

# Static and dynamic analysis of high-rise building with consideration of two different values of subsoil stiffness coefficients

*Olga Ivankova*<sup>1,\*</sup>, *Janka Kovacikova*<sup>1</sup> and *Lenka Konecna*<sup>1</sup>

<sup>1</sup>Slovak University of Technology in Bratislava, Faculty of Civil Engineering, Department of Structural Mechanics, Radlinskeho 11, SK-810 05 Bratislava, Slovakia

**Abstract.** This paper deals with the analysis of 21-storeyed cast in-situ reinforced concrete high-rise building. Two different 3D models were created, because of two considered values of subsoil stiffness coefficient - fixed structure (alt. 1) and the structure supported by elastic soil (alt. 2). For both alternatives of foundation of structure, required analyses (static and dynamic) were done and obtained results were compared in this paper. Short description of the structure, applied loads and other input parameters are also mentioned here. The main purpose of this analysis was to provide more information to planning engineers about the behaviour of structure exposed the wind load or seismic load when different soil conditions were considered.

## 1 Description of analysed structure

Investigated high-rise office building (Fig. 1) had 21 overground floors and 3 underground floors used as parking spots. The superstructure of the building was designed as a combination of cast in-situ reinforced concrete skeleton system with one stiffening core inside. Maximum dimensions of typical floor were (45.5×37.7) m. Total height of the building was 79.2 m over the ground. Under the structure, foundation slab with total dimensions (46.7×38.9) m and the thickness of 1.2 m was designed. In the place of stiffening core, the thickness of foundation slab was changed in the slope of 45°. Then, the final thickness was 1.7 m.

The thickness of reinforced concrete walls placed on the boundaries of underground floors was 300 mm. The walls of stiffening core had various thicknesses: 200, 250 and 300 mm. All columns had squared cross-sections with varied lengths of edge. From the 3<sup>rd</sup> underground floor up to the 6<sup>th</sup> overground floor, the length of edge was 900 mm. From the 7<sup>th</sup> to the 15<sup>th</sup> overground floor, it was 700 mm. From the 16<sup>th</sup> to last overground floor (21<sup>st</sup>), the length of edge was 550 mm. The slabs were designed as the combination of beamless slabs with the headers in the places of columns. The thickness was different in the case of underground floors (300 mm) and overground floors (270 mm). The thickness of headers was 100 mm. On the boundaries of slabs, the beams with the width 450 mm and the

---

\* Corresponding author: [olga.ivankova@stuba.sk](mailto:olga.ivankova@stuba.sk)

height of 650 mm were designed. They were used for the fixing of façade panels. Structural height was also different. The height of all underground floors was 3.3 m, but the height of 1<sup>st</sup> overground floor was 4.0 m. Other overground floors had the same structural height 3.6 m. The main considered material of the structure (reinforced concrete) was the combination of steel B500B with the classes of concrete C30/37, C35/45 and C45/55.

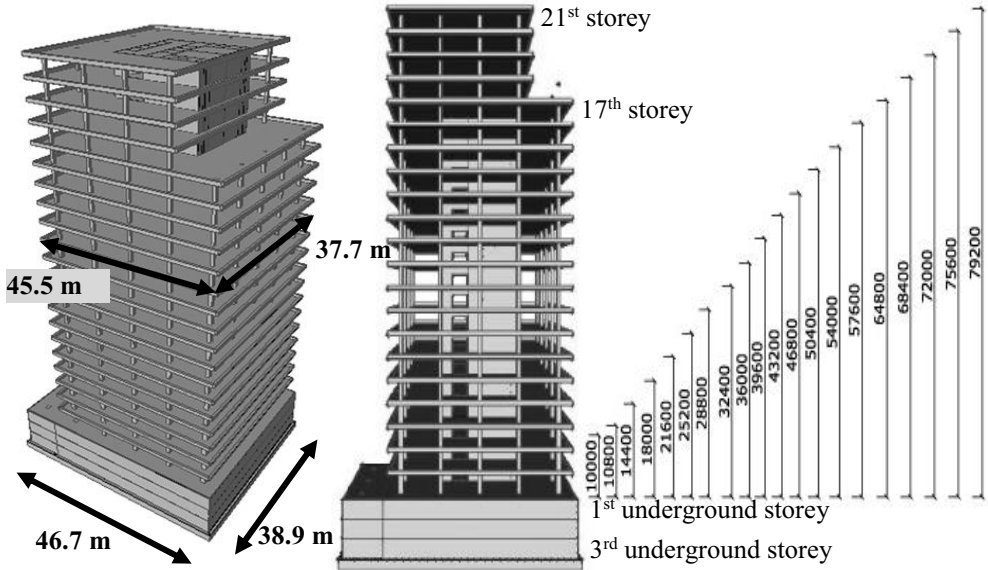


Fig. 1. The geometry. and total dimensions of analysed high-rise building.

## 2 Considered input parameters and the solution of the computing model

The following vertical and horizontal loads were applied on the structure. In the case of vertical loads, permanent loads (the self-weight of the structure, the weight of floor layers, the weight of roof layers) and variable loads (category B for office purpose and the category F for parting spots, partition walls) according to [1, 2] were taken into account. The snow load was calculated according to [3]. For the calculation of the wind load according to [4], the following input parameters were used: fundamental wind velocity  $v_{b,0} = 26$  m/s; the coefficient of season  $c_{season} = 1.0$ ; the coefficient of prevailing wind  $c_{dir} = 1.0$ ; the coefficient of orography  $c_o = 1.0$  and the coefficient  $k_r = 0.234$  for terrain category IV (built-up areas where the minimum 15 % of the area is covered by the high-rise buildings with total heights more than 15 m - centres of big cities). For the assessment of the structure, external and internal wind pressure coefficients were considered. Wind load was applied on the structure as horizontal load in  $x$ -direction and also in  $y$ -direction.

For the dynamic analysis, seismic load according to [5] was considered. Investigated structure was classified to the 2<sup>nd</sup> class of importance (the buildings with significant seismic response such as schools, halls, cultural institutions, etc.). Reference peak acceleration  $a_{gR} = 0.4$  m/s<sup>2</sup> and the category of soil B were used in the calculation. It has to be noted, that the ground plan of the building was asymmetric. Also, the building was asymmetric in regard to its height (see Fig. 1).

Mentioned loads with adequate partial ratio were used in the combinations of the loads required for the assessment of the structure (Ultimate Limit State and Serviceability Limit State) [6].

As it was mentioned above, two models of the same structure were created because of considered different value of subsoil stiffness coefficient. In the 1<sup>st</sup> alternative, the structure was fixed and the value of subsoil stiffness coefficient  $k$  was  $\infty$ . In the 2<sup>nd</sup> alternative, the structure was supported by the elastic soil and the value of  $k_z$  [kN/m<sup>3</sup>] was calculated using Eq. 1 (Winkler's one parametric hypothesis), where  $p$  is the contact stress in foundation gap [kPa] and  $s$  is the value of settlement of the structure [m].

$$k_z = p/s \tag{1}$$

Resultant value of subsoil stiffness coefficient for z-direction was  $k_z = 8\,484$  kN/m<sup>3</sup>. According to recommendations given in [7], for other two directions ( $x$  and  $y$ ), a half of this value can be considered. Hence, resultant value was  $k_x = k_y = 4\,242$  kN/m<sup>3</sup>. These values were used for static analysis.

However, for dynamic analysis, new value of subsoil stiffness coefficient had to be calculated using Eq. 2, where  $E_{dyn}$  is dynamic modulus of elasticity of the soil [kPa] (considered value was 150 000 kPa) and  $\nu$  is Poisson's ratio of the soil [-] (considered value was 0.02).

$$k_z = E_{dyn}/(1 - \nu^2) \tag{2}$$

Resultant values were:  $k_z = 21\,347$  kN/m<sup>3</sup> and  $k_x = k_y = 10\,178$  kN/m<sup>3</sup>. By the consideration of increased value of subsoil stiffness coefficients, the consolidation of the soil during the dynamic event can be expressed. In this case, the resultant value was 2.5 times larger than calculated value used for static analysis. The literature recommends the increasing of this value for dynamic analysis in range of 2.5-5.

For the solution of this structure, 3D model was created and solved by using the software Scia Engineer. The average size of the spatial/curved element was set as 1 000 mm in the case of static analysis and 2 000 mm in the case of dynamic analysis. Minimum length of beam element was set as 100 mm and maximum value was set as 100 000 mm.

### 3 Static analysis – wind effects on the high-rise building

In the case of static analysis, the influence of the wind on the stiffness of structure was analysed in this paper. From the most non-favorable combination of the permanent load, variable load and wind load were calculated maximum horizontal deflections. Wind load was applied on the structure in two directions (from the left side and also from the right side) in both axis  $x$  and  $y$ . Obtained maximum horizontal deflections are listed in Table 1.

**Table 1.** Maximum values of horizontal deflections due to wind load.

Direction of applied load	Subsoil stiffness coefficient [kN/m <sup>3</sup> ]		Maximum values of deflections [mm]		Increase of deflection [mm]
	Alt. 1	Alt. 2	Alt. 1	Alt. 2	
max. X	$\infty$	$k_z = 8\,484$ and $k_x = k_y = 4\,242$	15.2	35.1	19.9
min. X			14.9	28.5	13.6
max. Y			13.6	23.6	10.0
min. Y			3.9	5.5	1.6

These values were compared with the limit value calculated using Eq. 3.

$$\Delta H \leq u_{lim} H \quad (3)$$

Where  $\Delta H$  is calculated maximum horizontal deflection of the structure [m],  $H$  is total height of analysed structure measured from the foundation to the top [m] (in our case  $H = 90.3$  m), and  $u_{lim}$  is the limit value defined in [8] (in our case this value was 1/2000). Resultant value  $u_{lim} H$  was 45.15 mm.

All calculated values were less than the limit value. From this result is evident that if the stiffer subsoil ( $k$  equalled to  $\infty$ ) is considered horizontal deflections are smaller. Hence, it is very important to consider right value of subsoil stiffness coefficient for the calculation because it affects the assessment of the structure.

### 3 Dynamic analysis – seismic effects

For this assessment, the seismic design combinations of the loads were used. It means, that the permanent loads (self-weight and other permanent loads) with the partial ratio equalled to 1.0 were considered. Variable loads (considered according to utilization of internal spaces – the category B for office and the category F for parking spots) with partial ratio equalled to 1.0 and the factor defining representative value of variable actions for combination value  $\psi_2$  equalled to 0.3 (for B) and 0.6 (for F) were used. Snow load and wind load were not to be considered. Seismic load was considered in all three directions ( $x$ ,  $y$  and  $z$ ) with using the factor 1.0 or 0.3 according to requirements in [5].

Calculated maximum horizontal deflections from the most non-favorable combination of the loads are listed in Table 2. It is evident from the comparison of the values, that elastic soil caused the larger values of the horizontal deflections.

**Table 2.** Maximum values of horizontal deflections due to seismic load.

Direction of applied load	Subsoil stiffness coefficient [kN/m <sup>3</sup> ]		Maximum values of deflections [mm]		Increase of deflection [mm]
	Alt. 1	Alt. 2	Alt. 1	Alt. 2	
max. X	$\infty$	$k_z = 21\,437$ and $k_x = k_y = 10\,718$	18.40	22.5	15.9
min. X			35.00	44.1	11.1
max. Y			24.70	28.8	20.9
min. Y			29.80	38.8	25.8

Maximum values of horizontal deflections listed in Table 2, were compared with the limit value calculated using Eq. 3 and  $u_{lim}$  defined in [9] ( $u_{lim} = (1/500)H$ ). Resultant value was 180.6 mm.

Also, maximum horizontal deflection between two slabs was compared with the limit value defined as 1% of structural height (4.0 for 1st overground floor or 3.6 m for other overground floors). In the case of alt. 1 (fixed structure), the maximum deflection between two slabs was 2.8 mm. In the case of alt. 2 (elastic soil), it was 3.9 mm. Limit value used for the assessment was 36 mm, because mentioned values were calculated between last two slabs.

### 4 Modal analysis – eigen frequencies

For the modal analysis, it was required to satisfy two conditions:

- the sum of effective mass considered eigen modes of vibration had to be equalled to 90 % of total mass of structure or more,
- all eigen modes with effective mass larger than 5 % had to be taken into account.

In this case, these conditions were not to be satisfied, therefore other two conditions (Eqs. 4 and 5) were considered.

$$k \geq 3\sqrt{n} \tag{4}$$

where  $k$  is considered number of eigen modes and  $n$  is the number of floors of the structure over the ground. Hence,  $k \geq 3\sqrt{21} \Rightarrow k \geq 13.75$ .

$$f_n \geq 5 \text{ Hz} \tag{5}$$

where  $f_n$  is eigen frequency of  $n^{\text{th}}$  eigen modes.

Calculated eigen frequencies for both considered alternatives (fixed structure and the structure placed on elastic soil) are compared in Table 3. The values of effective mass for selected eigen modes are listed in Table 4. From the comparison of eigen frequencies is evident that the stiffness of soil had no significant influence on the change of the value of eigen frequencies.

**Table 3.** Calculated eigen frequencies – alt. 1 and alt. 2.

n	Alt. 1	Alt. 2	n	Alt. 1	Alt. 2	n	Alt. 1	Alt. 2	n	Alt. 1	Alt. 2
	f [Hz]	f [Hz]		f [Hz]	f [Hz]		f [Hz]	f [Hz]		f [Hz]	f [Hz]
<b>1</b>	0.69	0.539	<b>9</b>	3.896	3.656	<b>17</b>	5.726	5.66	<b>25</b>	6.779	6.768
<b>2</b>	0.7	0.544	<b>10</b>	3.977	3.784	<b>18</b>	5.905	5.77	<b>26</b>	7.016	6.839
<b>3</b>	1.253	1.234	<b>11</b>	4.038	3.9	<b>19</b>	6.021	5.816	<b>27</b>	7.041	6.942
<b>4</b>	2.47	2.389	<b>12</b>	4.172	4.168	<b>20</b>	6.318	5.981	<b>28</b>	7.123	7.034
<b>5</b>	2.924	2.71	<b>13</b>	4.379	4.348	<b>21</b>	6.427	6.325	<b>29</b>	7.137	7.067
<b>6</b>	3.065	2.821	<b>14</b>	4.51	4.375	<b>22</b>	6.479	6.34	<b>30</b>	7.186	7.12
<b>7</b>	3.629	3.16	<b>15</b>	4.942	4.966	<b>23</b>	6.632	6.557			
<b>8</b>	3.722	3.581	<b>16</b>	5.28	5.218	<b>24</b>	6.768	6.742			

**Table 4.** Effective mass – alt. 1 and alt. 2.

Alt. 1					Alt. 2				
n	f	$\frac{W_{xi}}{W_{xtot}}$	$\frac{W_{yi}}{W_{ytot}}$	$\frac{W_{zi}}{W_{ztot}}$	n	f	$\frac{W_{xi}}{W_{xtot}}$	$\frac{W_{yi}}{W_{ytot}}$	$\frac{W_{zi}}{W_{ztot}}$
	[Hz]	[%]	[%]	[%]		[Hz]	[%]	[%]	[%]
1	0.69	<b>50.5</b>	2.8	0.0	1	0.54	0.0	<b>54.8</b>	0.0
2	0.70	2.8	<b>50.0</b>	0.0	2	0.54	<b>54.7</b>	0.0	0.0
6	3.07	0.0	15.0	0.4	4	2.39	0.6	0.4	13.4
7	3.63	0.1	0.0	15.2	5	2.71	16.3	0.2	0.1
9	3.90	0.1	0.1	7.5	6	2.82	0.2	15.5	1.2
12	4.17	0.0	0.0	8.2	7	3.16	0.0	0.1	<b>63.2</b>
22	6.48	0.1	0.0	<b>20.5</b>	22	2.39	0.6	0.4	13.4

## 5 Conclusions

From the obtained results is evident, that considered value of subsoil stiffness coefficient have a significant influence on the behaviour of analysed structure. Hence, it is very important to take into account real soil parameters when the structure like this (high-rise building) is modelled and analysed. From these parameters, the subsoil stiffness coefficient is calculated. Its value affects horizontal deflections of structure caused by wind load or seismic load. These values have to be compared with the limit values defined by standards or recommendations in literature. If too large value of subsoil stiffness coefficient is used, horizontal deflections are smaller and the structure can satisfy the limit values, but it does not match with reality. On the other hand, if the value of subsoil stiffness coefficient is small and calculated horizontal deflections are too large, but we can suggest the possibilities how to make a structure stiffer. By this solution, we can prevent failures of the structure (and also damaging of non-bearing structures) in the future.

It has to be noted, that the value of subsoil stiffness coefficient is not the same during the whole life of structure. By the consolidation of the soil, its value is changed and the soil becomes stiffer. Winkler's one parametrical model of soil, used also in this paper, is only simplified calculation and it gives the value of subsoil stiffness coefficient in the beginning of building of structure. Therefore, for the detailed analysis of behaviour of structure, it should be useful to calculate changed value of subsoil stiffness coefficient with respect to the time (lifetime of structure).

The change of the value of subsoil stiffness coefficient did not have significant influence on the value of eigen frequencies. In both alternatives, calculated eigen frequencies had to be similar.

This paper was supported by the Grant Agency of Ministry of Education, Science, Research and Sports of the Slovak Republic - the grants No. 1/0544/15 and No. 1/0265/16. Authors thank to the Slovak University of Technology for the support - grant No. 1651.

## References

1. STN EN 1990. Eurocode: Design of structures, SUTN, Bratislava, 2011.
2. STN EN 1991-1-1. Eurocode 1: Actions on structures. Part 1-1: General actions. Densities, self-weight, imposed loads for buildings, SUTN, Bratislava, 2007.
3. STN EN 1991-1-3. Eurocode 1. Actions on structures. Part 1-3: General actions. Snow loads, SUTN, Bratislava, 2012.
4. STN EN 1991-1-4. Eurocode 1: Actions on structures. Part 1-4: General actions. Wind actions, SUTN, Bratislava, 2010.
5. STN EN 1998-1. Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings, SUTN, Bratislava, 2005.
6. J. Bilcik, L. Fillo, V. Benko, J. Halvonik, Concrete structures: designed according to EN 1992-1-1, second ed., Publishing House of STU in Bratislava, Bratislava, 2008. ISBN 978-80-227-2940-6. (in Slovak)
7. N. Jendzelovsky, *Modelling of foundations by FEM* (in Slovak), (Publishing House of STU in Bratislava, Bratislava, 2009)

8. I. Harvan, *Concrete structures. High-rise buildings, design according to European standards* (in Slovak), (Publishing House of STU in Bratislava, Bratislava, 2015)
9. STN 73 1201. Design of concrete structures, SUTN, Bratislava, 1986