PROBABILISTIC SAFETY ASSESSMENT OF THE BUILDINGS DESIGN DUE TO THE BLAST LOAD

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This project was realized with the support of the Grant Agency SR into VEGA 1/0849/08 and 1/1039/12.

Abstract: This paper deals with the problem of the probabilistic design of buildings under influence of extreme loads from above-ground explosion. The shock wave propagation in the air and the effect of space wave propagation to high rise building is evaluated. There is considered explosion of 100kg TNT at 40m in accordance of IS:4991-1968 for residential buildings. The response of the concrete structures under the shock load is a complex nonlinear and rate-depend process. Willam-Wärnke's failure criterion and the smeared approach for cracking and reinforcement modeling is used. The simulations of the variable input parameters (model and material uncertainties) are used on the base of the response surface method (RSM). On the example of the panel building the efficiency of the probabilistic analysis to optimal design of the high rise buildings is presented.

Keywords: probabilistic analysis, reinforced concrete, blast load, ANSYS

1 Introduction

In recent years, there has been considerable concern about the terrorism [2, 4, 8, 9, 11 and 12]. A structure may be subjected during its lifetime to extreme loading conditions that exceed its design loads [1, 3, 5 and 6]. Amongst these loading conditions are major earthquakes, explosions, unexpected impact forces, and fire. Unfortunately, many structures are not being designed to resist such extreme loads due to economic reasons. Numerical solution is complicated, because it is the problem of wave propagation in non-homogeneous medium and problem of time variation of the blast load. The wave propagation is simulated on 3D FEM nonlinear model in software ANSYS on the base of the Brode's explosion model [12]. The probabilistic analysis of the structure reliability may be used for the dynamics calculation and variable parameters in the form of the histograms in accordance of requirements of the Eurocode 1991.

Fig.1 Building collaps in Khobar Towers due to blast action (1996)

Fig.2 Building of Federal Murrah Building after blast attack (1996)

Fig.3 Buildings in Oslo after terrorist attack (2011)

2 Reliability analysis methods

From the point of view of one's approach to the values considered, structural reliability analyses can be classified in two categories, i.e., deterministic analyses and stochastic analyses. In the case of the stochastic approach, various forms of analyses (statistical analysis, sensitivity analysis, probabilistic analysis) can be performed. Considering the probabilistic procedures, The Eurocode 1 recommends a 3-level reliability analysis [3]. The reliability assessment criteria according to the reliability index are defined here. Most of these methods are based on the integration of Monte Carlo (MC) simulations [6 and 10]. Three categories of methods have been presently realized [6].

2.1 Straight Monte Carlo methods

The Monte Carlo methods are based on a simulation of the input stochastic parameters according to the expected probability distribution. The accuracy of this method depends upon the number of simulations and is expressed by the variation index:

$$\nu_{p_f} = \frac{1}{\sqrt{N p_f}} \tag{2}$$

where N is the number of simulations. If the required probability of failure is $p_f = 10^{-4}$, then by the number of simulations $N = 10^6$, the variation index is equal to 10%, which is an acceptable degree of accuracy.

2.2 Modified LHS method

The modified LHS method is based on the same number of simulations of the function g(X) as in the Monte Carlo method; however the zone of the distributive function $\Phi(X_j)$ is divided into N intervals with identical degrees of probability. This method provides good assessments of the statistical parameters of the structural response when compared to the Monte Carlo method. Using the LHS strategy, we get values like the reliability reserve parameter – the mean value , the standard deviation σ_z, the slant index α_z, the sharpness index e_z, or the empirical cumulative distribution function. The reduction of the number of simulations (tens to hundreds of simulations) means a valuable benefit from this method compared to the straight Monte Carlo method (thousands to millions of simulations).

2.3 Approximation methods - Response Surface Method

The approximation methods are based on the assumption that it is possible to define the dependency between the variable input and the output data through the approximation functions in the following form:

$$\hat{Y}=c_o+\sum_{i=1}^{NRV} c_i X_i+\sum_{i=1}^{NRV}\sum_{j=1}^{NRV} c_{ij} X_i . X_j \tag{1}$$

where c_o is the index of the constant member; c_i are the indices of the linear member and c_{ij} the indices of the quadratic member, which are given for predetermined schemes for the optimal distribution of the variables (Montgomery, Myers) or for using regression analysis after calculating the response (Neter). Approximate polynomial coefficients are given from the condition of the error minimum, usually by the "Central Composite Design Sampling" (CCD) method or the "Box-Behnken Matrix Sampling" (BBM) method. Drawbacks of the method: The number of simulations depends on the number of variable input parameters; in the case of a large number of input parameters, the method is ineffective, the method is unsuitable in the case of discontinuous changes in the dependencies between the input and output values (e.g., the method is not suitable for resolving the stability of ideal elasto-plastic materials beyond the failure limit...).

3 Material model of concrete

3.1 Willam and Warnke material model of concrete

The concrete is a material with a different behaviour under compression stress and tension stress, also there is different behaviour under static and dynamic loading [11]. Therefore, formulation of failure criterion is complicated. Several failure criterion are well known for concrete- Mohr – Coulomb, Drucker – Prager, Willam – Warnke, Chen (Chen, W. F., Ting, E. C., 1980). In this paper is used Willam and Warnke failure criterion for concrete. Willam and Warnke (1974) developed a widely used model for the triaxial failure surface of unconfined plain concrete. The failure surface is separated into hydrostatic (change in volume) and deviatoric (change in shape) sections. The failure criterion for triaxial stress state is defined as:

$$F/f_c - S \geq 0$$

where F is a function of principal stress state, f_c is uniaxial crushing strength, S is failure surface. A total of five input strength parameters (each of which can be temperature dependent) are needed to define the failure surface as well as an ambient hydrostatic stress state. This are : f_c - the ultimate uniaxial compressive strength; f_t - the ultimate uniaxial tensile strength; f_{cb} - the ultimate biaxial compressive strength and $f_1 = 1{,}45 f_c$; $f_2 = 1{,}7255 f_c$

The Willam and Warnke (1974) mathematical model of the failure surface for the concrete has the following advantages:

a) Close fit of experimental data in the operating range;
b) Simple identification of model parameters from standard test data;
c) Smoothness (e.g. continuous surface with continuously varying tangent planes);
d) Convexity (e.g. monotonically curved surface without inflection points).

For using Willam-Warnke's model of the concrete in the ANSYS is required to define 9 different constants. These 9 constants are: Shear transfer coefficients for an open crack; Shear transfer coefficients for a closed crack; Uniaxial tensile cracking stress; Uniaxial crushing stress (positive); Biaxial crushing stress (positive); Ambient hydrostatic stress state; Biaxial crushing stress (positive); Uniaxial crushing stress (positive); Stiffness multiplier for cracked tensile condition.

Typical shear transfer coefficients belong to the interval 0.0 to 1.0. Coefficient 0.0 represents a smooth crack (complete loss of shear transfer) and 1.0 represents a rough crack (no loss of shear transfer). Convergence problems occurred when the shear transfer coefficient for the open crack dropped below 0.2.

The failure of concrete is categorized into four domains:

1) Compression - compression – compression
2) Tensile - compression – compression
3) Tensile - tensile – compression
4) Tensile - tensile – tensile

3.2 Smeared approach for concrete cracking

The presence of a crack at an integration point is represented through modification of the stress-strain relations by introducing a plane of weakness in a direction normal to the crack face. A shear transfer coefficient βt is introduced which represents a shear strength reduction factor for those subsequent loads which induce sliding (shear) across the crack face. The stress-strain relations for concrete that has cracked in all three directions are:

$$\left[D_c^{ck}\right] = E \begin{bmatrix} \frac{R^t}{E} & 0 & 0 & 0 & 0 & 0 \\ 0 & \frac{R^t}{E} & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & \frac{\beta_t}{2(1+\nu)} & 0 & 0 \\ 0 & 0 & 0 & 0 & \frac{\beta_t}{2(1+\nu)} & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{\beta_t}{2(1+\nu)} \end{bmatrix} \qquad (3)$$

where the superscript ck signifies that the stress strain relations refer to a coordinate system parallel to principal stress directions with the x_{ck} axis perpendicular to the crack face. R_t is the slope (secant modulus) as defined on the figure 3.1.

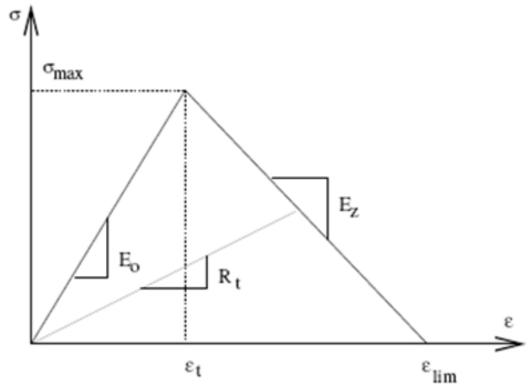

Figure 3.1. Strength of Cracked Condition

3.3 Constitution relationship for reinforcement bars

There is used smeared approach for reinforcement bars modeling. The stress-strain matrix with respect to each coordinate system:

$$\left[D_{sx}\right] = \begin{bmatrix} \rho E_s & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \end{bmatrix} \qquad (4)$$

where E_S is modulus of elasticity of reinforcement material, ρ is reinforcement ratio. For reinforcement steel is used bilinear stress-strain relationship and the breaking of reinforcement bars is given by maximal value of strain.

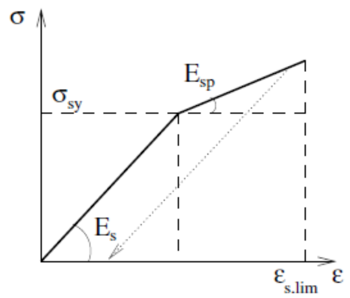

Figure 3.2. Bilinear stress-strain relationship for steel reinforcement

Material matrix of reinforced concrete is expressed in form:

$$\left[D\right] = \left[D_c^{ck}\right] + \sum_{i=1}^{n} D_{s,i} \qquad (5)$$

3.4 Material design strength

Under the action of rapidly applied loads the rate of strain application increases and this may have a market influence on the mechanical properties of structural materials. In comparison with the mechanical properties under static loading the effect may be summarized as in the table 3.1.

Table 3.1. Dynamics increasing factor for properties of structural materials

Type of stress	Concr -ete	Reinfor- cing bars	Concr- ete 38MPa	Reinfor- cing bars 500MPa
	f_{dcu}/f_{cu}	f_{dy}/f_y		
Bending	1,25	1,2	47,5	600
Shear	1,00	1,1	38,0	550
Compression	1,15	1,1	43,7	550

4 Definition of blast load

The term detonation refers to a very rapid and stable chemical reaction which proceeds through the explosive material at a speed, called the detonation velocity, which is supersonic in the unreacted explosive [12]. The detonation wave rapidly converts the solid or liquid explosive into a very hot, dense, high-pressure gas, and the volume of this gas which had been the explosive material is then the source of strong blast waves in air. The blast effects of an explosion are in the form of a shock wave composed of a high intensity shock front which expands outward from the surface of the explosive into the surrounding air. As the wave expands, it decays in strength, lengthens in duration, and decreases in velocity. Expressions for the peak of static over-pressure P_{so} developed in a blast have been presented in the literature to model free-field conditions in which dynamic interactions of the wave front with objects obstructing the blast wave path is small enough to be neglected. P_{so} have typically been correlated with the scaled distance parameter (Z) which is defined by:

$$z = R/W^{1/3}$$

where R is standoff distance in meters and W is the charge weight of the blast in kg based on TNT equivalence. Brode (1955) developed the correlation between P_{so} and Z based on numerical modeling. This correlation was subsequently reviewed by Smith (1994) [12] who compared Brode's model with results obtained from more recent experimental studies.

$$p_{so} = \frac{1407,2}{z} + \frac{554}{z^2} - \frac{35,7}{z^3} + \frac{0,625}{z^4}; 0,05 \le z \le 0,3$$

$$p_{so} = \frac{619,4}{z} - \frac{32,6}{z^2} + \frac{213,2}{z^3}; 0,3 \le z \le 1,0 \qquad (6)$$

$$p_{so} = \frac{66,2}{z} + \frac{405}{z^2} - \frac{328,8}{z^3}; 1,0 \le z \le 10$$

The reflected over-pressure P_r arising from the interaction of the blast waves with a flat- surface has been modeled by Smith [12] and is approximated by:

$$P_{rmax} = C_r P_{so} \quad \text{where} \quad C_r = 3(\sqrt[4]{P_{so}}) \qquad (7)$$

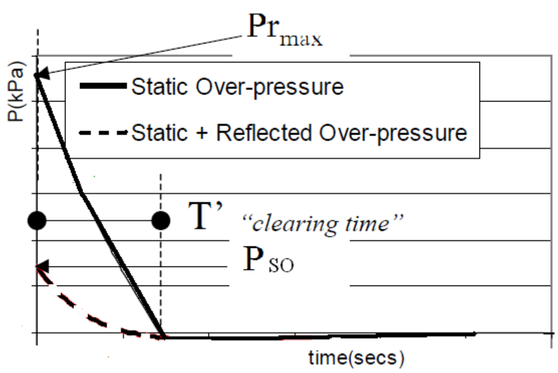

Figure 4.1. Time – pressure relationship

An important parameter in the reflected over-pressure is the "clearing time T" which defines the time taken for the reflected over-pressure to decay completely. The response of structure is dependent on the relationship of natural frequency of structure and "clearing time T''.

$$T' = 3S/U \qquad (8)$$

where S is minimum dimension on the frontal surface of the blast and U is the blast front velocity.

5 Computational model

There is analyzed shear wall of 21 storey residential building [11]. Analyzed wall is 60m high, 12m wide and the thickness of wall is 0,2m. Vertical load was modeled as additional node mass from element MASS21 from ANSYS element library. Vertical load is considered according to EN 1991 for residential buildings. Reinforcement concrete wall was modeled from 3D element SOLID65. Element SOLID65 from ANSYS element library is intended for modeling of nonlinear behavior of reinforced concrete structures. Willam and Wärnke material model of concrete is associated to SOLID65 element in ANSYS program. There was realized 78 RSM simulations for probabilistic and sensitivity analysis of the wall. Probabilistic analysis was compared with deterministic model. There was used Newmark time integration for solving the problem of structural dynamics and Newton-Raphson method for solving the problem of material nonlinearity.

5.1 Uncertainties of input data

The uncertainties of the input data were considered in accordance of the Eurocode and JCSS requirements [3 and 5]. The characteristics of the input data are presented in the table 5.1.

Table 5.1. Probabilistic model of input parameters

Characteristic	Material strenght	Elastic modulus	Density
Variabil. const.	f_var	m_var	g_var
Histogram	Normal	Normal	Normal
Mean values	1	1	1
Stand. deviation	9.96E-02	9.96E-02	0.10
Characteristic	Distance R	Model uncertainty	Resistance uncertainty
Variabil. const.	dis_var	e_var	r_var
Histogram	Normal	Lognormal	Normal
Mean values	1	1	1
Stand. deviation	0.10	0.10	9.97E-02

5.2 Blast load from explosion 100kg TNT distanced 40m from building

The blast load were defined as the time function in the form of the triangular diagram (fig.5.1) based on requirements of Smith [12].

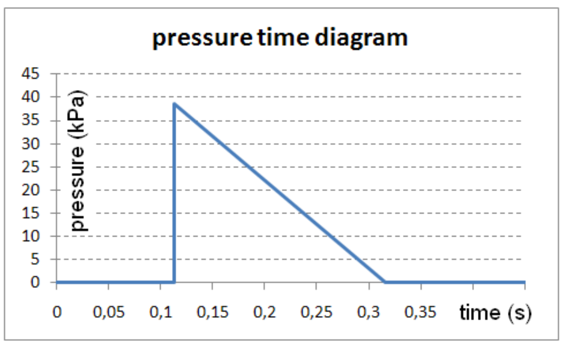

Figure 5.1. Pressure-time diagram of blast load

6 Analysis results

6.1 Criterion of damage limitation

Damage limitation of the reinforced concrete structures depend on the criterion of the maximum inter-storey drifts.

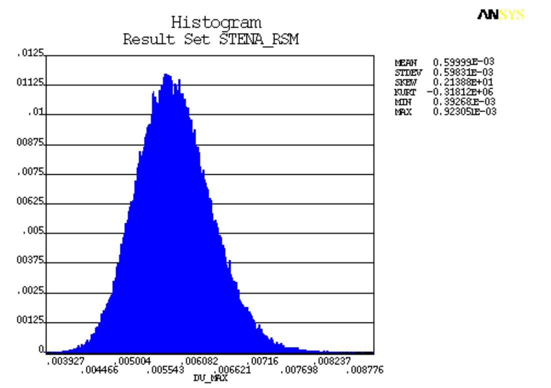

Figure 6.1. Histogram of output parameter d_E

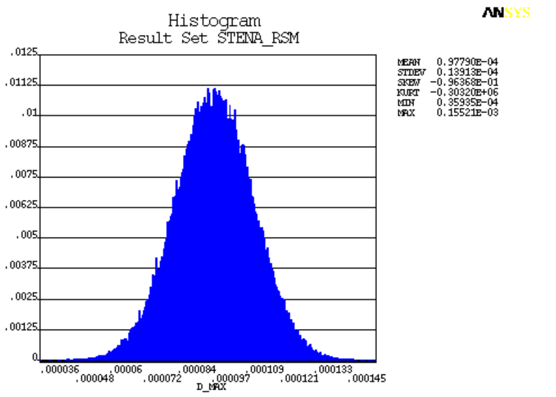

Figure 6.2. Histogram of output parameter d_R

The standard ENV 1998 define the function of failure in the form:

$$g(d)=1-d_E/d_R \geq 0 \tag{9}$$

where d_E is inter-storey horizontal displacement, d_R is limit value of inter-storey horizontal displacement defined in the form:

$$d_R=0{,}005.h/\nu \tag{10}$$

where h is storey height (h = 3m) and ν is reduction factor to take into account the lower return period of the seismic (blast) action associated with the damage limitation requirement.

6.2 Comparison of deterministic and probabilistic results

Table 6.1. Comparison of deterministic and probabilistic analysis

Method	Maximal interstorey drift (x 10^{-4})			
	Min	Max	Mean	St.dev
Deterministic	-	-	1,03	-
Probabilistic	0,36	1,15	0,98	0,1391
Failure probability	-	-	<1.10^{-6}	-

Method	Limit value of maximal interstorey drift (x10^{-4})			
	Min	Max	Mean	St.dev
Deterministic	-	-	500	-
Probabilistic	392,6	923,0	599,9	5,98
Failure probability	-	-	<1.10^{-6}	-

7 Sensitivity analysis

Sensitivity analysis of the influence of the variable input parameters to the reliability of the structures depends on the statistical independency between input and output parameters.

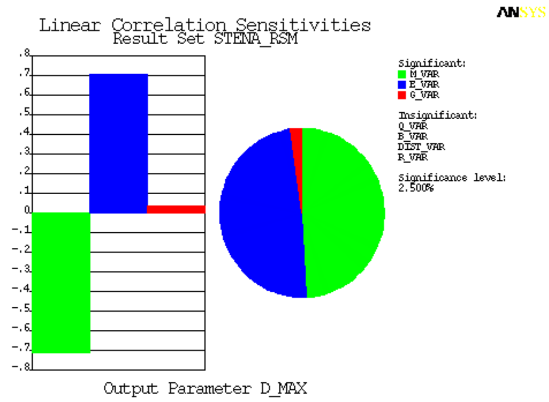

Figure 7.1. Sensitivity analysis for the reliability of the inter-storey drift

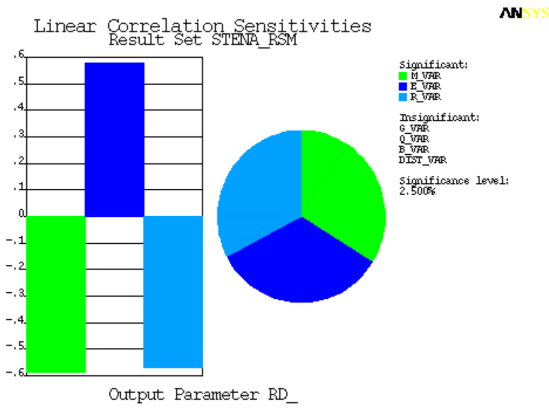

Figure 7.2. Sensitivity analysis for the reliability of the the function of failure

As is shown on Fig. 7.1, the main influence for interstorey drift resistance has the variability of input parameter of Young modulus of concrete, then parameter of model uncertainty and variability of density of concrete. The sensitivity analysis gives the valuable information about the influence of uncertainties of input variables (load, material, model,) to engineer for optimal design of the structures. As is shown on Fig. 7.2, the main influence for function of failure has the variability of input parameter of Young modulus of concrete, then parameter of model uncertainty and variability of parameter of resistance uncertainty.

8 Conclusion

This paper presents the methodology of dynamics analysis of the concrete shear wall on the base of deterministic and probabilistic assessment. There is presented nonlinear material model and stochastic and deterministic solution of resistance of reinforced concrete wall under shock wave. On the example of the sensitivity analysis the efficiency of the probabilistic analysis to optimal design of high rise buildings was presented. The results suggest that, the wall has due capacity to withstand the blast load.

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Primary Paper Section: J

Secondary Paper Section: JN, KA