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Soil-structure interaction calculations as a basis of interactive monitoring at construction and operation

Les calculs combinés des fondations et des structures pour une surveillance interactive pendant la construction et l'exploitation

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ABSTRACT: The paper is devoted to organization of the monitoring at operation of the skyscraper "Lakhta-center" in St. Petersburg. The monitoring system is based on the model of soil-structure interaction calculation allowing monitoring to be interactive. Special attention is paid to "alarm signals" which provide timely warning of emergence of negative trends, helping to exclude occurrence of a failure state. It has been shown that calculations for substantiation of the criteria of monitoring systems have significant differences from conventional calculations at design. The paper formulates the features of an efficient monitoring system, which completely corresponds to the system implemented in practice. There have been compared the logs of measurement equipment installed in structures and the design expectations. There has been shown on the example of the high-rise building how to set the criteria of "alarm" and "failure" to provide serviceability of the monitoring system at operation.

RÉSUMÉ: L'article est consacré à la mise en place de la surveillance de la tour « Lakhta Center » à Saint-Petersbourg pour la période de son exploitation. Le système de surveillance s'appuie sur le modèle de conception du bâtiment qui interagit avec les fondations ce qui permet une surveillance interactive. Une attention particulière est portée aux « alarmes » qui permettent de signaler en temps opportun l'émergence de toute tendance négative et prévenir ainsi des situations d'urgence. Il est démontré que les calculs servant à justifier les critères de surveillance diffèrent considérablement des calculs traditionnels utilisés dans la conception du bâtiment. Les critères d'un système de surveillance opérationnels sont formulés. Le système mis en pratique y est entièrement conforme. Les indications des instruments de mesure installés dans les structures sont comparées aux attentes prévisionnelles. Sur l'exemple d'un immeuble de grande hauteur, il est démontré comment attribuer les critères d'« alarme » et d'« accident » pour assurer un bon fonctionnement du système de surveillance lors de l'exploitation.

KEYWORDS: monitoring at operation; monitoring of engineering structures; geotechnical monitoring; criteria of failure state

1 INTRODUCTION

Monitoring of structures at operation is a direct requirement of the existing Russian legislation, which is especially relevant for technically difficult structures, including high-rise buildings. It is important not only for providing mechanical safety at operation of a structure but also as a tool of back analysis of actual strain-stress behavior of soil and structures as per corresponding to a calculation prediction implemented at design, that finally should facilitate development of construction theory and practice (Katzenbach R., 2005).

2 THE MAIN REQUIREMENTS TO THE SYSTEM OF MONITORING AT OPERATION

Monitoring systems should be based on the principles of interactive monitoring, i.e. obligatory include a possibility of comparison of monitoring data and a prediction, back analysis of monitoring results using computer models of an object, an opportunity to adjust the prediction or make necessary managerial decisions based on the conducted analysis.

The main features of an efficient monitoring system include (Shahkin K.G., 2018): (1) an ability to respond to possible hazardous processes; (2) adequate assessment of a danger degree, timely identification of dangerous processes to give experts an opportunity to analyze a situation and take necessary measures; (3) existence of a formalized managerial decision-making subsystem.

The efficient monitoring system should contain at least an intermediate state between the "normal" and "failure" states.

The "alarm" signal (like the yellow of the traffic lights) should timely emerge when dangerous trends occur and leave a sufficient time period for a required response – beginning from the analysis of causes of the signal emergence to taking measures on structure reinforcement (if necessary).

In order to create an efficient monitoring system, the criteria of a state of structures should not be set only based on limit states of these elements according to the standards, it is necessary to introduce the notion "a predicted value of a parameter". A significant deviation of measurement results from a predicted parameter is to be considered as an alarming signal.

Contrary to calculations at design, which are always conducted with a "safety factor", calculations for defining predicted parameters should reflect the most probable state of a calculated structure, if possible, computer models should completely reflect actual structural behavior.

The system of setting ultimate values of parameters is advisable to be generalized using relative distances to limit values for groups of physically interlinked parameters. At this approach, for a control subsystem a state of groups of any number of parameters is described by a minimum set of two numbers – a relative distance to the limit value and a value of the "alarm" signal.

3 THE THEORETICAL BASIS FOR GETTING CRITERIA OF “ALARM” AND “FAILURE” STATES

For a single measured parameter X the system of criteria should include a limit value (which in most of cases is defined according to the requirements of the existing standards) and a predicted value (or several predicted values as per different combinations of loads) of the parameter. A predicted value is calculated as the most probable value of the parameter in certain conditions (without tolerances towards “a safety factor”, with account of really applied loads etc.) As a result of a series of calculations for different combinations of loads we obtain a suite of possible results of the calculation $X_1 \dots X_n$. Then one should define the most probable average value $\bar{X}$ based on either general consideration (for example, a calculation without wind impacts), or on the arithmetic mean of the obtained calculation results.

Let a monitoring system observe some value of the parameter X . We indicate a difference between the predicted average value and observed value via $r = |X - \bar{X}|$. r_{lim} is a distance to the limit value, we sort out of two limits in the numerical axis the limit in the same direction, in which X deviates from $\bar{X}$. Mathematically it can be written down as:

$$r_{lim} = |X_{lim} - \bar{X}| \text{ при } (X - \bar{X})(X_{lim} - \bar{X}) > 0.$$

The system of control representation of the results is not expedient to be overloaded with difficult values in different measurement units. Therefore, in order to facilitate let introduce the notion of a relative distance to the limit value:

$$\varepsilon = \frac{r}{r_{lim}} \quad (1)$$

In this case the limit state corresponds to $\varepsilon = 1$. A location of the “alarming” boundary is obtained according to the results of a series of prediction calculations $X_1 \dots X_n$. For each of these calculations we compute a distance from the mean value $r_i = |X_i - \bar{X}|$ and a distance to the limit value in the corresponding direction $r_{lim i}$. Then the area of admissible values of the parameter can be limited by a relative value:

$$\varepsilon_a = \gamma \cdot \max \left(\frac{r_1}{r_{lim 1}} \dots \frac{r_n}{r_{lim n}} \right), \quad (2)$$

where γ – a factor, which takes into account prediction inaccuracy and it is accepted as, for example, 1.1...1.2.

For the system of interlinked data this approach can be generalized, in this case X represents not a single value, but a suite of values, instead of the limit values of parameters one should consider surfaces, which limit the areas of their admissible values.

Let consider building a system of restrictions for a set of parameters on the example of observation of a column cross-section. A set of strain gauges should be installed in a column cross-section to record not only longitudinal forces but also the moments. According to the log of these gauges least square method defines a longitudinal strain and angles of section rotation (with account of the hypothesis of flat sections, which is conventional for frame systems). Based on the obtained values of deformations, with account of the known material properties, forces are defined – a longitudinal force N and a moment in two directions My and Mz . The acting forces can be manifested as a point in the space N , My and Mz . One can depict with a point the predicted values of forces in the same space.

Then, according to the requirements of the standards, for example, as per non-linear strain model of reinforced concrete a limit surface is built in the space of coordinates N , My and Mz . The boundary of admissible values at monitoring is computed according to formula (2) around the area of predicted values of forces.

The described mathematic transformations result in

unification of operation with any groups of interlinked data. In any case, for the control system a group state is described by one number – a relative distance to the failure boundary ε . If this value exceeds 1, the “failure” signal sounds. An admissible value ε_a is computed for timely response to development of a process for each group of parameters in the monitoring system. If $\varepsilon > \varepsilon_a$ the system gives the “alarm” signal.

It is noteworthy that as the complex multidimensional system of limitations is actually reduced to one number ε_a , this value is not stationary. When gauge logs change, there is changing not only a value, but a direction of black and white vectors shown in fig. 1. Therefore, both values ε and ε_a should be recalculated.

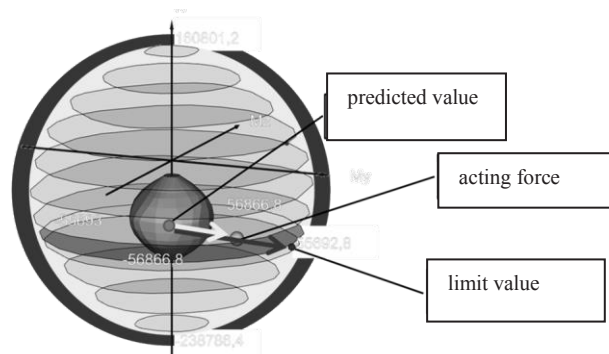


Figure 1. An example of the system of limitations for strain gauges in a column: the white vector shows the distance from the predicted values to the observed ones, the black vector shows the distance to the limit values (to the “failure” boundary); the relative distance to the limit values ε equals the ratio of the length of the white and black vectors; the boundary of admissible values (beyond which the “alarm” signal occurs) is shown in the center

4 POSSIBLE ACTIONS AT EMERGING THE “ALARM” AND “FAILURE” SIGNALS

In an efficient monitoring system for a building or a structure, which meets the requirements of mechanical safety, in practice the “failure” system should not emerge at all. The whole operation of the monitoring system is to be made after receiving the “alarm” signal without allowing the system to reach the limit state (the “failure” signal).

An approximate sequence of possible measures on the analysis of work of a structure and fixing possible negative trends after receiving the “alarm” signal can be represented as follows:

- informing people in charge of the corresponding monitoring service (at the building management service or at the construction site office).
- examining locations of gauges with “alarming” logs.
- back analysis of monitoring results using a calculation model for identification of causes of deviation from the predicted values.
- elimination of the “alarm” signal by means of taking the required measures.

Fig. 2 schematically depicts the algorithm of making managerial decision. The monitoring system must be interactive, otherwise it inevitably becomes inefficient.

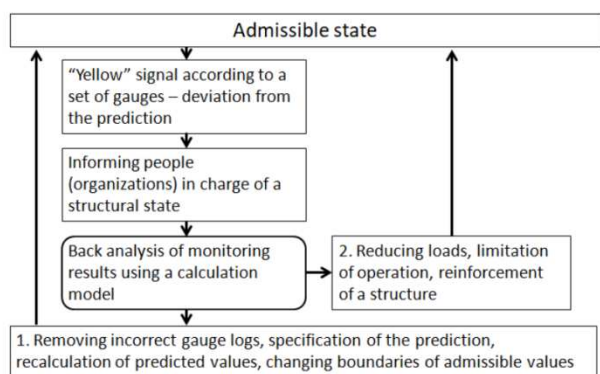


Figure 2. The facilitated algorithm of managerial decision making.

5 THE COMPONENTS OF INTERACTIVE MONITORING

In order to provide serviceability of the monitoring system it should include the following main components:

1. The system of gauges in structural elements and subsoil.
 2. The system of collecting and archiving the logs from gauges.
 3. The system of control observation of logs of groups of gauges, tracking emergence of "alarm" and "failure" signals and generating signals for informing people in charge.
 4. The system of managerial decision making.
 5. The interactive calculation schemes for assigning criteria of the "alarm" and "failure" with a possibility of recalculation and adjustment of predicted values during monitoring.
- Let consider determining the criteria of "alarm" and "failure" on the example of a high-rise building in St. Petersburg.

6 A BRIEF CHARACTERISTICS OF THE HIGH-RISE BUILDING

The construction of the tallest building in Europe – "Lakhta center" tower in St. Petersburg was completed in 2019. Considering a high degree of responsibility of the building, a part of information is closed but the authors think that open information is interesting for experts.

The structural section of the design was developed by the companies "Inforsproekt" and "Gorproekt" under the supervision of V.I.Travush and A.I.Shakhvorostov (Travush V.I., 2015, 2018), the design solution of the underground part was elaborated under the supervision of O.A.Shulyaev (Shulyaev O.A. , 2016). The main dominant of the multi-functional complex "Lakhta Center" reaches 462 m, it consists of 86 floors and 3 underground levels (fig. 3).

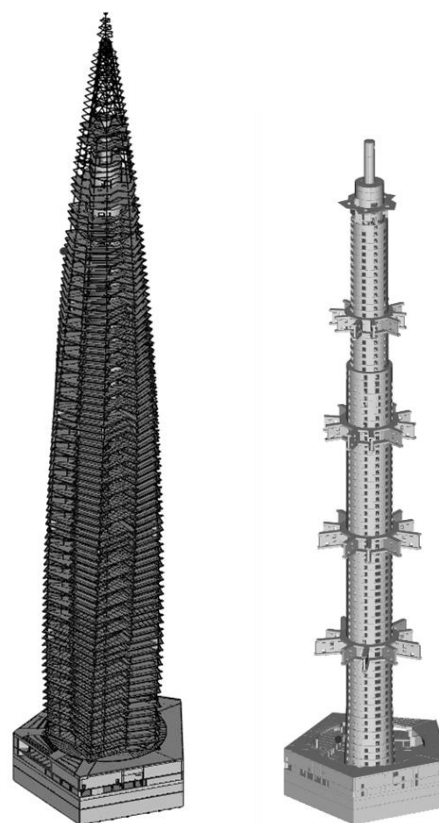


Figure 3. The general view of the calculation scheme and its fragment made of volumetric finite elements for organizing monitoring at operation

The high-rise building has a frame-and-stem structural scheme. Its rigidity and stability are provided with joint operation of the central RC core and 10 steel-reinforced concrete columns along the perimeter interconnected with outriggers, which are located in several levels. To decrease spans in the building there were implemented other 5 steel-reinforced concrete columns up to the level of the 47th floor.

The underground part of the high-rise building has the shape of an equilateral pentagon in plan, the side length is 57.5 m. It forms the box foundation, which consists of a lower 3.6-m-thick slab, located at the depth 17.65 m from the ground surface, an upper 2.0-m-thick slab, the central rigidity core of 28.5 m diameter and 10 vertical rigidity diagram walls. The box footing is supported by 264 piles of 2 m diameter. The piles are 65-m-long (84 m from the surface) within the contour of the high-rise building and 55-m-long (74 m from the surface) – beyond it.

The geotechnical profile of the high-rise building consists of quaternary deposits of different genesis and Upper Proterozoic Kotlin solid clays (Venda deposits; their roof lies at the depths of about 25 m from the ground surface), which serve the subsoil for pile foundations of the structure.

As regards headed aquifers within the area under consideration, Venda aquifer suite is of special importance, it is connected with interlayers of sandstones in the lower part of Venda deposits, with the head 90 m to the absolute elevation – minus 12 m BS. Moraine deposits underlain by dislocated Venda clays lie directly under the base of the piled raft.

7 THE CONTROLLED PARAMETERS OF THE MONITORING SYSTEM

The controlled parameters of the monitoring system of technical condition of bearing structures of the building include:

- movements of the upper point of the tower;
- a tilt of the foundation;
- a tilt of the building;
- natural frequencies and maximum amplitudes and fluctuation curves of the walls of the tower core;
- frequencies of mutual fluctuations of parts of the tower;
- strains in RC crossarms;
- strains in RC walls of the superstructure;
- strains in metal columns;
- strains in outrigger beams.

Movements of the upper point of the tower are defined automatically via processing results of measurements by a high-precise GPS-system.

A tilt of the foundation and structures of the building is controlled with a help of automatic inclinometers installed in the foundation and structures of the building.

Frequencies, amplitudes and fluctuation curves are defined via processing the results of measurements of 3D vibro-sensors -accelerometers.

Strains in the tower structures are controlled by strain gauges installed in slabs and walls of the box foundation, walls of the core and columns of the building. Strain gauges were installed to obtain data on a relative longitudinal deformation of the box foundation, a relative longitudinal deformation of composite columns, core walls and outrigger frames during construction of the building and at the stage of operation.

Reading gauge logs is implemented with a help of the automated system of data collection.

The sensors of geotechnical monitoring, namely strain gauges controlling deformations in piles, pore pressure gauges, pressure gauges along the base of the piled raft slabs are also connected with the monitoring system.

The monitoring system at operation was developed by the Institute “Georeconstruction” together with “Telros” company. Geotechnical monitoring at construction of the building was implemented by the Institute “Georeconstruction”.

8 THE NUMERICAL MODEL AND PREDICTED VALUES OF THE CONTROLLED PARAMETERS

A mathematical model of the building implemented in the software complex FEMmodels-IEECloud, which was developed by the Institute “Georeconstruction” and it allows making soil-structure interaction calculations taking into account non-linear properties of structural materials and soils (Ulitsky V.M., 2014), is the key element of the monitoring system, representing a kind of an analytical center. The layout of the geometrical position as well as assigning parameters of the tower structures in the calculation scheme were made via conversion of an analytical model of a scheme built in “Revit”, issued as initial data in the environment of “FEMmodels-IEECloud”. The scheme consists of volumetric, plate and rod finite elements. The mathematical model considers joint behavior of the building structures, piles and subsoil.

Respective standard piecewise linear bilinear and trilinear diagrams of deformation of steel and concrete were used as deformation curves. The results of the scheme calculation with account of non-linear strain model of RC structures include not only more adequate consideration of their rigidity but also a possibility of building sought-after strength surfaces, which define “the red boundary” (a surface of the “failure” signal).

Therefore, “the red boundary” for each point of strain gauge installation is defined by a strength surface in axes N , M_y and M_z , obtained according to the results of the numerical calculation of the analytical model of the building, for a certain section of an element of the bearing structure, where a gauge is installed. Based on this limit strength surface, a limit strain surfaces in each point of installing strain gauges and ultimate values for each strain gauge are defined.

The value of the “red level” of strain-stress behavior of the walls of the tower core is suffice to be limited by a standard value of limit strain due to their predominantly compressive operation.

As for the “yellow level” (surfaces of the “alarm” signal) it is more justified to take some area near the predicted (based on the calculation results) forces (in the case under consideration deviation from the area of the predicted values of more than 10% is accepted as the yellow border). Exceeding the “yellow level” means that a structure operates not in compliance with the prediction, that requires additional examinations and identification of causes of the deviation from the assumed strain-stress state.

It is noteworthy that building a calculation scheme implies multiple simplifications at assigning models of material behavior, boundary conditions, loads, which in line with various violations and drawbacks in the course of construction work implementation often lead to inconsistency of a calculation scheme and actual operation of a structure. Therefore, during in situ measurements at the site monitoring initial parameters of the calculation scheme of the building should be specified, after it is necessary to make recalculation of the changed scheme. As a result of several iterations the calculation scheme of operation of the structural system of the object under consideration will start to be consistent with its actual behavior. It is this iteration method which allowed bringing the mathematical model of the tower of “Lakhta Center” into compliance with the in situ measurement results.

The figures below give the results of building-structure calculation. According to the results of the calculation for each group of installed gauges (ranges), predicted values of changeable parameters have been defined. The predicted values are taken directly from the calculation scheme as per a coordinate of location of a group of gauges (fig. 4, 5).

Foundations of the tower structure have different tilts from the vertical axis due to applying horizontal forces (wind loads) at different height in different time moments.

According to the design assessments, deviation of the tower top from the vertical line due to applying wind loads is 301 mm from the average wind load and 161 mm from pulsing loads (the total displacement of the top of the building is 462 mm). According to the results of monitoring, measuring a wind speed, there has been identified a period of time when the wind speed approached to the maximum calculated one.

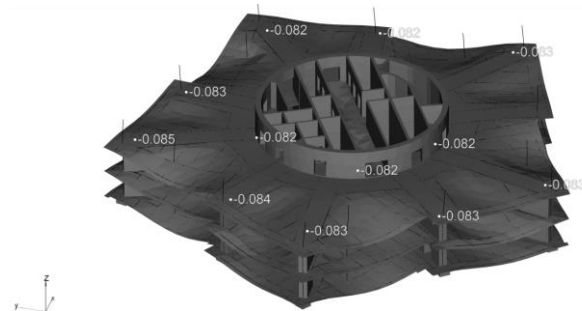


Figure 4. Vertical displacement of outrigger levels L17-L18, m, the scope of strains has been increased 100 times.

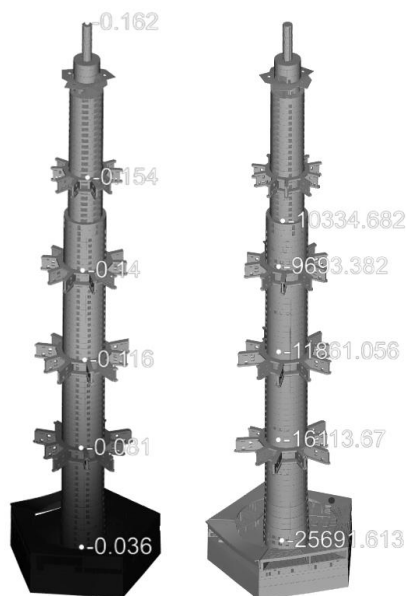


Figure 5. Vertical displacements of the core, m (left) and stresses in the core concrete, kPa (right)

According to the analysis, in this case the maximum value of deviation is in good agreement with the calculated one (fig. 6). According to the results of the analysis of the maximum angles of tilt there were accepted borders as per inclinometers which change depending on a level of a gauge location (fig. 7).

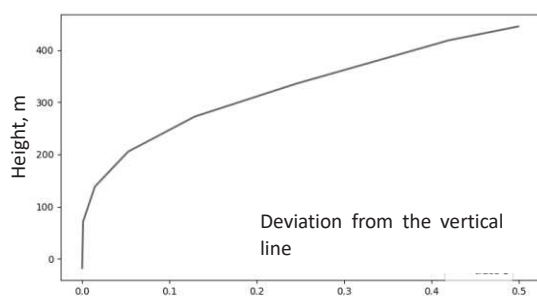


Figure 6. Evaluation of a shape of deviation from the vertical line of the building tower at wind loads, close to the maximum calculated ones (m)

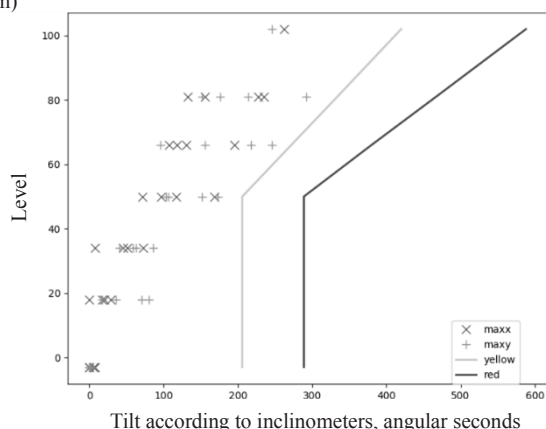


Figure 7. Maximum values of tilts in directions x and y according to the results of measurements at wind loads, which are close to the calculated ones, the location of the “yellow” and “red” boundaries depends on a number of a level as per the building height.

9 THE CRITERIA FOR DEFINING FREQUENCIES OF SELF-INDUCED OSCILLATIONS

A reduction of frequency of oscillations witnesses a decrease of the building rigidity or an increase of weights, it characterizes changes in the scheme of operation of bearing structures. An increase of frequency of oscillations of the building is highly unlikely, and it does not witness worsening of operation of structures.

According to the results of the calculations made by the authors of the design, the first self-induced frequency of oscillation was 0.12 Hz. The first actual measured frequency of vibrations after termination of the construction (March 2019) was 0.178 Hz. In the course of constructing floors, partitions in the tower, the increase of useful loads and changes of glaze ice loads the weight of the building changed along with reduction of the oscillation frequency. In June 2019 the first frequency ranged from 0.161 to 0.167 Hz, the second frequency – from 0.570 to 0.610 Hz; the third frequency – from 0.744 to 0.771 Hz.

It is possible to make approximate evaluation of the predicted frequency of oscillations depending on a share of loads from floors, partitions, useful and glaze ice loads applied to the building, without account of tolerances for quick creeping, and considering the dynamic RC modulus according to the following formula:

$$\nu = \nu' \cdot \sqrt{\frac{M_1}{M_1 - (M_2 \cdot (1 - k))}} \cdot \frac{1}{\phi_b} \cdot \frac{1}{\phi_0} = 0.160 \text{ Hz},$$

where $\nu' = 0.122 \text{ Hz}$ – the calculated frequency of oscillations, $\phi_b = 0.85$ – the factor of quick creeping, $\phi_0 = 0.83$ – the ratio between the initial and dynamic RC modulus (at very small deformations), $M_1 = 509392 \text{ t}$ – the designed weight of the building, $M_2 = 134206 \text{ t}$ – floors, partitions, useful and glaze ice loads, $k = 0.3$ – the approximate factor of account of loads of floors, partitions, useful and glaze ice loads.

Therefore, the changed frequencies are quite in a good agreement with the calculated ones.

As the limit values of oscillation frequencies are not regulated by the existing documents, the condition of reduction of the frequency of oscillations of the tower by 10% of the actual one is taken as the “yellow” boundary of accelerometer logs, the “red” boundary implies reduction of the recorded frequencies of oscillations by 30% of the measured value.

10 THE RESULTS OF THE GEOTECHNICAL MONITORING AT CONSTRUCTION

Creating the efficient monitoring system at operation is possible as a continuation of the efficient monitoring system at construction. Therefore, it is important to pay attention to the results of the geotechnical monitoring.

In the framework of the geotechnical monitoring before digging the pit a system of inclinometers was installed around the pit fence; pore pressure gauges and extensometers were installed in the subsoil; in the course of constructing the structures of the underground part of the building strain gauges were installed under the base of the piled raft.

The measurement results demonstrate that, opposite to the cylindrical shape of the underground part, the underground structure, which is pentagonal in plan, with the fence in the form of diaphragm wall with strut rings, does not allow reaching the same horizontal displacements of the pit fence, as expected. In more rigid corner areas displacements are almost twice as small as in the middle of each side of the pentagon (fig. 8). The maximum total displacement was observed at inclinometer I-11 at the depth 14 m, it was 46.4 mm, that did not exceed the maximum calculated values (76 mm). However, the correct shape of the underground structure in plan still allows using the effect of “a barrel with hoops inside”, which works for external pressure, and thereby carrying out works in an almost open pit with a minimum working area under cover of contour strut “hoops”.

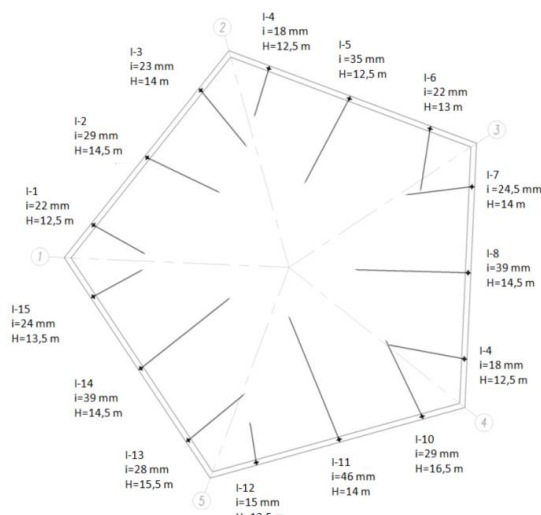


Figure 8. The scheme of layout of inclinometers and vectors of maximum total displacements of inclinometers with indication of adepth of the maximum displacement (from 28.08.2013 till 05.04.2015).

Observations of layer by layer settlements of the subsoil were made according to extensometers of the depth 97 m (see fig. 9). As the lower elevation of an extensometer is only 12 m lower than the toes of piles of the central part of development of the high-rise building and it can settle itself, location of the upper elevation was controlled using geodetic methods.

The settlements of the high-rise building developed along an increasing curve and came to an end by the moment of termination of the construction in 2018 - 32 mm in the area under the core. In the edge zones the settlements were twice as small. Almost the same settlements were recorded by geodetic methods connected with foundation benchmarks. Therefore, one can state that quite a limited layer of soil under pile toes has settlements. The compressed bulk below the pile toe is about 15. This conclusion is rather expected as the thickness of the bulk of solid Venda clays under the pile toe is 20 m, they are underlain by sandstones, which have higher rigidity and lower flexibility.

The effect of compression of piles and soil in the inter-pile area also made a contribution to the development of settlements (its share is almost 2/3 of the accumulated settlements). Based on the simplest calculations, elastic compression of the shaft of the pile of 2.0 m diameter and 65 m length under the load 3000 t is approximately 2 cm, that is in full agreement with the measured value of settlement at the expense of self-compression of the "conditional foundation".

The measured values of settlements correlate with the results of the conducted calculation of long-term settlements of the building on non-linear visco-plastic soil (with account of possible development of long-term creeping) (Shashkin A.G., 2014). According to the calculation results the maximum long-term settlement of the building will reach 8 cm at a relatively small differential settlement. About 1/3 of settlements occur during construction, i.e. about 3 cm, that is in a good agreement with the observation results.

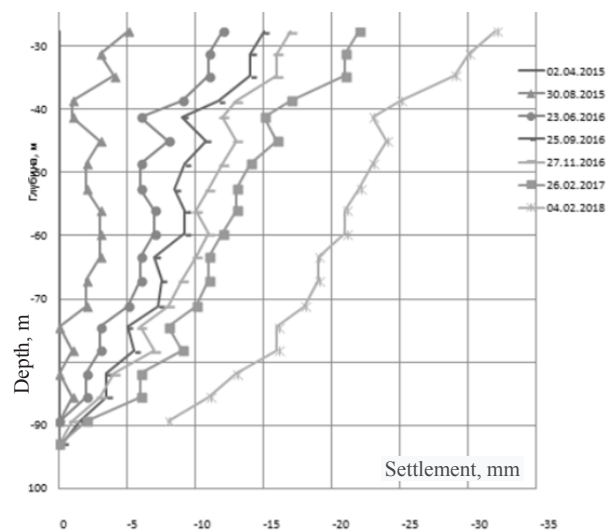


Figure 9. Strain curves of the soil bulk as per extensometer #1.

The gauges installed under the base of the piled raft demonstrated that contact pressures of soil are small (they do not exceed the pressure of the dead load of the slab of the piled raft). Therefore, the slab of the piled raft does not participate in operation of the pile foundation, it witnesses that the idea of a pile-raft foundation works only for a homogenous soil (Ulitsky V.M., 2014). If under the slab of the piled raft there is more flexible soil than under piles, the raft is excluded from operation. Identical logs, which almost correspond to the water head pressure with zero elevation at rest approximately at the level of the Gulf of Finland, were recorded at all borehole piezometers (fig. 10). In the period of construction of the underground part of the building gauge logs drop (from 07.08.2013 till 07.04.2014) precisely by a value of the water head released at the pit excavation (15-16 m). Then, in the course of the building construction there was observed a slow growth of the gauge readings.

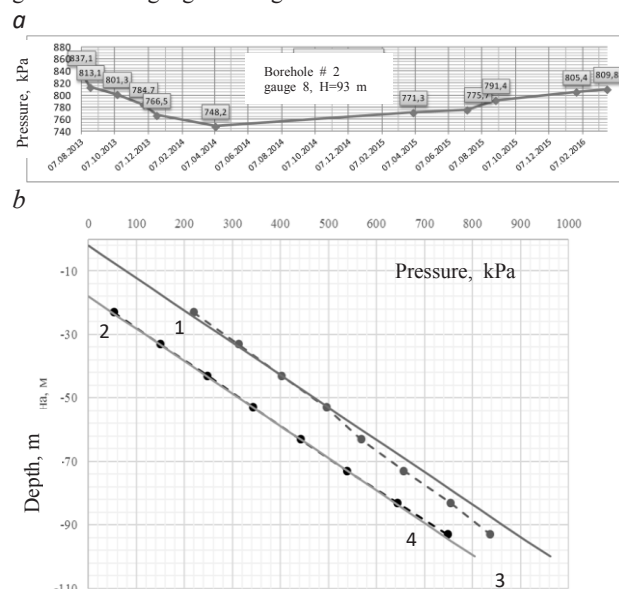


Figure 10. The logs of pore pressure gauges installed in borehole #2: a – measurement of pore pressure in time; b – graphs of pore pressure development as per the depth (1 – hydrostatic pressure before excavation and 2 – after excavation of a 18-m-deep pit; 3, 4 – logs of pore pressure gauges before and after excavation of the pit, respectively).

The gauge logs demonstrate permeability of Venda clays due to their intrinsic fracturing-block structure (Dashko R.E., 2000, 2011), earlier they used to be considered almost impermeable (it is noteworthy that chemical composition of water samples taken at the level of the slab of the piled raft showed its origin from Venda aquifer, i.e. from the depth 90 m).

As expected, no excessive pore pressure was observed: it cannot occur in solid clays where pore water is in cohesive state.

11 CONCLUSIONS

The practical implementation of the system of interactive monitoring at the site of “Lakhta Center” has demonstrated efficiency of using the soil-structure interaction calculation model at the analysis of the monitoring results and definition of the boundary values of the parameters for the “alarm” signal.

The calculation model for monitoring purposes, which reflects the most probable state of the object without tolerances and safety factor simplifications, considerably differs from calculation models applied at design.

Assigning parameters of the “alarm” signal at the boundary of the predicted parameters of the most probable state of the system ensures sensitivity of the system to negative changes, which appear possible to be identified at an early stage of their development, it provides timely analysis of a situation and taking adequate measures.

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