23rd International Conference
ENGINEERING MECHANICS 2017
Svratka, Czech Republic, 15 – 18 May 2017

A HIGH-RISE BUILDING - INFLUENCE OF CHANGE OF SOIL STIFFNESS ON HORIZONTAL AND VERTICAL DEFLECTIONS

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Abstract: *This paper deals with static analysis of 17-storey high-rise building with complicated design of slabs above the 1st and the 2nd overground levels. An influence of settlement of the structure in time was taken into account with various values of the subsoil stiffness coefficients. For their calculations, two-parametric model of subsoil was used for modeling of soil-structure interaction. Short description of analyzed structure, applied loads and other input parameters are mentioned. At the end of the paper, obtained results for static analysis (deflections of the structure due to combinations of the permanent, variable and wind loads) are presented.*

Keywords: Soil stiffness, Static analysis, High-rise building, Reinforced concrete structure, Deflections.

1. Description of the structure

Analyzed 17-storey high-rise building (Fig. 1a) was designed as an office building. In bottom part, there were 3 storeys designed as parking spots. Structural height was 3700 mm and total height of the building from the foundation to the top was 72.5 m.

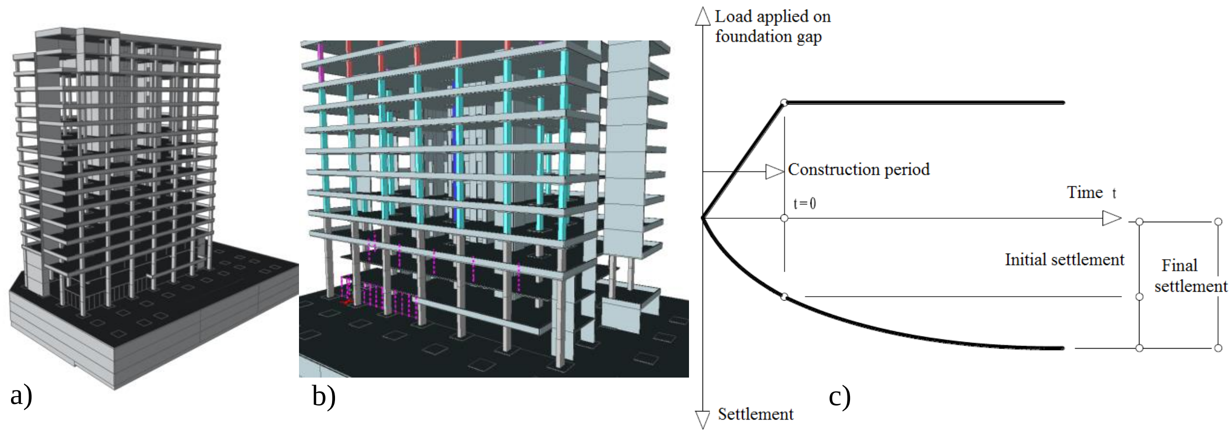

Fig. 1: a) Analyzed high-rise building, b) view of the 1st and 2nd overground levels, c) time dependence of settlement of structure (Simek, 1990).

Cast in-situ reinforced concrete superstructure was designed as a combination of columns with beamless slabs and central stiffening core located in the center of gravity of ground plan of a typical floor. The grid of load-bearing walls and columns had the dimensions of (7.8 × 8.1) m. The foundation slab was designed as a one dilatation block with the dimensions of (75.175 × 52.200) m and various thickness, because applied load (axial force) on columns in underground part under the high-rise building was larger than it was in the case of the columns placed in other places of underground part. Therefore, the thickness of foundation slab under the high-rise building was 1000 mm added by column capital with the thickness of

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1600 mm. In other places of the underground part, the thickness of foundation slab was 400 mm added by column capital with the thickness of 1000 mm. The walls of stiffening core had various thicknesses with respect to the change of the height (200 - 400 mm) and they were made of the concrete C30/37. The columns in the high-rise building were made of concrete C40/50. The columns in underground part were made of concrete C30/37. The slab of typical floor (4^{th}-17^{th} overground levels) was designed as beamless slab with the thickness of 200 mm added by visible planked capitals with the thickness of 350 mm (together with slab). The same design was used for the slabs above the 1^{st} and 2^{nd} underground floors.

Atypical design of superstructure was in the part between the underground levels and overground levels. With the respect to the requirement of an architect to highlight the entrance space, the slabs above the 1^{st} and 2^{nd} floor had atypical shapes and they were not supported by all columns placed on the boundaries of ground plan (Fig. 1b).

The slab above the 1^{st} floor was supported by steel sections on its boundaries. They were a part of glass façade and they were anchored to the slab above 3^{rd} underground level. The slab above 2^{nd} floor was suspended by draw rods on its boundaries. These draw rods were anchored to the slab above 3^{rd} floor. Then, the lengths of columns located on the boundaries of slabs on 1^{st}, 2^{nd} and 3^{rd} floor, were two times or three times of the structural height (Fig. 1b).

For the solution of 3D computing model, the Spatial Deformation Variant of Finite Element Method using 1D and 2D elements, introduced in Scia Engineer Software, was used.

2. Applied loads and considered input parameters

Permanent and variable loads were considered according to (STN EN 1991, Bilcik 2008). For the snow load, the height above the sea level of building site 139.4 m was taken into account (STN EN 1991-1-3). For the wind load, terrain category IV (big cities) and 2^{nd} wind area with the basic wind velocity 26 m/s were considered (STN EN 1991-1-4). For the analysis, the following geological profile was considered: 0.0 - 4.0 m made-up ground, 4.0 - 14.90 m gravel poorly grained (classified as G2 (Turcek, 2004)), 14.90 - 20.00 m clay with middle plasticity (classified as F6). The level of underground water was 6.60 m.

Soil-structure interaction was considered by the calculated value of subsoil stiffness coefficient. It defines compliance of soil during insertion of structure (or its parts) to the soil. According to Winkler's hypothesis, there is direct proportional dependency between the load applied on foundation gap and its deflection. Then, the subsoil stiffness coefficient k [kN/m^3] can be calculated as:

$$k = \frac{p(x,y)}{w(x,y)} \tag{1}$$

where $p(x,y)$ is contact stress [kN/m^2] and $w(x,y)$ is deflection [m] (Jendzelovsky, 2009). Resultant value of subsoil stiffness coefficient was 7150 kN/m^3. This value represents the material properties of soil before the loading of foundation gap. Therefore, other two values of subsoil stiffness coefficient were calculated (Frankovska, 2011 and Simek, 2009).

The value of subsoil stiffness coefficient after the primary soil consolidation $k_{z,red,I}$ can be calculated using Eq. 2. The total time of primary soil consolidation was estimated as 1 year (it is time needed for building such structure).

$$k_{z,red,I} = k_{z,eff} e^{(\varphi_0 - \varphi_{0z})} \tag{2}$$

where $k_{z,eff}$ is subsoil stiffness coefficient calculated by Winkler's hypothesis (Eq. 1) [kN/m^3]. In our case it was $k_{z,eff}$ = 7150 kN/m^3. φ_0 is final creep coefficient of concrete (Harvan, 2006). φ_{0z} is final creep coefficient of soil (for normally consolidated clay can be considered by φ_{0z} = 1.0 (Frankovska, 2011)). Resultant value of subsoil stiffness after the primary soil consolidation was 19500 kN/m^3.

Compliance of the soil in the time when the settlement of structure will be finished, can be expressed by Eq. 3.

$$k_{z,red,II} = k_{z,eff} (1 + \varphi_{0z}) e^{(\varphi_0 - \varphi_{0z})} \tag{3}$$

Where $k_{z,red,11}$ is subsoil stiffness coefficient [kN/m^3] after final settlement of structure, $k_{z,eff}$ is subsoil stiffness coefficient calculated by Winkler's hypothesis (Eq. 1) [kN/m^3] (considered by 7150 kN/m^3). φ_0 (they were described thereinbefore). Then, calculated value of $k_{z,red,11}$ was 38900 kN/m^3. Time dependence of settlement of the structure from applied load is shown in Fig. 1c, (Frankovska, 2011), (Simek, 1990).

3. Results – comparison of deflections and its assessment

Obtained results for different directions of applied loads are compared in Figs. 2 - 6. Maximum value of deflection in *z*-direction was considered as 50 mm (STN 73 1001). Maximum value of deflection in *x* and *y*-direction was considered as *H*/2000 (Harvan, 2006), where *H* is the total height of the structure above the foundation (in this case *H* = 72.5 m and *H*/2000 = 36.25 mm). For this assessment, characteristic values of load were used. Contact pressure in subsoil area is 353.83 kPa.

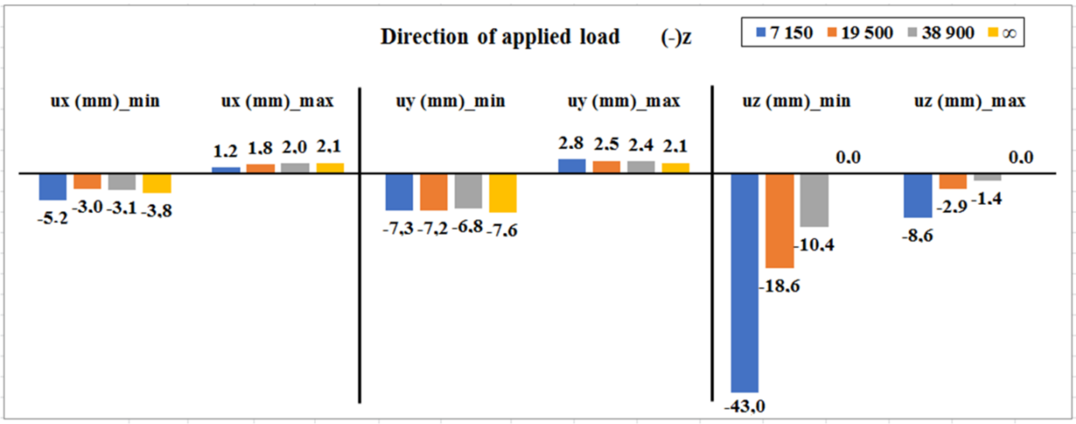

Fig. 2: Maximum and minimum values of deflections due to load applied in -z-direction.

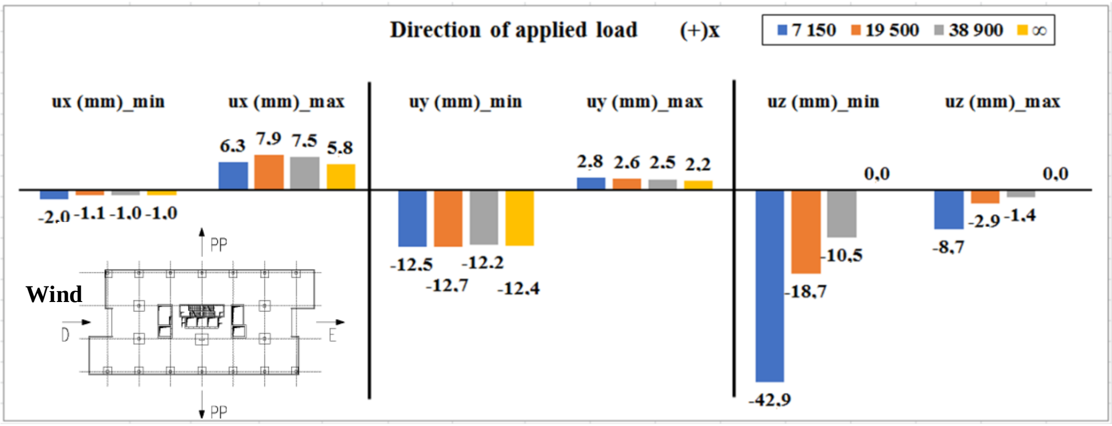

Fig. 3: Maximum and minimum values of deflections due to wind load applied in +x-direction.

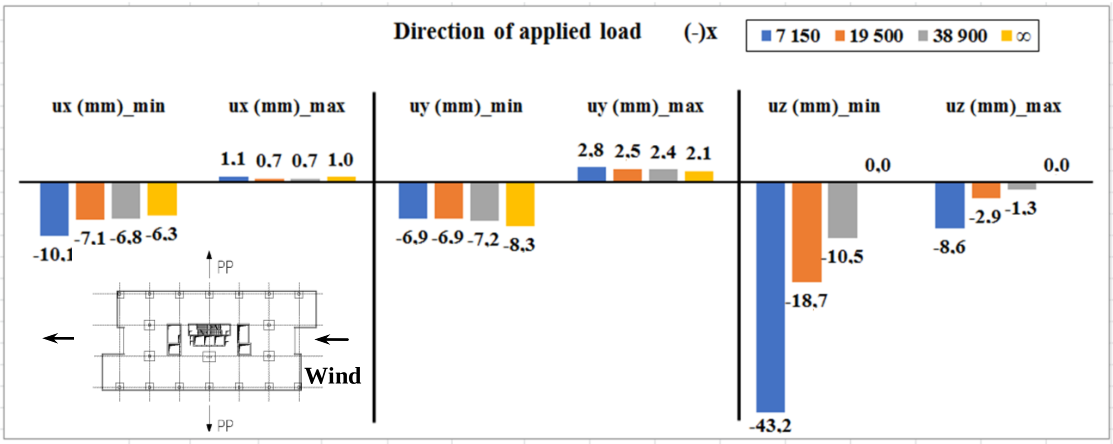

Fig. 4: Maximum and minimum values of deflections due to wind load applied in -x-direction.

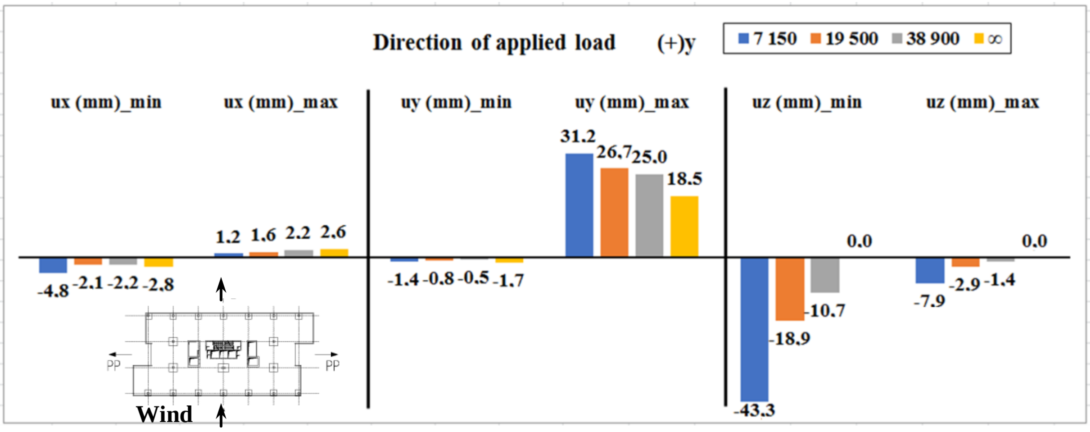

Fig. 5: Maximum and minimum values of deflections due to wind load applied in +y-direction.

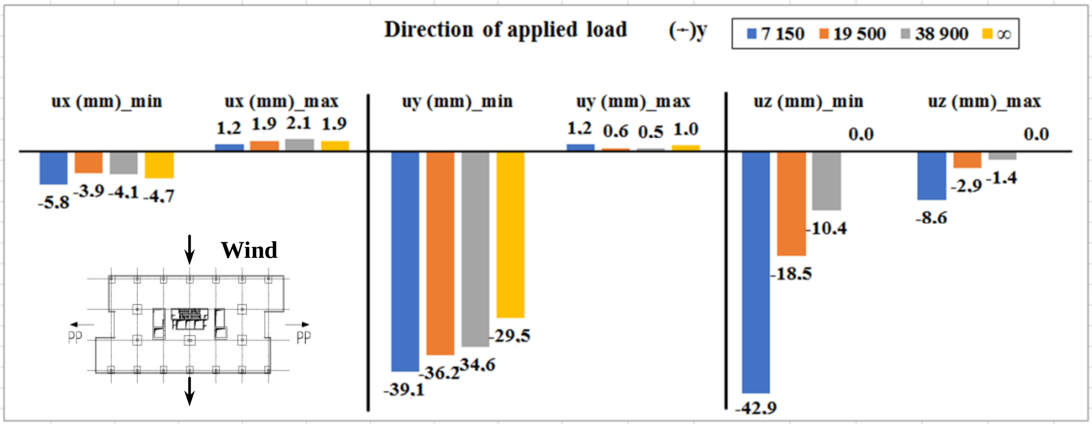

Fig. 6: Maximum and minimum values of deflections due to wind load applied in -y-direction.

4. Conclusions

The building met the requirement for limit value of deflection in z-direction. The problem was in the case of the wind applied in –*y*-direction, when calculated maximum value for subsoil stiffness coefficient equalled to 7150 kN/m^3 was larger than limit value. Assessment of the structure for this value of subsoil stiffness coefficient is not correct, because the construction of structure is not completed. Furthermore, the structure is not loaded by full value of considered load in this time. For the assessment, it is better to take into account the primary soil consolidation, because it helps to increase the stiffness of soil. For the reduction of calculated deflections, it should be necessary to design additional stiffening walls in the ground plan of structure. Then, the structure should satisfy the limit values given by standards.

Acknowledgement

The paper was created with the support of the Scientific Grant Agency – VEGA grants No. 1/0265/16 and No. 01/0544/15. Authors thank to the STU in Bratislava for the support (grants No. 1619 and No. 1651).

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