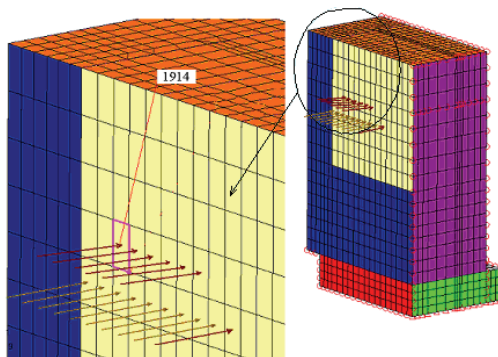


Fig. 8. Finite element model of the Ignalina NPP building for aircraft crash analysis

The wall of the building was modelled using the four-node quadrilateral plate element (see section 3.1). Some composite metal frames, made from different steel components, are imbedded in the walls. These structures were modelled using separate beam finite elements (see section 3.1) and were added to the walls and slabs at appropriate locations along the edges of quadrilateral elements.

5.2.2 Probabilistic Analysis Results

In probabilistic analysis of failure of the building due aircraft crash as in case of pipe whip impact analysis, uncertainties in numerical values are modelled as random variables. The following mechanical properties and geometrical parameters, which determine the strength of the structures, were used as random variables:

- Mechanical properties: Concrete - Young's modulus, stress points of the compressive stress-strain curve of the impacted and support walls; Reinforcement bar - stress points of the stress-strain curve of the impacted wall and support walls.
- Geometry data: Concrete wall - thickness of the impacted wall and support walls; Reinforced concrete - rebar area of the impacted wall and support walls.

The selected random variables, distributions and coefficients of variation of the mechanical properties of concrete and reinforcement bars, and geometry data are used same methodology as for pipe whip impact (section 5.1, Table 1 and Table 2).

The points defining the load curve are considered to be random variables. These points represent the beginning/end points at which different components of the aircraft structure (e.g., fuselage, wings, engine, etc.) begin to contact or end contact on the building wall.

Thus, in a sense, this approach takes into account the variations in loading from the individual structural components. The normal distribution for the load points is used. The load data is presented in paper Dundulis et al., 2007c.

The objective of the transient analyses is to evaluate the effects of an aircraft crash on an Ignalina NPP building structure. The structural integrity analysis was performed for a portion of the Accident Localization System (ALS) using the dynamic loading of an aircraft crash impact model caused by civil aircraft travelling at a velocity of 94.5 m/s. The aim of the transient analysis was to evaluate:

- Structural integrity of the impacted wall of the building;
- Structural integrity of the building walls adjacent to the impacted wall.

Based on the objective of the transient analyses, the following limit states were selected:

- Limit States 1-5 - The concrete in element number 1914 (Fig. 8) in the impact area reaches the ultimate strength in tension and a crack starts to open. This impacted wall is the outside wall of the ALS and a through-the-thickness crack should not develop. Additional description see in section 5.1.2 "Limit states 2, 3 ..".
- Limit States 6-10 - The concrete element of the support wall of the building reaches the ultimate strength in compression and a compressive failure occurs. This neighbouring wall is an inside compartment wall of the ALS and the cracks in this wall may open. Therefore, the strength of wall was evaluated for compression. The same limit states at all five integration points through the thickness were checked.
- Limit State 11- 13 - The splice failure strain limit of 4% for the rebars in element number 1914 (Fig. 8) would be reached and the rebars would fail. All three layers (i.e., L1 through L3) of rebars were checked. Note, Layer L3 is on the impact side of the wall.
- Limit State 14- 17 - The splice failure strain limit of the first layer of rebars in the interior concrete wall is reached and the splice would break. The same limit states at all four layers of the reinforcement bars were checked.

It is important to calculate the probability of concrete failure at all five integration points in the same computer run. Also it is important to calculate probability of reinforcement bar failure in all layers in same run. Therefore, the following four system events were used in the probability analyses:

- System Event 1 - Limit state 1 - 5. This system event is evaluated as true if all the limit states are true within the same run. Additional description see in section 5.1.2 "System Event 1 ..".
- System Event 2 - Limit state 6 - 10. This system event is evaluated as true if all the limit states are true within the same run. Additional description see in section 5.1.2 "System Event 2 ..".
- System Event 3 - Limit state 11 - 13. This system event is evaluated as true if all the limit states are true within the same run. This system event evaluated the probability of rebar failure at all layers of the impacted wall.
- System Event 4 - Limit state 14 - 17. This system event is evaluated as true if all the limit states are true within the same run. This system event evaluated the probability of rebar failure at all layers of the neighbouring support wall.

5.2.2.1 Probabilistic Analysis Results Using Monte Carlo Simulation Method

Using the MCS probabilistic analysis method, the probabilities of limit states and the probability of failure for system events were calculated for both the impacted wall and the adjacent interior wall, which provides support to the impacted wall. The number of MC simulations was 3000. It should be pointed out that because of the small number of MC simulations performed, the probabilistic analysis using the MCS method was performed as a scoping study.

For the impacted wall, the calculated probability of 'Limit states 1 -3' is from 0.645 to 0.964. These probabilities indicate that the tensile failure surface of the concrete element within the impact area will be reached at three of the five integration points and a crack could develop in these three layers. The calculated probability of 'Limit states 4 -5' is very small, i.e., at the fourth integration point it is 0.007, and at the fifth integration point it is 0. These values indicate that the probability of a crack opening in the fourth and fifth layers of this concrete element is very small.

The probability of a crack opening at all five integration points in a concrete element within the impact area during the same run was calculated. The system event was used to analyze this probability of failure. The calculated probability of 'System event 1' is 0. This indicates that, within the same run, the tensile failure surface of the concrete at all the integration points of this element is not reached, and the probability of crack opening in the concrete element of impacted wall is very small.

The calculated probabilities for 'Limit states 11 - 13' and of 'System event 3' are also 0. This indicates that the splice failure strain of the rebars within the impact area will not be reached in any of the rebars, and the probability of rebar splice failure is very small. For layers 1, 2 and 3 of the impacted wall, the probabilities for concrete failure are near 1. In contrast, for concrete layers 4 and 5 and the rebars of the impacted wall, the probabilities are near 0.

Based on these results, only very small probabilities of failure exist in several layers of concrete and in all layers of rebars. Therefore in the next section, the FORM method was used for additional evaluation of failure probabilities of the impacted wall.

For the interior support wall, the calculated probabilities of 'Limit states 6 -10' and of 'System event 2' are 0. Thus, the compressive failure surface of the concrete of the support wall will be reached with a probability of 0 for all the integration points of this element, and the probability of compressive failure is very small. The calculated probability for 'Limit states 14 - 17' and of 'System event 3' are also 0. This indicates that the splice failure strain for the rebars in the support wall will not be reached for any of the rebar layers, and the probability of a rebar splice failure is very small.

For the interior wall, the probabilities of failure are 0 for all concrete layers and rebar layers. Since this wall is an inside wall of the building and is not very important for leak tightness of the ALS, no additional evaluation of the probability of failure of this wall was carried out.

5.2.2.2 Probabilistic Analysis Results Using First Order Reliability Method

FORM was used to study the probability of failure of the impacted wall of the Ignalina NPP building due to the effects of an aircraft crash onto the building..

The same mechanical properties and geometrical parameters used in the MCS analysis of the impacted wall were used as random variables in the FORM analysis. The 'Limit states 1-5' and 'Limit states 11-13,' (see section 5.2.2) were used here in this analysis. It is important to calculate the probability of concrete failure at all five integration points of the element

within the same run. Similarly, it is important to calculate the probability of reinforcement bar failure in all three layers within the same run. Therefore, the two system events were used in the probability analysis, i.e. the 'System event 1' and 'System event 3' (see section 5.2.2). The number of simulations performed was 1419. The probabilities of limit states and system events were calculated. The results of the probabilistic analysis are presented in Tables 5 and 6.

Limit State	Definition	Probability	Beta
1-5	CF ¹ at IP1-IP5: StE ² > 3.79e+06	0.498~0.51	-5.78e-3~4.12e-3
11	RBF ³ in L1: SE ⁴ > 0.04	0.2296	-7.4e-1
12-13	RBF in L2/L3: SE > 0.04	0.003/5.3236e-171	~-2.7616

Table 5. Failure probabilities for Limit States in Element 1914 (¹CF-Concrete Failure, ²StE-Stress Equivalent, ³RBF-Reinforcement bar failure, ⁴SE-Strain Equivalent)

Name	Probability	Beta
Through-the-thickness CF ¹ (i.e., Failure in LS 1 through 5)	0.0266	-1.93307
Failure of all RBL ² (i.e., failure in LS 11 through 13)	0	-4.01317

Table 6. Failure probabilities for System Events in Element 1914 (¹CF-Concrete Failure, ²RBL-Reinforcement Bar Layers)

The calculated probabilities for 'Limit states (LS, Table 5) 1 - 5' are from 0.498 to 0.510. This indicates that the tensile failure surface of the concrete in the impact area could be reached at each of the five integration points (IP1 through IP5) but not during the same computer run. Thus, a crack in each layers of this concrete element could open. The probability of a crack occurring at all five integration points in the concrete element during the same run was calculated (i.e., System Event 1, Table 6), and the value was 0.0266. This indicates that the probably of the ultimate strength for tension being exceeded through the thickness of the concrete element in the impact area is 0.0266. Recall that the MCS indicates that the probability for System Event 1 was 0.

The probability for 'Limit State 11,' which checks for failure of the first rebar layer (L1) in the concrete wall at the location of impact, was found to be 0.2296. The calculated probabilities for 'Limit States 12 and 13' are very small; the probabilities for the third and fourth rebar layers were 0.003 and ~0, respectively. These limit state values indicate that the probability for splice failure of the rebars in the third and fourth rebar layers is very small. The probability of exceeding the splice failure strain in all rebar layers of the impacted wall (i.e., System Event 2) was 0. Thus, the aircraft should not penetrate the reinforcement in the impacted wall.

5.2.2.3 Probabilistic Analysis Results Using Response Surface / Monte Carlo Simulation Method

During an aircraft crash, the dynamic impact loading is uncertain. Therefore, it is important to estimate the dependence of the failure probability of the building due to the uncertainty in loading. The RS/MCS method was used for the determination of such a relation

expressed by the probability-loading function. In this section only concrete failure (limit state 1) is presented.

First using the modified finite element model of the building, a limited number of MC simulations were performed to determine the response surfaces, i.e., the probability functions for failure of the walls of the Ignalina NPP building. Then the MCS method was used on the response surfaces to study the probability of failure of the building walls as indicated by concrete cracking and reinforcement bar rupture (Dundulis et al., 2007c). In this analysis the probability function was used as an internal function in ProFES to determine the failure probability, which greatly reduces computational effort.

The same mechanical properties and geometrical parameters used in the MCS and FORM analyses were also used as random variables in the RS/MCS method. Also, the same limit states used in the FORM analysis were used in the RS/MCS analyses.

As a basis, the loading function for a civil aircraft traveling at 94.5m/s (Dundulis et al., 2007b) was used. The nonlinear function consists of a series of straight line segments with the peak load of 58MN occurring at 0.185s. Because of limitations on the number of random variables imposed by ProFES, it was necessary to use a surrogate loading function that was a linear function starting at 0 MN at the instant of impact and reaching a peak value of 58MN at 0.185s. It is noted that this surrogate function provides a larger impulse than the original function. For this part of the analysis, which is to do a preliminary study of the effect that the loading has on the probabilistic analysis, the peak load was varied, arbitrarily, from 0MN to 700MN, and the time at which the peak load occurred was kept constant at 0.185s. The probability distribution function for the loading was chosen to be uniform distribution.

Using the modified FE model, two hundred (200) MC simulations were performed to determine the probability functions for Limit States 1 (see section 5.2.2). The distribution for the loading was assumed to be uniform. The range of loading was from 0.02 MPa to 2.2 MPa, and this loading range encompassed the probability of failure range from 0 to 1. The maximum point of loading used in the deterministic transient analysis of civil aircraft travelling at a velocity of 94.5 m/s crash is 1.557 MPa (correspond to 58 MN). Using the Response Surface Method, the dependence functions for the response variables based on the input random variables were calculated.

The probabilistic function for Limit State 1, which is the development of a crack on the initial tension side of the wall, is given by:

$$\begin{aligned}
 Y1 = & 2.726e+6 + -2.816*L + -2.743e+6*T + 0.043*c1 + 0.012*c2 + 0.005*c3 + \\
 & 2.991e+008*R1 + 1.320e+9*R3 + -1.775 e+9* R4 + -0.004*s1 + -0.007*s2 + \\
 & -0.004*s3 + 0.008*s4
 \end{aligned}
 \tag{8}$$

where Y1 - Limit State 1, i.e. $Y1 > 3.79e+6$; (the value 3.79e+6 is the concrete experimental ultimate strength in tension); L - maximum force loading (pressure) point in impacted wall; T - wall thickness of the impacted wall; c1, c2 and c3 - stress points 1, 2 and 3 for concrete respectively; R1, R3 and R4 - thickness of reinforcement bars in the first, third and fourth layers, respectively; s1, s2, s3 and s4 - stress points 1, 2, 3 and 4 for the steel reinforcement bars, respectively.

The equations obtained using the RS method were used as internal response functions in the subsequent MCS analysis. The number of MCS simulations was 1,000,000. The probability

function (8) was used to determine the relationship between the probability of 'Limit state 1' being reached and the applied load, i.e. peak value of the impact force. The force loading was applied to the assumed aircraft impact area in the form of pressure in the transient analysis of Ignalina NPP building. The pressure value was recalculated to a force value and the probability results were presented as the relation between probability and force. The results of the probabilistic analysis are presented in Fig. 9. The probability of failure for Limit State 1 (concrete element reaches ultimate strength in tension) is zero (0) up to 10 MN, and the probability of failure is 1 at the resultant force approximately equal to 80 MN. Note, the maximum force used in the deterministic structural integrity analysis of the Ignalina NPP building for an aircraft impacting into the building at 94.5 m/s is approximately equal to 58 MN.

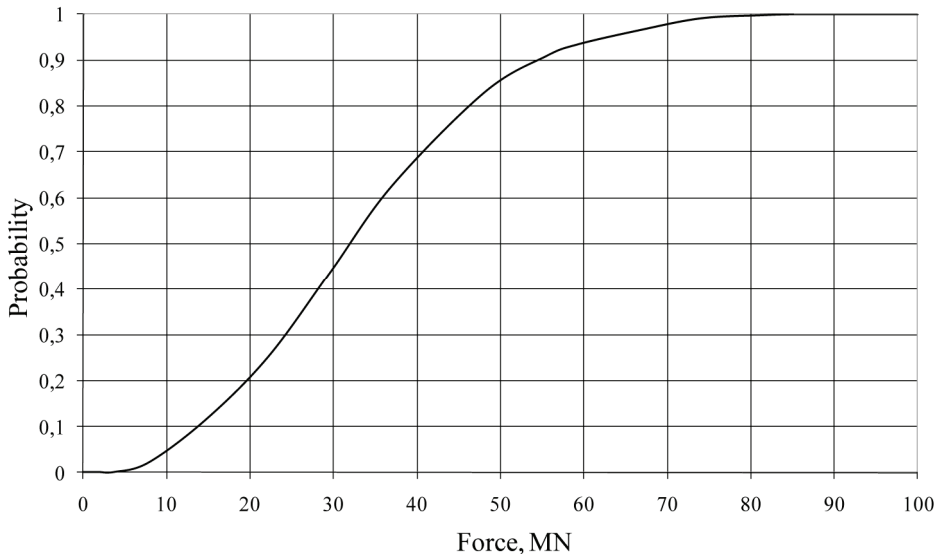


Fig. 9. Probability of a crack developing in the initial tension side of the impacted wall

6. Conclusions

The probability-based approach that integrates deterministic and probabilistic methods was developed to analyse failures of NPP buildings and components. This methodology was applied to safety analysis of the Ignalina NPP. The application of this methodology to two postulated accidents—pipe whip impact and aircraft crash—is presented in this chapter. The NEPTUNE software system was used for the deterministic transient analysis of the pipe whip impact and aircraft crash accidents. Many deterministic analyses were performed using different values of the random variables that were specified by ProfES. All the deterministic results were transferred to the ProfES software system, which then performed probabilistic analyses of piping failure and wall damage.

A probabilistic analysis of a group distribution header guillotine break and the damage consequences resulting from the failed group distribution header impacting against a neighbouring wall was carried out.

The Monte Carlo Simulation method was used to study the sensitivity of the response variables and the effect of uncertainties of material properties and geometry parameters to the probability of limit states and also for probability of failure of building structures.

The First Order Reliability method was used to study the probability of failure of building structures. The results of the probabilistic analyses show that the MCS method— using a small number simulations—is more conservative than the FORM method when determining large values for failure probability. The MCS method, however, is not conservative for determining small values for failure probability.

The Response Surface / Monte Carlo Simulation method was used in order to express failure probability as function and to investigate the dependence between impact load and failure probability.

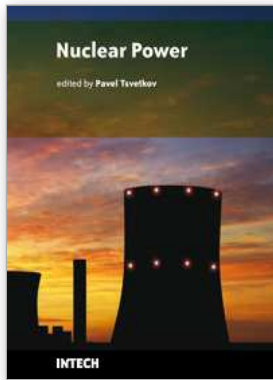
With the large uncertainty in values for material properties and loadings that exist in complex structures—such as nuclear power plants—it is not acceptable to only perform a deterministic analysis. Probabilistic analysis as outlined in this chapter and applied to two extreme loading events is a necessity when credible structural safety evaluations are performed.

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The world of the twenty first century is an energy consuming society. Due to increasing population and living standards, each year the world requires more energy and new efficient systems for delivering it. Furthermore, the new systems must be inherently safe and environmentally benign. These realities of today's world are among the reasons that lead to serious interest in deploying nuclear power as a sustainable energy source. Today's nuclear reactors are safe and highly efficient energy systems that offer electricity and a multitude of co-generation energy products ranging from potable water to heat for industrial applications. The goal of the book is to show the current state-of-the-art in the covered technical areas as well as to demonstrate how general engineering principles and methods can be applied to nuclear power systems.

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