Structural safety of buildings in excess values of differential settlements

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Abstract. The paper discusses the strength of load-bearing structures in a frameless building in case of excess differential settlements. A numerical simulation of the structural performance of load-bearing and enclosure structures has been made for the tilted block of flats erected in the city of Tyumen at the time of the maximum deviation of its skeleton from the vertical.

1. Introduction
In recent years, multi-storey housing is extensively developing in the Russian Federation and it is often necessary to erect foundations in complex engineering-geological conditions. For this reason, errors occur when choosing the type of foundation and excess absolute and differential settlements appear.

Taking into account geotechnical safety, it is most risky to erect multi-storey buildings on shallow foundations, slab foundations among them, with the beds composed of silt-clay soils. As practice shows, in buildings erected on such soils the actual settlements are often higher than the calculated values [2, 3 and 19]. The settlement itself is not as problematic as its variations which result in the deviation of the building skeleton from the vertical [1, 19 and 20].

The junctions of structural members are the most vulnerable zones in buildings subjected to the differential settlement: columns, party walls of lower floors from the side of extra loading with an additional moment of the tilt; joints of panels, walls, coverings, floor structures, junctions of walls and floor structures [7, 8]. Due to poor-quality erecting works and differential settlements of foundations, deviations from the vertical and misalignments of flat areas can cause redistribution of stresses and result in structural failures [19].

The tilted block of flats located in Belyaeva Street in the city of Tyumen is an example of a significant tilt which has provided a basis for studying the stress-strain state of its superstructure. The building constructed in 2014-2015 consists of two blocks – 9-storey and 14-storey blocks respectively [12].

The slab foundations of both blocks are made of mass concrete B30, 1.0 m in thickness. The outer walls of the substructure are made of FBS panels, 600 mm in thickness, and the inner ones - 500 mm. The brick outer walls of the superstructure are insulated - 820 mm in thickness; the inner walls are 510 mm in thickness. The floor structures are prefabricated and made of reinforced concrete hollow core slabs.

The average pressure under the foundation slab footing of the 14-storey block with the area of $S = 914.36 \text{ m}^2$ is $p^f = 257 \text{ kPa}$ from the specified characteristic load; $p^d = 285 \text{ kPa}$ - from the design load.
The first geological surveys were carried out in 2012, and then control surveys were carried out during building site works in 2014. The comparison data of the results obtained after engineering and geological surveys are given in Table 1. Excavated soil and construction debris occur on the site up to 2.5 meters in thickness, and then soil formations are in accordance with Table 1. The depth of the constant groundwater level is 4.4-5.3 m from the ground surface.

Figure 1. Soil formations.

Table 1. Comparison data of physical-mechanical properties of soils: A – engineering-geological surveys in 2012; B – control engineering-geological surveys in 2014.

| № of a layer | Layer thickness, m | Soils                                      | $E$, MPa | $\gamma$, kN/m$^3$ | $c$, kPa | $\phi$, degrees |
|--------------|-------------------|-------------------------------------------|----------|--------------------|----------|-----------------|
|              | $A = B$           | $A = B$                                   | $A$      | $B$                | $A$      | $B$             |
| 1            | 2.1               | High-plastic clay loams                   | 13.7     | 8.4                | 19.1     | 19.5            | 22 | 18 | 21 | 12 |
| 2            | 4.7               | Fluidal and fluidal-plastic clay loams   | 6.3      | 2.7                | 18.8     | 17.8            | 15 | 10 | 14 | 10 |
| 3            | 4.6               | Alternation of fine sands with sand loams and clay loams | 20.6 | 20.6 | 20.1 | 19.6 | 9 | 9 | 27 | 27 |
| 4            | 11.5              | Fine sandy silt with layers of sand loams and clay loams | 29 | 29 | 20.6 | 20.6 | 3 | 3 | 31 | 31 |

During building site works, both blocks suffered from excess differential settlements which developed rapidly. In here, cracks or failures of the foundation and walls were not observed. Hair-like vertical cracks were found only in the building basement, but deformations of the foundation bed were not caused by them.

Figure 2. Inclination of the building in completion.

Figure 3. Horizontal deviation of the blocks from the vertical in (cm); the maximum permissible values are given in brackets in (cm).
The geodetic survey showed that the facade of the 14-storey block (Figure 2) deviated from the vertical in 660 mm at the maximum value - 9 cm after Building Regulations [16] and 11.5 cm after Building Regulations [17]. The relative difference in settlements reached $\Delta s/L = 0.019 > [\Delta s/L] = 0.0024$ – Building Regulations [17] (Table D1[4]). The 9-storeyed block suffered from a dramatic tilt in two directions (Figure 2).

The differential settlements were caused by the following factors: displacement of the center of gravity of the superstructure towards the inner angle of the building in plan (Figure 3), non-uniform soil formations with thinning of the layers (Figure 3, Table 1), and excavation works done partly in place of the former vegetable storehouse.

![Figure 4. General view of the building design diagram in Lira SOFT 10.6.](image)

![Figure 5. Schematic illustration of the deformed model in Lira SOFT 10.6.](image)

![Figure 6. Horizontal displacements along Y-axis in Lira SOFT 10.6.](image)

These factors were not taken into account at the design stage, and thus, it resulted in the differential settlement and the building tilt respectively.

In such cases, when the differential settlements are significant and exceed the recommended limit values dramatically, structural safety must dominate. To identify the reserve bearing capacity of the superstructure, the following issues were considered:

- extra loading of brick party walls of the ground floor with increase of the tilt;
- appearance of shearing forces in the junctions of floors structures and walls due to tensile forces in the horizontal disks of the building frame with increase of the tilt.

A numerical simulation of the building was created in order to reveal the regularities in the changed stress-strain state of the main load-bearing structures in the excess permissible tilt. The initial data were taken in accordance with the project, the design diagram was compiled in the Lira SOFT 10.6
software package (Figure 4), and the Mohr-Coulomb model of strength was used to simulate the foundation bed. Further, the results of calculation are given only for the 14-storey block. Based on the results of numerical simulation, the average settlement of the foundation was 47.3 cm \( S_u = 10 \) cm, Building Regulations [17] (Figure 5). The relative difference in the settlements of the foundation extreme points was \( \Delta S/L = 0.019 > [\Delta s/L] = 0.0024 \), Building Regulations [17].

The maximum horizontal deviation of the building frame from the vertical along the B-axis was also obtained in the course of the numerical calculation of the 14-storey block. It was 621 mm (Figure 6).

Thus, when analyzing such parameters as maximum settlement, relative variations of settlements and horizontal displacement, the difference from the real deformations obtained from the geotechnical monitoring at the site is not above 6%.

2. Assessment of the stress-strain state of the most loaded brick party wall

In accordance with the Building Regulations [15], the ultimate load-bearing capacity of the 14-storey building brick walls in an upright position is \( N = 3004 \) kN with regard to operational loads. It should be noted that when the building deviates from the vertical, eccentricity \( e \) is added to the real loads \( N \). This results in the extra moment and shear force which appear in the base of the walls.

Based on the results of numerical simulation and design calculations, the reserve load-bearing capacity of the most loaded party wall of the 14-storey block is 27% in the vertical position, with \( N^{0.019} = 2177 \) kN. Taking into account the variations of settlements \( \Delta S/L = 0.019 \), the reserve load-bearing capacity of the most loaded party wall decreases from 27% to 5% (\( N^{0.019} = 2848 \) kN).

To assess structural safety, the building model was given different values of the foundation bed performance to successively increase the difference in settlements. This technique has made it possible to analyze the influence of non-uniform deformations of the foundation bed on the overall stability of the system and assess the structural safety of the building.

![Figure 7. Dependency graph of the reliability factor on stability of the system versus the values of the differential settlements of the foundation bed.](image)
It is generally accepted that the reliability factor on stability of the system for structures calculated using certified software packages should not be less than $\gamma_c = 1.3$ [18]. The system is considered to be unstable when the coefficient approaches $\gamma_c = 1.0$ [18].

Figure 7 shows the dependency graph of the reliability factor on stability of the building system versus the values of the differential settlements of the foundation bed. In accordance with the results of geotechnical monitoring and numerical simulation $\Delta S/L = 0.019$ in relative variations of settlements, the coefficient $\gamma_c = 1.05$ is at least equal to the ultimate value $[\gamma_c] = 1$. After the results of geomonitoring, cracks in the party walls due to deformations were not found given the excess values of the differential settlements. This is confirmed by the results of numerical simulation (Figure 7), and stability of the system is $\gamma_c = 1.05$.

In accordance with the results corresponding to the ultimate values of Building Regulations [17], namely: the value of the relative deformation $[\Delta S/L] = 0.0024$ and horizontal displacement of the building frame $[Y] = 9$ cm [16], the reserve of the load-bearing capacity of the brick walls should be 15%, and stability of the system $[\gamma_c] = 1.18$.

The results of numerical simulation show that the stress in the brick party walls will reach the ultimate load-bearing capacity if the values of the differential settlements are $\Delta S/L = 0.024$ (Figure 7). Deviation of the building frame from the vertical is $Y_{\text{max}} = 83$ cm.

3. Assessment of shear forces in junctions of floor structures and walls

In order to assess the stress-strain state of the floor structure to wall joint from the horizontal tensile stresses arising from the non-uniform deformations of the foundation bed, it is necessary to determine the stress $N_{\text{max}}$ when the floor structure breaks away from the wall [13].

In order to solve this problem, a 3D model of the floor structure to brick wall joint was created in a numerical software package.

The construction was approximated by the 3D FE-36 - universal spatial eight-joint isoparametric finite elements. The horizontal tensile stresses between the floor structure slabs were set with regard to the stages of variations in settlements (Figure 8).

![Figure 8. Distribution of stresses in the floor structure of the 14th floor along X-axis (p.3), along Y-axis (p.4) in Lira Soft 10.6](image)

The tensile stresses in the floor structure slabs of the 14th floor were assessed; the relative variations in settlements - $i_1 = 0.0024$, $i_2 = 0.007$, $i_3 = 0.019$ [11].

Since the graph in Figure 9 is a linear relationship, it is necessary to find the horizontal stress $N_{\text{max}}$, and then determine the relative variations in settlements which refer to this stress.

$$N_{\text{max}} = N_c + N_o$$

where $N_c$ – longitudinal compressive force at the bearing stress zone; $N_o$- crimping force;
where \( A_0 \) – bearing surface of the brick work onto the slab; 
\[
\tau = \sigma \cdot t g \varphi + c
\]
where \( \sigma \) – normal stresses from the brick work proper weight

\[
\varphi = \arcsin \left( \frac{R - R_b}{R + R_b} \right); 
C = \frac{R_b \cdot (1 - \sin \varphi)}{2 \cdot \cos \varphi}; \quad [10]
\]

\[
N_c = K_{ct} \cdot \psi \cdot d \cdot R_c \cdot A_c \quad [15]
\]

\[\text{Figure 9.} \quad \text{Dependency graph of the horizontal stress in the floor structure versus relative variations of settlements}\]

According to the results of calculation \( N_c = 8.5 \text{ ton-meters.} \quad N_0 = 3.6 \text{ ton-meters.} \)

Thus, \( N_{\text{max}} = 8.5 + 3.6 = 12.1 \text{ ton-meters.} \quad \Delta S/L = 0.025 \) in such force value from the graph (Figure 9).

4. Conclusion
These two controlling cases have made it possible to conclude that the loss of the load-bearing capacity of the party wall is most likely to occur with the relative variations of settlements \( i = 0.024 \) and deviation of the building frame from the vertical by \( Y = 830 \text{ mm,} \) and then the floor structure will possibly break away from the wall due to the horizontal force if \( i = 0.025 \) and \( Y = 870 \text{ mm.} \)

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