PROBABILISTIC ASSESSMENT TO ANALYSE OF SOIL STRUCTURE INTERACTION OF TALL BUILDINGS

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DOI: 10.35181/tces-2022-0010

Abstract. *This paper presents the results from the deterministic and probabilistic analysis of the accidental torsional effect on seismic resistance of the tall building. The methodology of the seismic analysis of the structures in Eurocode and JCSS is discussed. The possibilities of the utilization the RSM and LHS method to analyse the extensive and robust tasks in FEM is presented. The influences of the local site effects – masses, stiffness, and thickness of the layered subsoil - can significantly modify the stresses and deflections of the structural system. The influence of the uncertainties of the input and output parameters is considered. The deterministic and probability analysis of the seismic resistance of the structure was calculated in the ANSYS program.*

Keywords

Probability, Safety, Seismic, SSI, Building, ANSYS.

1. Introduction

During the structural design process, an engineers must consider problems of the safety, reliability, and durability of a single structural element as well as the entire structure from the point of view of its planned life cycle. Randomness in the loading and the environmental effects, the variability of the material and geometric characteristics of structures, subsoil and many other "uncertainties" affecting errors in the computing model led to a situation where the actual behaviour of a structure is different from the modelled one [1-4]. Recent advances and the general accessibility of information technologies and computing techniques give rise to assumptions concerning the wider use of the probabilistic assessment of the reliability of structures using simulation methods [5-9].

Most problems concerning the reliability of building structures are defined today as a comparison of two stochastic values [2, 4, 5], loading effects E and the resistance R, depending on the variable material and geometric characteristics of the structural element. The variability of those parameters is characterized by the corresponding functions of the probability density $f_R(r)$ and $f_E(e)$. In the case of a deterministic approach to a design, the deterministic (nominal) attributes of those parameters R_d and E_d are compared.

The deterministic definition of the reliability condition has the form

$$R_d \geq E_d \tag{1}$$

and in the case of the probabilistic approach, it has the form [4, 5, 8]

$$RF = R - E \geq 0 \tag{2}$$

where RF is the reliability function, which can be expressed generally as a function of the stochastic parameters X_1, X_2 to X_n, used in the calculation of R and E.

$$RF = g(X_1, X_2, ..., X_n) \tag{3}$$

The failure function $g(X)$ represents the condition (reserve) of the reliability, which can either be an explicit or implicit function of the stochastic parameters and can be single (defined on one cross-section) or complex (defined on several cross-sections, e.g., on a complex finite element model).

The most general form of the probabilistic reliability condition is given as follows:

$$p_f = P(R - E < 0) \equiv P(RF < 0) < p_d \tag{4}$$

where p_d is the so-called design ("allowed" or "acceptable") value of the probability of failure. From the analytic formulation of the probability density by the functions $f_R(r)$ and $f_E(e)$ and the corresponding distribution functions $\Phi_R(x)$ and $\Phi_E(x)$, the probability of failure can be defined in the general form:

$$p_f = \int_{-\infty}^{\infty} dp_f = \int_{-\infty}^{\infty} f_E(x)\Phi_R(x)dx = \int_{-\infty}^{\infty} \Phi_E(x) f_R(x)dx \tag{5}$$

This integral can be solved analytically only for simple cases; in a general case it should be solved using numerical integration methods after discretization.

In the case of simulation methods, the failure probability is calculated from the evaluation of the statistical parameters and theoretical model of the probability distribution of the reliability function $RF = g(X)$. The failure probability is defined as the best estimation on the base of numerical simulations in the form [2, 5, 10]

$$p_f = \frac{1}{N}\sum_{i=1}^{N} I\left[g\left(X_i\right) \le 0\right] \quad (6)$$

where N in the number of simulations, $g(.)$ is the failure function, $I[.]$ is the function with value 1, if the condition in the square bracket is fulfilled, otherwise is equal to 0.

Variation of the failure function can be defined by Melchers [10] in the form (7)

$$s_{p_f}^2 = \frac{1}{(N-1)}\left\{\frac{1}{N}\left[\sum_{i=1}^{N} I^2\left[g\left(X_i\right) \le 0\right]\right] - \left[\frac{1}{N}\sum_{i=1}^{N} I\left[g\left(X_i\right) \le 0\right]\right]^2\right\}$$

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, Eurocode 1 recommends a 3-level reliability analysis. 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. Three categories of methods have been presently realized [5, 11]:

- **Direct methods** (Importance Sampling - IS, Adaptive Sampling - AS, Direct Sampling - DS)
- **Modified methods** (Conditional, Latin Hypercube Sampling - LHS)
- **Approximation methods** (Response Surface Method - RSM)

The advantages and disadvantages of these methods are described in detail in the book [5]. The ANSYS Program belongs among the complex programs for solving potential problems. It contains a postprocessor, which enables the execution of the probabilistic analysis of structures. There is presented the procedural diagram sequence from the structure of the model through the calculations, up to an evaluation of the probability of structural failure. The ANSYS postprocessor enables the modelling of a structure as a solid body having a general shape (solid modelling), using Boolean operations, general spline planes (non-uniform rational B-splines), automated meshing and adaptive meshing. The postprocessor enables the displaying of numerical results and, using APDL language (ANSYS Parametric Design Language), the compiling of the numerical results obtained. The postprocessor for the probabilistic design of structures enables the definition of random variables using standard distribution functions (normal, lognormal, exponential, beta, gamma, Weibull, etc.), or externally (user-defined sampling) using other statistical programs like AntHILL or FReET [5, 6]. The probabilistic calculation procedures are based on Monte Carlo simulations (DS, LHS, user-defined sampling) and "Response Surface Analysis Methods (RSM)" (CCD, BBM, user-defined sampling). The statistical postprocessor compiles the results numerically and graphically in the form of histograms and cumulative distributional functions. The relations between input (X_i) and output data (Y_i) are defined by the approximation function

$$\hat{Y} = c_o + \sum_{i=1}^{NRV} c_i X_i + \sum_{i=1}^{NRV}\sum_{j=1}^{NRV} c_{ij} X_i . X_j \quad (8)$$

where c_o is the constant term, c_i is the linear term and c_{ij} is the quadratic term of this approximation function, which will be determined from the optimal solution (Montgomery, Myers) [12] or using the regression analysis after response calculation (Neter). The principal advantages are that the number of the simulations is significantly smaller and there's are independent of each other, and thus parallel calculations can be used.

3. Sensitivity analysis

The sensitivity analysis of the influence of the variable input parameters to the seismic response is based on the statistically dependency between the input and output parameters [5, 13]. Matrix of correlation coefficients of the input and output parameters is defined by Spearman in the form

$$r_s = \frac{\sum_{i=1}^{n}\left(R_i - \bar{R}\right)\left(E_i - \bar{E}\right)}{\sqrt{\sum_{i=1}^{n}\left(R_i - \bar{R}\right)^2}\sqrt{\sum_{i=1}^{n}\left(E_i - \bar{E}\right)^2}} \quad (9)$$

where E_i is rank of input parameters within the set of observations $[X_i]^T$, R_i is rank of output parameters within the set of observations $[Y_i]^T$, are average ranks of the parameters R_i and E_i respectively.

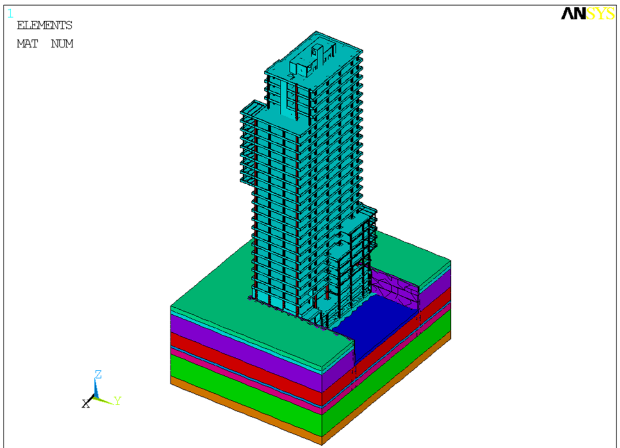
4. Calculation model of tall building

The RSM methodology was used for the probabilistic analysis of the effect of soil-structure interaction during the seismic excitation in case of the asymmetric tall administrative building in region Bratislava (Fig. 1). In the case of this building, the complicated foundation conditions were considered from the point of view of the optimal foundation of the building [5, 7, 14].

This tall building CBC has floor plan dimensions of 40x22 m and a height of 98.3 m. The building has 3 underground floors with a foundation joint at the level of -11.1 m.

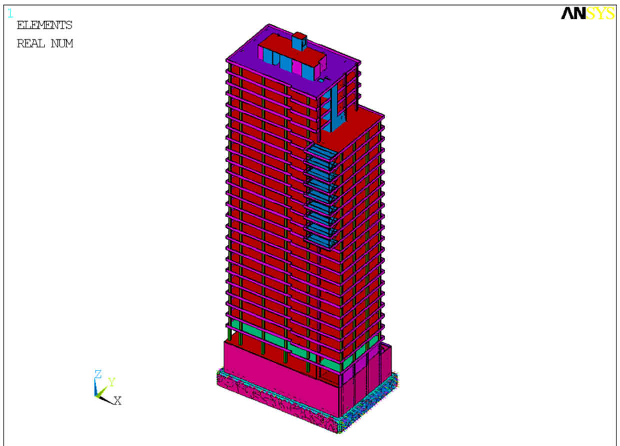
The building is based on a monolithic foundation slab with a thickness of 1.9 m.

Fig. 1: Administrative building CBC in Bratislava.



CBC1. Administrative building CBC Bratislava

Fig. 2: Calculation model CBC1 with 3D soil model.



CBC3. Administrative building CBC Bratislava

Fig. 3: Calculation model CBC3 with 3D soil model.

The structural system of the CBC (City Business Center) building is designed as a combined system consisting of two reinforced concrete monolithic cores and a system of columns and flat slab. In the lower part of the building, the reinforced concrete walls are designed around the perimeter from three sides of the building, except for the module in axis 5, where a low-rise building is connected to the building.

The geological and seismotectonic evaluation of the site is based on the geological survey. The geotechnical properties of the subsoil under the high-rise building are characterized by probes VS1, VS2, VS9 and VS10. The subsoil consists of Quaternary soils in the upper layers above the level of about 18 to 21 m below the ground and with layers of Neogene soils in the depths below this level. The gravel formations were located at a depth of about 2.8 to 7.1 m below the ground. The groundwater level is below the level of about 5.3 m to 6.1 m below the ground.

The basic conditions are characterized as complicated due to the variable thickness of individual soil types and high groundwater levels.

In terms of seismic hazard of the building, the subsoil is classified according to [15] in the category of soils *B* in accordance with Eurocode 8 [15].

Tab.1: The original static material properties of the subsoil layers.

Layer	h_i	γ	E_{def}	ν	Mat.
	[m]	[kNm^{-3}]	[kPa]		number
1	1.97	26.25	155625	0.26	11
2	1.39	25.30	11314	0.49	12
3	7.74	25.98	9452	0.51	13
4	2.70	20.00	15331	0.43	14
5	3.50	19.00	17810	0.42	15
6	1.00	23.97	12729	0.46	16
7	12.30	19.00	10466	0.38	17
8	4.50	19.00	18692	0.35	18
9	2.70	20.00	102383	0.20	22

Tab.2: The modified static material properties of the reinforced subsoil layers.

Layer	h_i	γ	E_{def}	ν	Mat.
	[m]	[kNm^{-3}]	[kPa]		number
1	1.97	26.25	155625	0.26	11
2	1.39	25.30	11314	0.49	12
3	7.74	25.98	9452	0.51	13
4	2.70	23.05	75735	0.20	23
5	3.50	22.59	33748	0.40	24
6	1.00	25.00	900000	0.20	25
7	12.30	19.00	10466	0.38	17
8	4.50	19.00	18692	0.35	18
9	2.70	20.00	102383	0.20	22

Due to the complicated subsoil geological profile, the average values from individual probes were considered in the calculation model considering the uncertainties of these values in the probabilistic analysis.

The origin subsoil in the layers 4-6 has the lower material properties. These layers were problematic from the view of the stability of the building in case of the wind and seismic impact. These subsoil layers were modified.

The building pit was upgraded by a reinforced concrete tub created by Keller-type compression grouting with walls anchored to the surrounding body by micropyles.

The dynamic properties of the subsoil were considered on base of the seismic monitoring of shear wave speed at this locality.

Four calculation models were created with different subsoil stiffness

CBC1 – subsoil discretized by 3D elements (SOLID64) in following scope $-21.0\text{m} \leq X \leq 59.2\text{m}$; $-21.0\text{m} \leq Y \leq 44.8\text{m}$; $0.0\text{m} \leq Z \leq -34.8\text{m}$, The FEM model has 220 164 elements.

CBC2 - subsoil discretized by 3D elements (SOLID64) in following scope $-21.0\text{m} \leq X \leq 21.0\text{m}$; $-11.0\text{m} \leq Y \leq 11.0\text{m}$; $0.0\text{m} \leq Z \leq -6.0\text{m}$, under the level -6.0m the subsoil was modeled by Winkler soil with $k = 1500$kPa/m‘ (SURFACE154), the wall of reinforced concrete basic bathtub was discretized by shell elements (SHELL43), the FEM model has 79 711 elements.

CBC3 - subsoil discretized by 3D elements (SOLID64) in following scope $-21.0\text{m} \leq X \leq 21.0\text{m}$; $-11.0\text{m} \leq Y \leq 11.0\text{m}$; $0.0\text{m} \leq Z \leq -6.0\text{m}$, under the level -6.0m the subsoil was modeled by Winkler soil with $k = 3000$kPa/m‘ (SURFACE154), the wall of reinforced concrete basic bathtub was discretized by shell elements (SHELL43), the FEM model has 79 711 elements.

CBCR – rigid subsoil was considered for the comparison the influences of the various subsoil models.

The dynamic properties of the soil layers (Tab. 3) were determined using the results of the experimental test in publication [16].

Tab.3: The comparison of the static and dynamic properties of the soil

Soil	E_{stat} [MPa]	E_{dyn} [MPa]
Incoherent soils		
Loose sand, round	40-80	150-300
Loose, square sand	50-80	150-300
Sand moderately flat, rounded	80-160	200-500
Fine gravel and sand	100-200	200-500
Lean soil, gravel	100-200	300-800
	150-300	300-800
Cohesive soils		
Cohesive clays and clays	3-50	100-500
Semi-solid clays and clays	6-20	40-150
Compacted clays	3-6	30-80
Solid clays	6-50	100-500
Fine clays, loess,	4-8	50-150
Clay loess	3-8	30-100
Muddy soils, clays, org.	2-5	10-30

Tab.4: The static and dynamic modulus ratio [16]

Soil	E_{dyn}/E_{stat}	E_{stat} [MPa]
Incoherent soils	2.5÷4.0	40-300
Cohesive soil	2.0÷50.0	6-30
Rock	6.0÷60.0	60-700

In the case of a seismic event. the soil body in the subsoil is disturbed and the layers in the subsoil are partially plasticized. because of which it is necessary to consider a reduction in the dynamic characteristics of the soil. Based on the recommendation of Eurocode 8 [15] it is possible to consider a reduction at the level of 80% of the maximum values.

Tab.5: Damping values depended on the acceleration value S_a/g

S_a/g	Damping ξ	$v_s/v_{s.max}$	G/G_{max}
0.1	0.03	0.9(±0.07)	0.80(±0.10)
0.2	0.06	0.7(±0.15)	0.50(±0.20)
0.3	0.10	0.6(±0.15)	0.33(±0.20)

5. Seismic hazard of locality

The seismic hazard of the site is defined in accordance with the recommendations of STN EN 1998/NA [15].

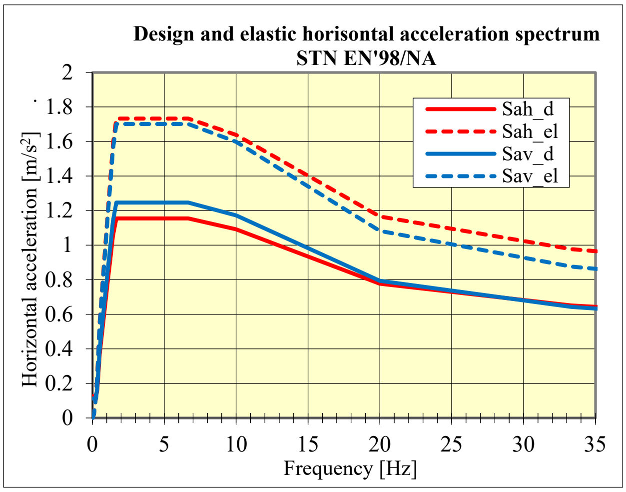


Fig. 4: Acceleration spectrum according to STN EN 1998/NA

The site is in the source area of seismic risk with a basic value of seismic acceleration $a_r = 0.63$ ms^{-2}. Based on the seismic and geological characteristics of the locality, we classify the area into the category of subsoil type *B*.

The design seismic acceleration a_g is $a_g = \gamma_I.a_r = 1.0\text{x}0.63 = 0.63\text{ms}^{-2}$. The elastic and design spectrum for both horizontal (S_{ah}) and vertical direction (S_{av}) is presented in fig. 4.

6. Load and load combinations

The load and load combination in the case of a deterministic assessment of the ultimate limit state of the structure is considered according to STN EN 1991-1 as follows

D1) Permanent and temporary design situations

$$E_d = \sum_{j\geq 1} \gamma_{Gj} G_{kj} "+" \gamma_P P_k "+" \gamma_{Q1} Q_{k1} "+" \sum_{i\geq 1} \gamma_{Qi} \psi_{0i} Q_{ki} \quad (10)$$

D2) Seismic design situation

$$E_d = \sum_{j\geq 1} G_{kj} "+" P_k "+" \gamma_1 A_{Ed} "+" \sum_{i\geq 1} \psi_{2i} Q_{ki} \quad (11)$$

where G_{kj} is the characteristic value of permanent loads. P_k - characteristic value of prestressing load. Q_{k1} - characteristic value of prevailing variable load. Qki - characteristic value of extraordinary load. A_{Ed} - design value of seismic load. γ_{Gj} - partial factor for permanent load. γ for prestressing load. γ_{Qi} - partial factor for variable load i. γ_1 - coefficient of significance (building structure). ψ - coefficients of combinations (according to STN EN 1991-1).

In case of the probabilistic analysis the following load combinations are considered in accordance with [18]

P1) Permanent and temporary design situations

$$E = \sum_{j\geq 1} g_{\mathrm{var}.j} G_{kj} "+" p_{\mathrm{var}} P_k "+" q_{\mathrm{var}1} Q_{k1} "+" \sum_{i\geq 1} q_{\mathrm{var}.i} Q_{ki} \quad (12)$$

P2) Seismic design situation

$$E = \sum_{j\geq 1} \overline{g}_{\mathrm{var}.j} G_{kj} "+" \overline{p}_{\mathrm{var}} P_k "+" \overline{a}_{\mathrm{var}} A_{Ed} "+" \sum_{i\geq 1} \overline{q}_{\mathrm{var}.i} Q_{ki} \quad (13)$$

where g_{var}. q_{var}. a_{var} ($\overline{g}_{\mathrm{var}}, \overline{p}_{\mathrm{var}}, \overline{a}_{\mathrm{var}}, \overline{q}_{\mathrm{var}}$) are the variable parameters defined in the form of the histogram calibrated to the load combination in compliance with Eurocode [18] and JCSS requirements [12].

7. Modal and spectral analysis

The modal analysis of the CBC object was performed on a spatial finite element model using the iterative LANCZOS method in the ANSYS program [5]. Boundary conditions at the level of the foundation joint were considered as the fixed connection with the solid (rock) subsoil. 500 natural frequencies were solved on the CBC model with a modal mass content of 99.9% in the X direction, 99.7% in the Y direction and 99.9% in the Z direction.

Tab.6: The significant eigenvalues of the CBC models

Modal results			CBC1	CBC2	CBC3	CBCR
Direction	X	Frequency [Hz]	0.28	0.35	0.41	0.50
		Part. Factor [%]	37.00	44.00	41.40	36.10
	Y	Frequency [Hz]	0.25	0.26	0.29	0.33
		Part. Factor [%]	20.80	40.70	38.70	35.50
	Z	Frequency [Hz]	1.89	1.95	2.81	4.04
		Part. Factor [%]	72.90	95.20	89.30	27.00

The significant eigenvalues of the individual models in the X, Y and Z directions are presented in the tab. 6.

The stiffness of reinforced concrete walls and piles with subsoil modelled by 3D elements in CBC1 model has a significant effect on the frequency characteristics of the object (Tab. 6). Substrate stiffness changes in the case of the CBC2 and CBC3 models do not have a significant effect on the decisive natural frequencies of the object in the horizontal direction. These frequencies can be considered as the acceptable frequencies of the object in the horizontal direction.

The eccentric arrangement of the object's mass resulting from the asymmetry of the object is manifested by the rotation of the object in the case of decisive oscillation shapes in the XZ and YZ planes.

The seismic analysis of the tall building's structure was realized using the linearized response spectrum method. This method allows an approximate determination of the maximum response of an MDOF system (Multi Degrees of Freedom Systems) without performing a time history analysis. The response spectrum method is based on the solution of dynamic equation by modal superposition method in time.

The dynamic equation for MDOF system with n-DOF due to support excitation is defined in the form

$$\boldsymbol{M}(\ddot{\boldsymbol{u}} + \ddot{\boldsymbol{u}}_s) + \boldsymbol{C}\dot{\boldsymbol{u}} + \boldsymbol{K}\boldsymbol{u} = \boldsymbol{0}, \quad (14)$$

where **M**, **C**, **K** are matrix (n x n) of the mass. damping and stiffness, $\boldsymbol{u}$, $\dot{\boldsymbol{u}}$, $\ddot{\boldsymbol{u}}$ are vectors (n x 1) of relative displacements. velocities and accelerations, $\ddot{\boldsymbol{u}}_s$ is vector (n x 1) of support accelerations (seismic excitation). After transformation the equations (14) to the modal coordinate system by next substitution

$$\mathbf{u} = \sum_{i=1}^{m} \boldsymbol{\Phi}_i . \mathbf{Y}_i \,, \quad (15)$$

we obtain the m- independent equations of motion in the form

$$\ddot{\mathbf{Y}}_i + 2.\xi_i.\omega_i.\dot{\mathbf{Y}}_i + \omega_i^2.\mathbf{Y}_i = -\Gamma_i.\ddot{\mathbf{u}}_g, \quad (16)$$

where $\boldsymbol{\Phi}_i$ is an eigenvector (m x 1) for mode i after normalization of mass matrix $\boldsymbol{\Phi}_i^T \mathbf{M} \boldsymbol{\Phi} i = 1$, $\mathbf{Y}_i$ is a modal coordinate vector (m x 1). ξ_i is relative damping for i-mode, ω_i angular frequency i-mode, $\boldsymbol{\Gamma}_i$ is participation factor for i-mode in the form

$$\mathbf{u}_i = \boldsymbol{A}_i \boldsymbol{\Phi}_i \quad (17)$$

where $\mathbf{A}_i$ is a mode coefficient, $\boldsymbol{\Phi}_i$ is an eigenvector.

Participation factor $\mathbf{A}_i$ depends on the accelerogram on the base

$$\boldsymbol{A}_i = \frac{\boldsymbol{S}_{ai} \Gamma_i}{\boldsymbol{\omega}_i^2} \quad (18)$$

where $\mathbf{S}_{ai}$ is an acceleration spectrum for i-mode and defined damping, $\boldsymbol{\Gamma}_i$ is a participation factor for i-mode, $\boldsymbol{\omega}_i$ is natural angular frequency for i-mode.

The response spectrum of the displacements and forces from the excitation in direction a = 1, 2, 3 is calculated from the modal response by method square root of sum of squares mode (SRSS) in the form

$$R_a = \left[\sum_{i=1}^{N} (R_i)^2 \right]^{\frac{1}{2}}, \qquad (19)$$

The total response spectrum is calculated from three base acceleration spectra (in space) alternatively from the combination SRSS or standard combination rule [14]

$$R_{\mathrm{tot}} = R_1 + 0.3R_2 + 0.3R_3 \quad \text{or} \quad R_{\mathrm{tot}} = 0.3R_1 + 0.3R_2 + R_3 \quad \text{or}$$

$$R_{\mathrm{tot}} = 0.3R_1 + R_2 + 0.3R_3 \qquad (20)$$

where R_i (i =1, 2, 3) are response values from the acceleration excitation in the direction 1, 2, 3.

The modal and spectral analysis of these models of buildings was realized on the software ANSYS.

8. Uncertainties of the input data

In the case of a probabilistic approach of load, subsoil stiffness and structural strength uncertainty [2, 10, 11], they can be effectively investigated by sensitivity analysis. In the probabilistic and sensitivity analysis of the CBC building. the uncertainties of load and resistance are expressed depending on the characteristic values of input variables and variable parameters based on the recommendations of JCSS [12] and ASCE 7/95 [1] (Tab. 7).

Tab.7: The probability model of the soil parameters.

Quantity	Soil stiffness		
Charact. value	$k_{z.k}$	$k_{z.k}$	$k_{z.k}$
Variable	$k_{z.var}$	$k_{xx.var}$	$k_{yy.var}$
Histogram	Normal	Normal	Normal
Mean value	1	1	1
Deviation	0.200	0.033	0.033
Min. value	0.148	0.851	0.853
Max. value	1.867	1.163	1.135

Variable values of subsoil stiffness are based on the results of geological survey the determined profile under the base slab area and on the design of subsoil improvement by the KELLER system ($k_{z.k}$ = 1.5MPa /m). The variability of the subsoil stiffness in the horizontal plane is expressed by the variables of the global rotations $k_{xx.var}$. $k_{yy.var}$ of the object as a rigid whole.

The random distribution of the subsoil stiffness is approximated with bilinear function on the horizontal plane in dependency on three parameters $k_{z.var}$, $k_{xx.var}$, $k_{yy.var}$

$$k(x,y) = k_{z,k}\left(k_{z.\mathrm{var}} + \frac{(x-x_o)}{10*L_x} k_{yy.\mathrm{var}} + \frac{(y-y_o)}{10*L_y} k_{xx.\mathrm{var}} \right) \qquad (21)$$

where x_o, y_o are coordinates of building foundation slab gravity center, L_x and L_y are the plane dimensions of the foundation slab in directions x and y.

The uncertainties of the calculation model were considered by the variable parameters of the load (tab. 8) as well as the bending resistance $m_{u.var}$ and the model variability f_{var} ($\mu = 1, \sigma = 0{,}1$) according to the normal distribution.

Tab.8: The probability model of the load parameters.

Quantity	Load		
	Permanent	Variable	
	Dead	Live	Seismic
Characteristic value	G_k	Q_{k1}	S_k
Variable	g_{var}	q_{var}	s_{var}
Histogram	Normal	Beta (I)	Beta (I)
Mean value	1	0.57	0.67
Deviation	0.033	0.28	0.14
Min. value	0.569	0.858	0.40
Max. value	1.385	1.127	1.20

The variability of the stiffness of the subsoil in the horizontal plane is expressed through the variables of global rotations of the object as a rigid whole. The uncertainties of the calculation model were considered by the variable parameters of model uncertainties and load effects according to the Gaussian normal distribution.

9. Reliability criteria for seismic resistance of structure

Reliability of the structures is designed in accordance of the standard requirements [15, 16] for ultimate and serviceability limit state. The foundation slab was designed on the bending and shear loads for ultimate limit state function in the next form

$$g(M) = 1 - M_E/M_R \geq 0, \quad g(V) = 1 - V_E/V_R \geq 0 \qquad (22)$$

where M_E, V_E are design bending moment and design shear force of the action and M_R, V_R are resistance bending moment and resistance shear force of the structure element.

The damage limitation of the tall buildings depends on the criterion of the maximum interstorey drifts. The standard STN EN 1998 [15] define the function of failure in the form

$$g(d) = 1 - d_E/d_R \geq 0 \qquad (23)$$

where d_E is interstorey horizontal displacement, d_R is limit value of interstorey horizontal displacement defined (for non-structural elements of brittle materials attached to the structure) in the form

$$d_R = 0.005 * h/\nu \qquad (24)$$

where h is storey height (h = 3m) and ν is the reduction factor (ν = 0.4) to consider the lower return period of the seismic action and the damage limitation requirement.

Based on the RSM methodology, relations between input and output data are defined by the approximation function (8). The response surface of the resistance function RD is shown in fig. 5.

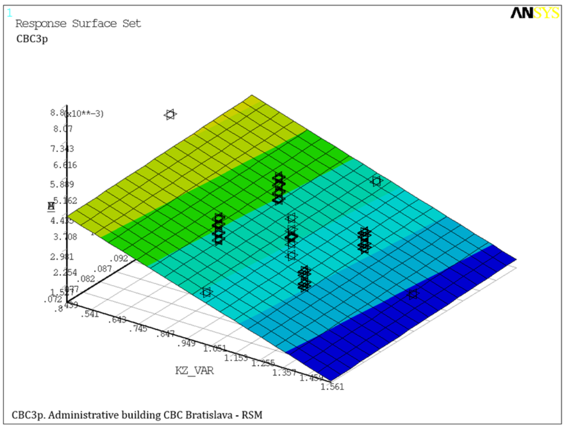


Fig. 5: Response surface area of the resistance function RD = g(d)

The sensitivity analysis of the influence of the variable input parameters to the seismic response is based on the Spearman methodology (9). The sensitivity analysis of the resistance function *RD* shown in fig. 6 and the resistance function *RM* shown in fig. 7.

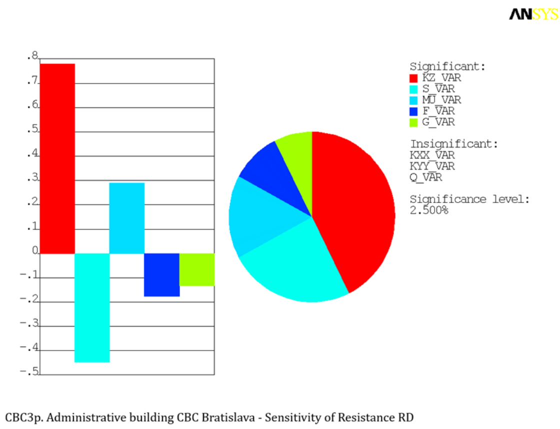


Fig. 6: Sensitivity analysis of the resistance function RD = g(d)

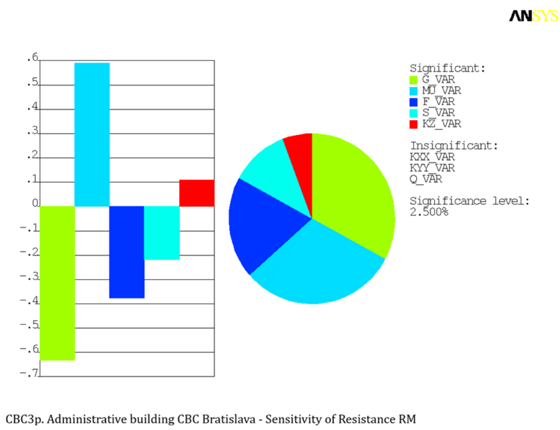


Fig. 7: Sensitivity analysis of the resistance function RM = g(M)

The histograms of the resistance function *RD* (Fig. 8) and *RM* (Fig. 9) are calculated using the Monte Carlo simulations following the results from the RSM method.

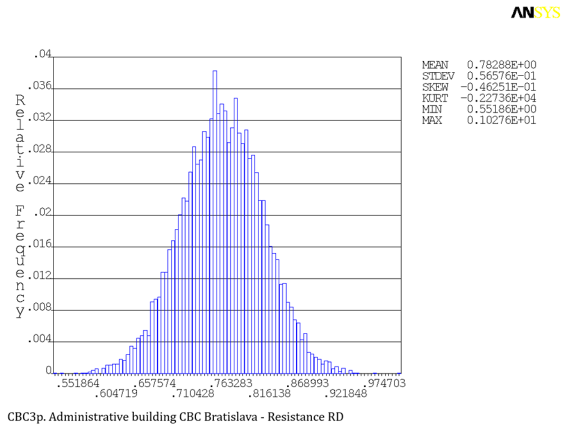


Fig. 8: Histogram of the resistance function RD = g(d)

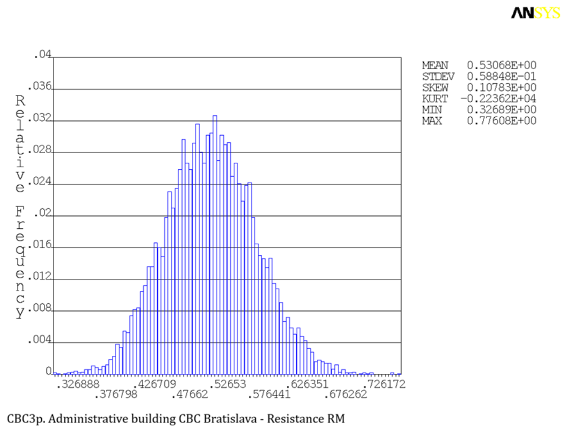


Fig. 9: Histogram of the resistance function RM = g(M)

The results of the sensitivity analysis of the damage limitation *RD* (23) show us that the variability of the soil stiffness ($k_{z.var}$) and the seismic load (s_{var}) are dominant. In case of the foundation slab the variability of the dead load (g_{var}) and slab resistance ($m_{u.var}$) are dominant using function *RM* (22).

10. Conclusion

This paper presented the methodology of the seismic analysis of the tall building structures considering soil-structure interaction on the base of deterministic and probabilistic assessment. This analysis was realized on the example of the administrative building CBC in Bratislava. The uncertainties of the calculation model were considered by the variable parameters of model uncertainties and load effects according to the Gaussian normal distribution. The geological and seismotectonic evaluation of the site was based on the geological survey. The basic conditions are characterized as complicated due to the variable thickness of individual soil types and high groundwater levels. The variable values of subsoil stiffness are based on the results of geological survey the determined profile under the base slab area and on the design of subsoil improvement by the KELLER system ($k_{z.k}$ = 1.5MPa /m). The variability of the stiffness of the subsoil in the horizontal plane is expressed through the variables of global rotations of the object as a rigid whole. The probabilistic analysis was based on the RSM methodology. The advantages and disadvantages of the various probabilistic methods were presented in this paper. The principal effect of the probabilistic methodology is that in the case of the complicated subsoil conditions we have the information what uncertainty parameter is dominant for the optimal design of the structure. The results of the sensitivity analysis give us the important information to design the effective structures.

Acknowledgements

This article was created with the support of the Ministry of Education of Grant Agency of the Slovak Republic (grant VEGA No. 1/0453/20).

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