Lakhta Center: Automated Structural and Geotechnical Health Monitoring

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Abstract

«Lakhta Center» became a large scale public and office project in Primorskiy district of Saint-Petersburg, Russia. The Complex is comprised of supertall Tower, Multifunctional Building and the Arch with long span structures integrated with stylobate part. A number of innovation technologies and design solutions have been applied during the construction of the project.

According to the Building Codes and Regulations, to improve structural safety during the construction and maintenance periods, permanent structural health monitoring (SHM) program has been developed.

The main objective of SHM is to minimize structural failure risks due to uncertainties in soil and structural materials behavior.

General information about design and organizational arrangements for deploying automated structural and geotechnical health monitoring system of the «Lakhta Tower» is provided in the following paper. SHM architecture and topology, applied instrumentation, measurement methodology, software and monitoring parameters of the structure are described.

The article presents the results of the Tower structural behavior monitoring during the construction period. The correlation between the measured and the predicted Tower structure performance was analyzed and found in good agreement. A few anomalies were identified and investigated.

The focus is made to the informational value of the monitoring data for the increasing of soil, foundation and structure FE-modelling quality for construction accompaniment purposes.

Keywords Lakhta Center · structural health monitoring · geotechnical monitoring · SHM software · structural performance · monitoring program · high-rise building · SHM

1 Introduction

The mixed-use social and business Lakhta Center (LC) project, built on the shores of the Gulf of Finland within the boundaries of St. Petersburg, has become the tallest skyscraper in Europe. The height of the Tower, which is the main part of the Center, is 462 meters.

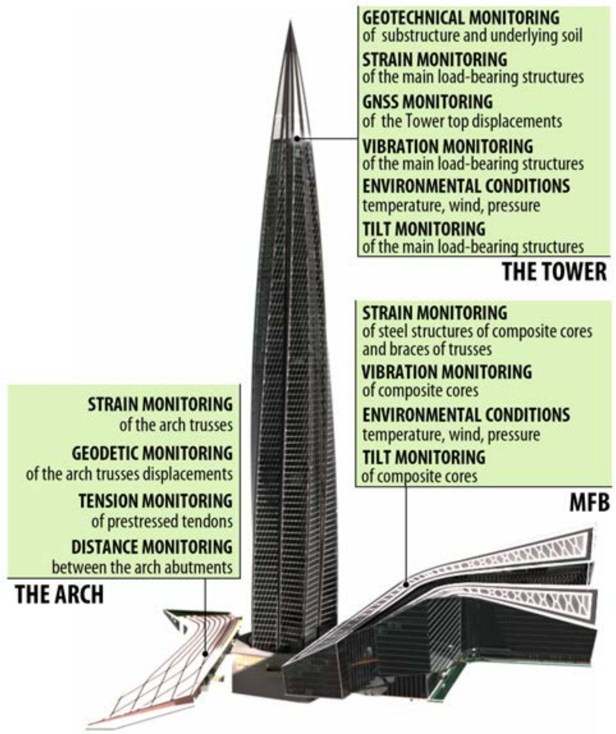

Fig. 1: Automated Structural Health Monitoring system of Lakhta Center

An automated structural health monitoring (SHM) system was designed to observe the unique buildings of the LC and joined geotechnical monitoring instrumentation, box foundation (BF) strain monitoring system, structural health monitoring equipment of the Tower high-rise part, the Multifunctional Building (MFB) and the Arch together.

The development of the monitoring program (Fig. 1) for the foundations and the above-ground part of the Tower was carried out by the experts of SAMSUNG C&T, GORPROJECT, SODIS LAB, INFORCEPROJECT and NIIOSP named after N.M. Gersevanov in 2015.

Since 2013, the periodic manual geotechnical monitoring of the basement has been established. In 2015, with the beginning of the foundation construction, scheduled monitoring of BF strain started, which continued until the launch in May 2016 of an automatic data acquisition system that allowed for the monitoring data automatic transfer to the office of the Contractor. Subsequently, as the Tower structure was erected, a new measuring equipment became connected to the automatic system. After the completion of the commissioning works, the monitoring system will be integrated with the building management system (BMS) and augmented with a number of measuring systems (Fig. 5) that will contribute to the safe operation of unique buildings of Lakha Center.

2 Brief information about the monitored object

The Lakhta Center Tower has three underground and 86 above-ground floors. The shape of the building is swirling tapered. The floor slabs are made in the form of five square “petals” interconnected by a circular central core. As the elevation increases, the square “petals” rotate around its axis counterclockwise, and their area decreases.

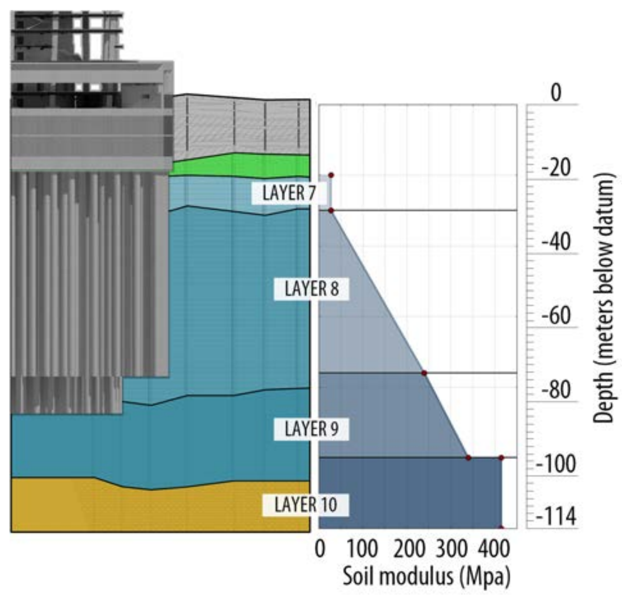

Fig. 2: Geotechnical profile

The Tower project required its developers to solve a large number of complex engineering problems. One of them was the design of foundation structures supported by clay soils with a low bearing capacity. It was decided to transfer the load from the high-rise building via piles to the layer of very firm Vendian clay (Layer 7. . . 9 in Fig. 2), discovered at a depth of about 20 meters with a soil modulus increasing with a depth from 28 MPa to 340 MPa (right side of Fig. 2).Vendian clays are featured with rheological properties that can increase the final settlement of the structure up to 25%.

In accordance with the design solutions, the most loaded part of the Tower is the central core, through which up to 70% of the load from the structure is transferred. To compensate for the uneven load on the basement, a decision was made to install 264 bored piles 2 m in diameter with lengths of 55 m and 65 m. Longer piles were placed under the central core in order to create additional rigidity [1] (Fig. 3).

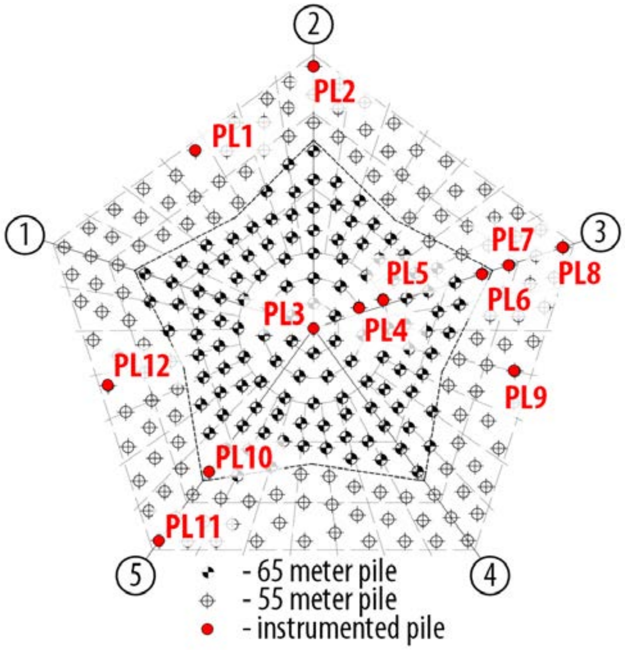

Fig. 3: Pile field layout

A large portion of the building weight falls on a small area with diameter of 26 meters, limited by a circular central core. The direct transfer of this load on the ground would create a pressure of up to 6.5 MPa. To increase the area of load transfer to the basement and preserve an acceptable difference of settlements, a 16.6 m high box-shaped (BF) foundation was designed, consisting of two plates: lower – 3.6 m thick ,upper – 2.0 m thick and 10 2.5 m thick diaphragm walls. A high-strength concrete of C50/60 class was used during the manufacture of BF.

The Tower has a frame-core structural system. The main load-bearing structural elements are the central reinforced concrete core and ten steel-reinforced concrete columns along the perimeter connected to the core by outrigger frames, which increase the rigidity of the building and its resistance to progressive collapse. Outriggers are designed in the form of reinforced concrete walls with steel trusses embedded. To manufacture vertical structures of the above-ground part a C70/85 class concrete was used.

Steel-reinforced concrete columns consist of a metal core and a reinforced concrete part, additionally reinforced with flexible steel bars.

The steel spire of the Tower is made in the form of a pentahedral pyramid located around the central core and resting on the 83rd floor (L83) at the elevation of +344,400 in the areas of composite columns. The height of the spire is approximately 118 m, the width of the face at the base of the spire pyramid is approximately 16 m.

The main structural analysis of the Tower was made in the LIRA-SAPR software package, the geotechnical part was modeled in the PLAXIS 3D software. Verification analysis was performed in the SOFiSTiK. Along with gravitational loads, wind loads were decisive during the building design, in order to study them a scale model was researched in a wind tunnel.

The construction of a skyscraper was started in October 2012 with the manufacture of test piles and retaining wall. During the construction of the above-ground part of the Tower, advanced construction technologies were actively used. Concreting of the central core proceeded ahead of the construction of perimeter structures by an average of 40–60 meters. Installation of the spire was completed in early 2018.

3 Monitoring of the above-ground part of the Tower

The above-ground part of the Tower is equipped with an automated strain monitoring system consisting of 1,257 vibrating wire strain gauges of various types. They are used to measure axial strain of composite columns, vertical and horizontal strain of the core walls, strain of outrigger trusses members and steel structures of the spire (Fig. 4). The system is commissioned gradually in stages (Fig. 13) during the construction of the supporting frame.

The layout of the instrumentation installed in the above-ground part of the building in addition to the strain monitoring system is shown in Fig. 5.

The control of the Tower inclination in two planes, which is also necessary for the normal operation of elevator equipment and facade systems, is carried out with the help of 26 Leica Nivel 220 high-precision inclinometers installed on outrigger levels and spire structures.

Analysis of the dynamic response of the building (determining natural modes and frequencies, modal damping ratios) caused by wind load, operation of technological equipment and other factors is performed using 17 low-frequency three- and two-component force-balanced accelerometers Geosig AC-

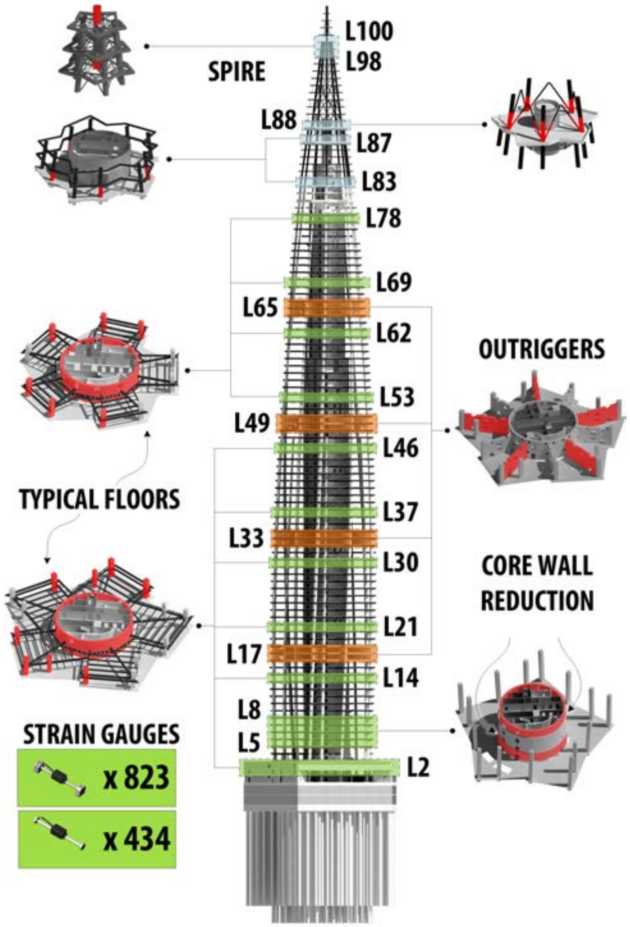

Fig. 4: Strain monitoring system of the above-ground part of the Tower

72 (73) mounted across the entire height of the Tower. The system provides information on the building oscillations, which is necessary for an integrated assessment of changes in the structural condition over time and the calibration of the FE-design model parameters.

Real-time and post-processing monitoring of the upper point displacements is performed using the satellite geodetic monitoring system. The antenna of the Novatel ProPak6 GNSS receiver at the observed point is attached to the top of the spire. The reference point, relative to which the measurement of displacements is made, is set on a deep monument, mounted at a distance of about 500 meters from the Tower. Communication with receivers is made via fiber-optic line. Satellite monitoring of the top point displacements in combination with the results of inclination and vibration monitoring provides comprehensive information on the deformation of the building axis.

The installation of two weather stations (the second is mounted on the roof of the MFB and measures the climatic parameters in the surface zone) will allow to perform a correlation analysis of stress-strain parameters and separate the changes in parameters associated with the degradation of building structures from seasonal and climatic changes.

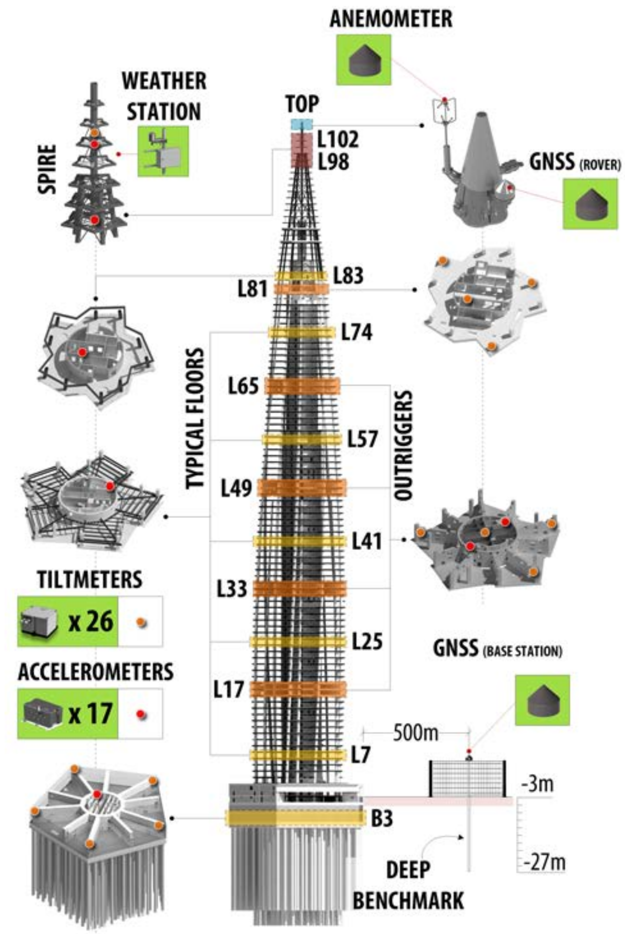

Fig. 5: Monitoring system of the above-ground part of the Tower

The commissioning of the listed measuring systems is expected at the final stages of construction.

4 Box foundation strain monitoring system

The BF strain monitoring system (Fig. 7) is the most ambitious and complex subsystem within the SHM of the Tower - was designed to analyze changes in stress-strain state of the foundation, which in 2015 got into the Guinness Book of Records as the most massive reinforced concrete structure (19.6 thousand cubic meters), made using continuous casting method.

To control the stress-strain state, a total of 1,210 vibrating wire strain gauges were combined into 196 gauge sections on the bars of the main reinforcement of the bottom plate, wall diaphragms, and the top plate (Fig. 6).

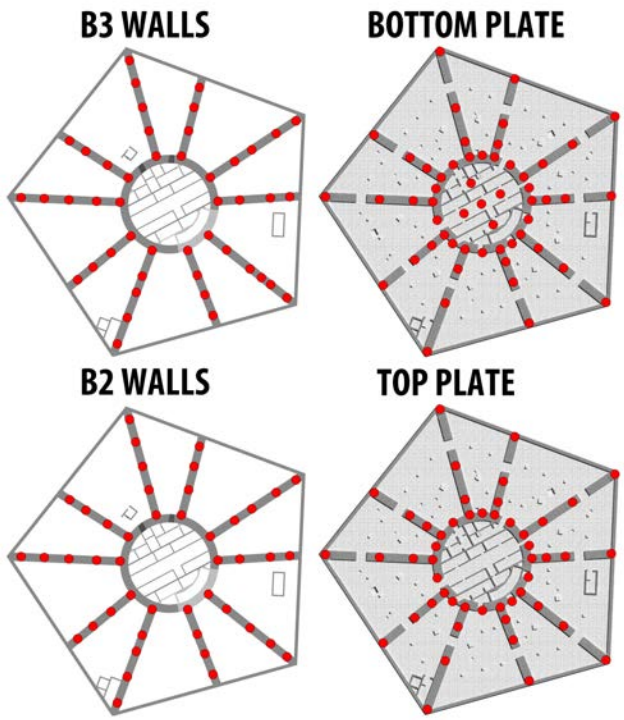

Fig. 6: Strain gauges alignment in box foundation structures (gauge sections shown with red dots)

Each gauge section, depending on the type of structure and the nature of its deformation, includes two or three measuring points. The measuring points are equipped with a pair of sensors oriented in perpendicular directions along the main reinforcement bars. Such configuration of the system made it possible to determine stress distribution across the thickness of the plates and walls of the BF with accuracy sufficient for comparison with the FE-analysis.

5 Geotechnical monitoring system

The geotechnical monitoring program was developed by the employees of NPPP SPETSGEOPROEKT LLC with the scientific and technical support of the NIIOSP named after N.M. Gersevanov in 2013. The program provided for monitoring of the retaining wall, bearing structures of the underground part of the Tower, the MFB and the stylobate part, as well as the surrounding soid media, including monitoring of the groundwater regime. Installation of the instrumentation for geotechnical monitoring of the Tower was carried out in 2013–2014 (Fig. 7).

To control the actual distribution of the load on the pile field, 12 of the 264 piles were equipped with embedded vibrating wire strain gauges Geokon 4200, installed in seven levels across the pile length. To monitor the load distribution between the BF bottom plate and the piles, ten pressure sensors Sisgeo L143 were placed under the foundation concrete mat.

The obtained information allows to adjust the parameters of soil and foundation structural models for accurate prediction of the structure settlement. The results of the forecast make it possible to timely arrange compensating actions to prevent the consequences of uneven foundation settlements.

As mentioned earlier, long-term consolidation processes in the underlying soils will lead to the development of settlements during the structure operation. To assess the dynamics of the settlements, the basement was equipped with five boreholes for monitoring pore water pressure with eight vibrating wire piezometers Sisgeo PK45M in each. Piezometers were fully grouted [2] and installed with an average step of 10 m in depth.

6 Automated SHM system design solutions

The development of the automated system architecture, detailed design documentation, and equipment selection were carried out by SODIS LAB staff. The long service life of the system (at least ten years after construction) imposes many restrictions on the choice of sensors. The standard for long-term monitoring of slowly varying strains in building structures is a vibrating wire (VW) technology. VW gauges remain operable for decades [3, 4] and demonstrate excellent zero stability [5].

The advantage of VW technology over more traditional resistive or semiconductor sensors is the type of signal itself – the natural vibration frequency of the tensioned wire (not voltage, current, or resistance). This signal is easily transmitted without distortion over long distances, it is not susceptible to corrosion or moisture on the conductors of cable lines, it is only slightly sensitive to the presence of electromagnetic interference and does not depend on the length of the cables.

The installation of strain gauges in the BF structure and the core wall was complicated by very dense reinforcement (Fig. 8).

The sensors were mounted on the main reinforcement by arc-welding the end blocks to the reinforcing bars (Fig. 9).

Cable laying through horizontal structures was performed in PVC pipes, which were built up as the reinforcement cage was installed (Fig. 10).

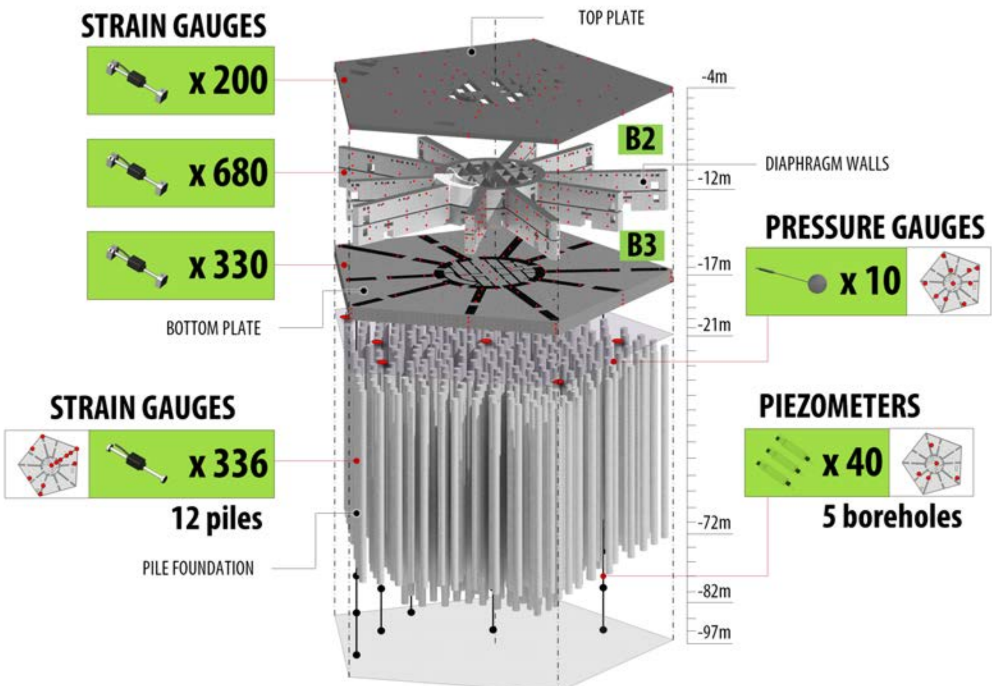

Fig. 7: Automated geotechnical monitoring system layout

Fig. 8: Embedded PVC details for gauge signal cables

Fig. 9: VW strain gauges installed on the reinforcement of BF bottom plate

The outlet of cables from the concrete body was carried out exclusively on vertical structures. At the time of concreting, the cables were stored inside specially prepared embedded details made of PVC products. After dismantling the formwork, embedded parts were opened, and the cables were brought out (Fig. 11), where they were connected to the data acquisition system.

One of the main tasks for arranging an automated strain monitoring system was the need to launch it in the early stages of construction, which imposed a number of significant restrictions on the entire system architecture. It was decided to deploy a distributed data acquisition system as opposed to centralized. Equipment for automatic recording, located on the technical levels of the Tower (B3, B2, L18, L34, L50, L66, L81) as the bearing structures were built (Fig. 12), formed seven autonomous subsystems to which the downstream sensors were connected.

Fig. 10: Strain gauge cables arrangements (outlet from the BF plate)

Fig. 11: Embedded detail in BF diaphragm wall after formwork removal

Chronologically, the deployment of the strain monitoring system was divided into six stages (Fig. 13): At the first stage, the BF strain monitoring system and the geotechnical monitoring system instrumentation were commissioned, then standard and outrigger levels were connected in series.

All data acquisition nodes were connected to a temporary battery backed power supply system in case of power outages. Communication between nodes was provided via temporary wired data network.

Despite the large distances between the nodes, the use of fiber-optic communication lines was rejected due to the complexity of their repair in construction conditions, focusing on the use of “copper” DSL and Ethernet technologies.

Fig. 12: BF data acquisition enclosure

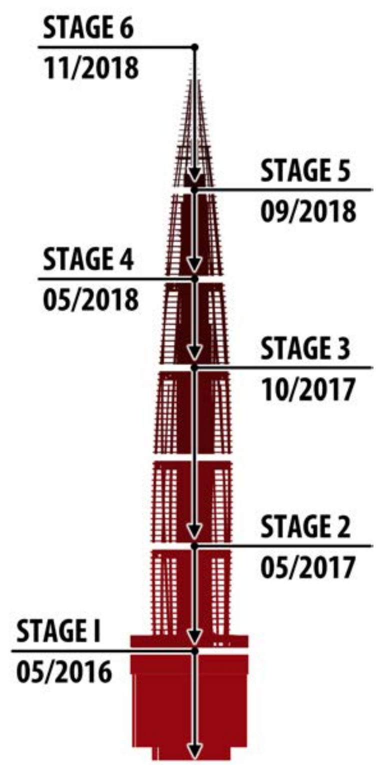

Fig. 13: SHM commissioning stages

To reduce the likelihood of information loss, monitoring data is recorded with programmable data loggers with internal memory. The principal system architecture is shown in Figure 14.

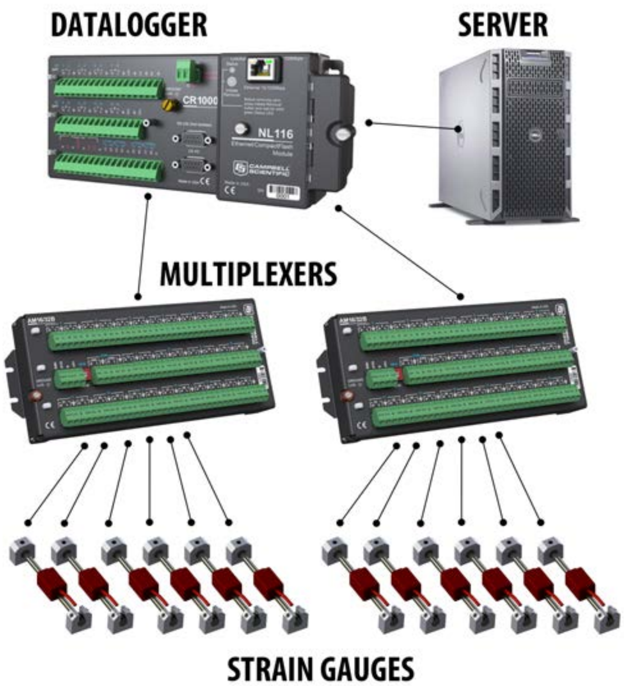

Fig. 14: Principal layout of the automated data acquisition system

All data acquisition nodes were built using Campbell Scientific equipment (Fig. 15).

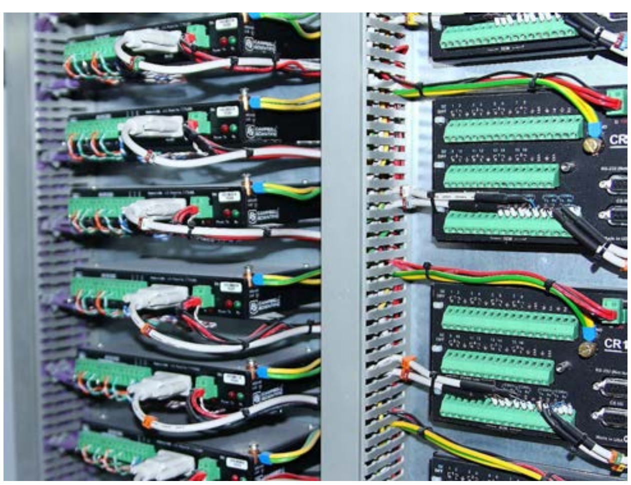

Fig. 15: Typical contents of data acquisition enclosure

For the monitoring of BF structures, two data acquisition nodes were arranged at the levels B3 and B2. Multiplexing hardware was installed directly above the embedded details at every outlet point of the signal cables (Fig. 16).

Fig. 16: Multiplexer enclosure installed above the embedded detail with gauge cables

This solution has significantly reduced the length of cable lines, reduced the likelihood of cable damage during construction, simplified and cheapened repairs, allowed promptly change of cable routing, and eliminated the need to splice strain gauge cables.

Cables were laid in trays (Fig. 17), which reduced the damage to lines during the construction to isolated cases.

Fig. 17: The supporting system of gauge cables

On outrigger levels, multiplexers were installed in the immediate neighbourhood to dataloggers – inside steel enclosures. This led to the need to install additional splicing points to connect trunk cables to strain gauges, but reduced the total number of necessary equipment.

The versatility of the VW interface made it easy to integrate equipment for geotechnical tasks into the strain monitoring system.

Delivery of monitoring data to the office of the Contractor (Fig. 18, 19) was performed via wireless Wi-Fi bridge.

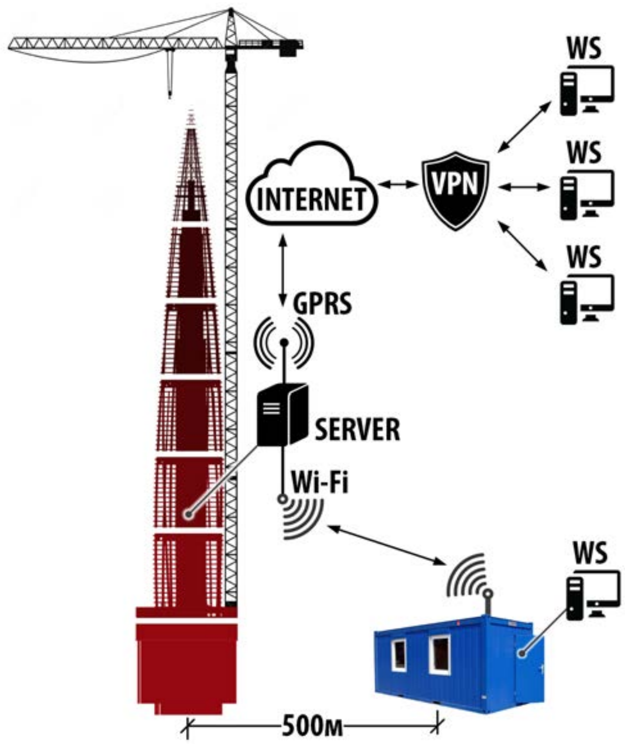

Fig. 18: Monitoring data transmission layout

The installation of a GPRS modem provided a remote connection to the monitoring network via the Internet using a secured VPN tunnel (see Fig. 18). This solution allowed us to promptly provide information, carry out maintenance and analyze monitoring results online.

Fig. 19: Temprorary SHM operator's laptop in the office of the Contractor

7 Monitoring system software

The implementation of a large-scale monitoring system, including more than 2,800 sensors, required the deployment and support of a modern IT infrastructure. As a software platform for working with monitoring data, the SODIS Building M4 environment was used (Fig. 20), it includes a large number of problem-oriented services and tools for solving the tasks of long-term monitoring of building structures.

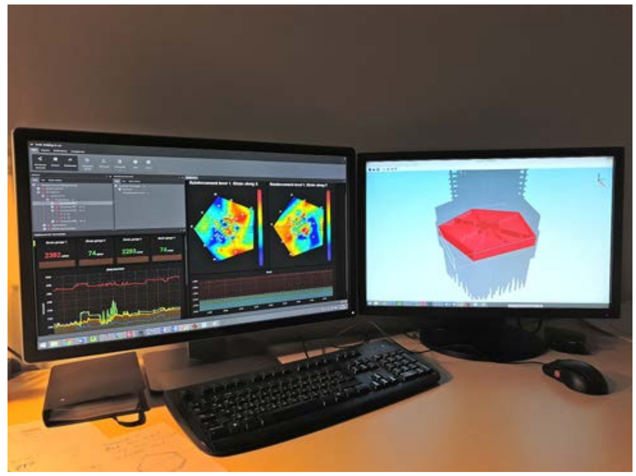

Fig. 20: SODIS Building M interface

Fig. 21: SODIS Building M interface

The capabilities of the SODIS Building M software platform (Fig. 21) made it possible to provide automated accounting for individual sensor characteristics (gauge factors, initial measurement data), meta information (coordinates of installation points, sensor orientation, structure type, deformation characteristics of materials, geometric characteristics of sections, affiliation to the gauge sections, etc.) required for data analysis, which allowed by using the automatic methods move from direct (strain) measurement results to stresses and internal forces in structural elements, for which it is possible to make a comparison with calculated values according to the results of mathematical modeling. The application programming interface (API) has allowed to automate the exchange of data with BIM and FE models of the Tower.

During the commissioning work, software tools were developed for the automatic generation of controller's firmware and processing of installation protocols. By the completion of the adjustment work on the first start-up phase, 95% of the working procedures for setting up the system were automated, which made it possible to eliminate the inevitable mistakes of the performers caused by the execution of routine operations, as well as to increase the speed of adjustment of subsequent blocks of the system.

8 Amount of obtained monitoring data

As of the end of August 2018, the Lakhta Center Tower monitoring system database contains almost 74 million measurements from 2346 strain sensors, 9 pressure sensors and 39 piezometers. All the collected data was carefully filtered, cleared of random outliers and non-physical values, and supplemented by the results of manual monitoring (Fig. 22).

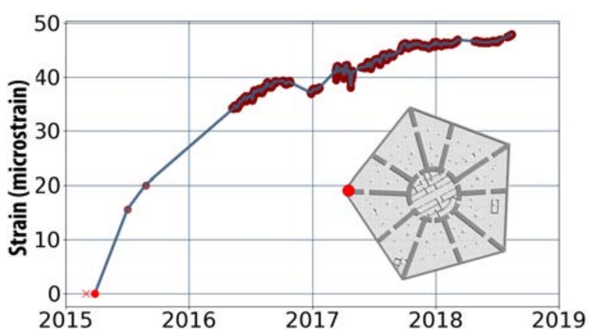

Fig. 22: Strain data from X10 sensor (installed into BF bottom plate)

The obtained data formed the basis for calculations of the stress-strain behavior components of the Tower structural elements (Fig. 23, 24).

Data obtained from geotechnical instrumentation (pressure sensors and piezometers) is used to clarify the stiffness and consolidation characteristics of the basement analysis model (Fig. 25).

9 Analysis of monitoring results

Comparison of strain monitoring data with the results of finite element (FE) modeling of Tower structures is the basic task, the solution of which allows to come closer to determining the actual nature of the

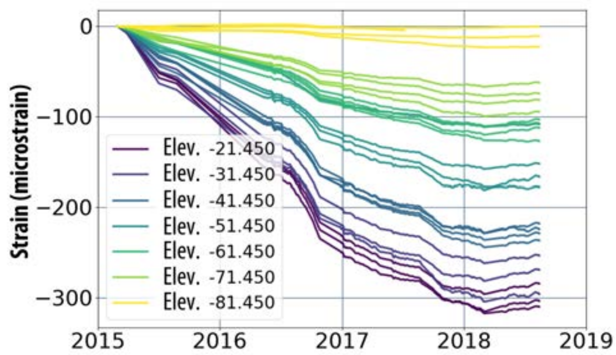

Fig. 23: PL4 pile strain data

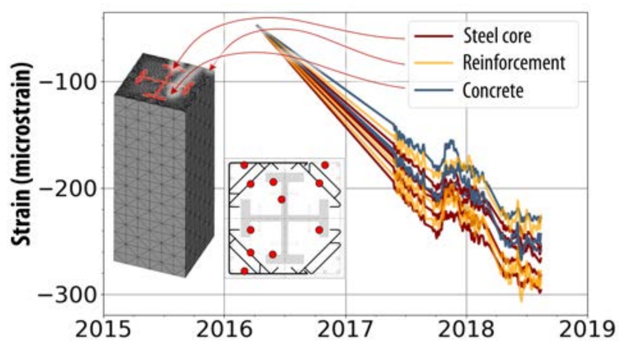

Fig. 24: SRC-2 column strain data on level L14

structural behavior and identify weak points in the design model, promptly respond to dangerous changes in stress-strain state and prevent an emergency from developing. For the analysis of results, monitoring data were taken as of July 1, 2018. According to our calculations, as of the date in question, the load-bearing structures of the building experienced 100% of dead loads, 100% of live loads on technical levels, 100% of loads from facade structures, and 50% of loads from floors, non-bearing walls and communications.

As one of the parameters for which the comparison was made, the vertical forces in the composite columns and the walls of the core in the levels of L2, L5, L8 and L14 were used.

The structural analysis of the building was carried out in three options:

1. on the effect of live loads on technical levels, dead load of structures and facades, 50% of the weight of floors, non-bearing walls and communications, on an elastic basement;

2. on the same loads as in the previous version, but taking into account the sequence of construction;

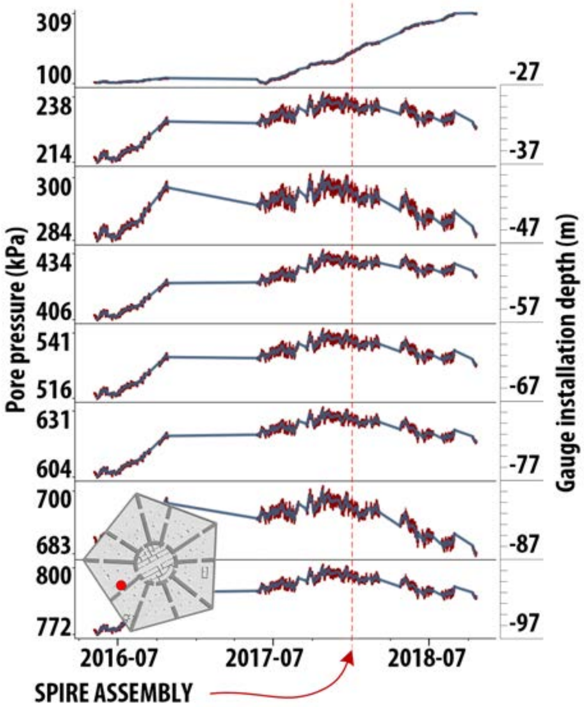

Fig. 25: Pore pressure data. Borehole PP5

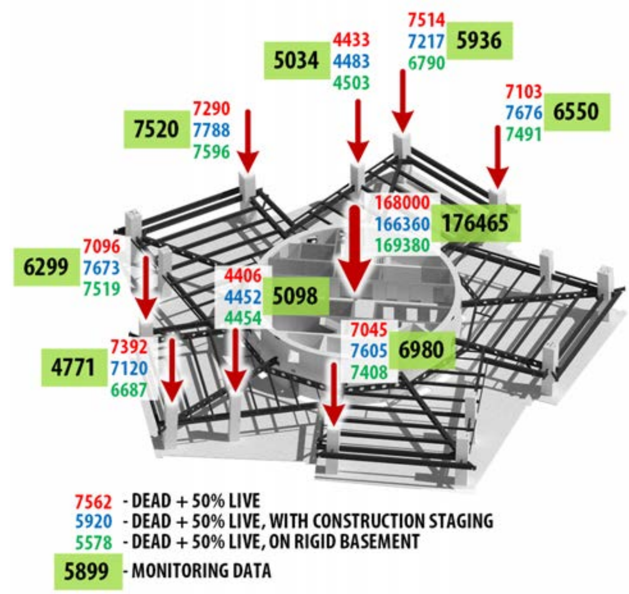

Fig. 26: Level L2. Predicted axial forces (tonne-force). Comparison with monitoring results.

3. on the same loads, taking into account the sequence of construction and on an absolutely rigid basement.

The analysis results for the level L2 are shown in Fig. 26. The total predicted load on the observed structures in the considered level differs from the measured value by less than 2%. The discrepancy between the predicted forces taken by individual composite columns and the core with the monitoring data is in the range of $3 - 33\%$.

A good agreement of the monitoring results was achieved in the level L5, where the predicted load on core differs from that measured value by 2.5%. Unfortunately, the unsatisfactory results on the load in the core were obtained at the level L8 (46% discrepancy, see the possible causes below).

At the level L14, the monitoring data indicate a significant difference in the load distribution between the columns and the core compared to the calculation results. The difference in the total load on the structural frame is less than 7%; however, according to the monitoring data, 20% more vertical force is applied to the core than the value obtained via calculations. Presumably, this can be explained by the special aspects of outrigger trusses work under load, which is confirmed by the difference in the observed forces only in the columns located under the outriggers. Processing the entire amount of data on the strain of the frame above the level under consideration will allow for a more accurate determination of the values of the parameters under study.

A serious obstacle to the calculation of stresses and forces in reinforced concrete structures according to the strain sensors data is the need to take into account the creep of the material, which is considered when analyzing data by reducing the concrete stress-strain modulus.

For preliminary assessment of the effect of creep on measurement results, it is possible to use the ratio between the bearing capacities of concrete and reinforcement. More accurate data can be obtained from the results of field tests.

Accounting for the creep effect on the deformation of steel-reinforced concrete columns was carried out on the basis of test results [6], which showed that the steel core should, on average, take about 60% of the total load.

Analysis of the BF bottom plate monitoring results showed that the plate deformation at the level of the two lower reinforcement meshes on a significant area of the plate is in the range of 300–500 microstrain, which exceeds the maximum permissible tensile deformation for concrete under continuous load according to [7] (270–360 microstrain). Therefore, to calculate the forces for the bottom foundation slab, a reduced concrete stress-strain modulus was used, corresponding to the one used in the FE model.

Design solutions for the installation of strain gauges in composite columns can be considered successful — qualitative data were obtained that correlated well with each other and made it possible to calculate the integral compressive forces. It should be noted that in the columns of the most heavily loaded observed level L2 deformation of the reinforcement averages 92%, and deformation of concrete - 96% of the deformation of the core, which confirms the admissibility of the hypothesis of their compatibility. Steel cores of the observed columns in this level take 42–60% of the total vertical load.

For columns of level L14, the ratio between the deformations looks different: 76% of the deformation of the core are deformations of reinforcement, 73% are deformations of concrete. Conclusions about the reasons for the increased deformability of cores, which in this level take 45–73% of the total load, will be made after processing the monitoring data of the columns at all observed levels.

The result of monitoring the core walls strain was less obvious. On some of the observed levels (in particular, at the level L8), the choice of only five measuring points was sufficient to determine the local stresses in the material, but not allowing an accurate calculation of the load taken by the core. Analysis of the calculated stress distribution in the walls of the core on such floors showed the presence of a large number of concentrations that make the stress field very heterogeneous. Therefore, averaging over five points can lead to a significant error in the calculation of the force taken by the core.

As a result of the pile field strain monitoring, we obtained the strain diagrams of the piles during the construction of the Tower (Fig. 27), on the basis of which the load on the head of each observed pile was calculated (Fig. 28, for the layout of the piles in the plan see Fig. 3).

The analysis of obtained data showed that the piles located in the central part of the foundation take the greatest load (up to 3460 ton-force). The minimum load is observed at the corner piles (924 ton-force). The ratio of forces in the central pile to the forces in the border piles is in the range of 2.9–3.4, which does not correspond to the general ideas about the behavior of piles in the pile field. Such results are probably due to the presence of a more rigid basement. This issue requires an in-depth analysis of the geological surveys, pile testing, pore pressure measurements and stresses under the foundation, which will be discussed in a separate article.

According to the periodically performed geodetic measurements, the average settlement of the Tower foundation is less than calculated, which may be due to the higher rigidity of the basement noted above or the incomplete soil consolidation process (Fig. 29).

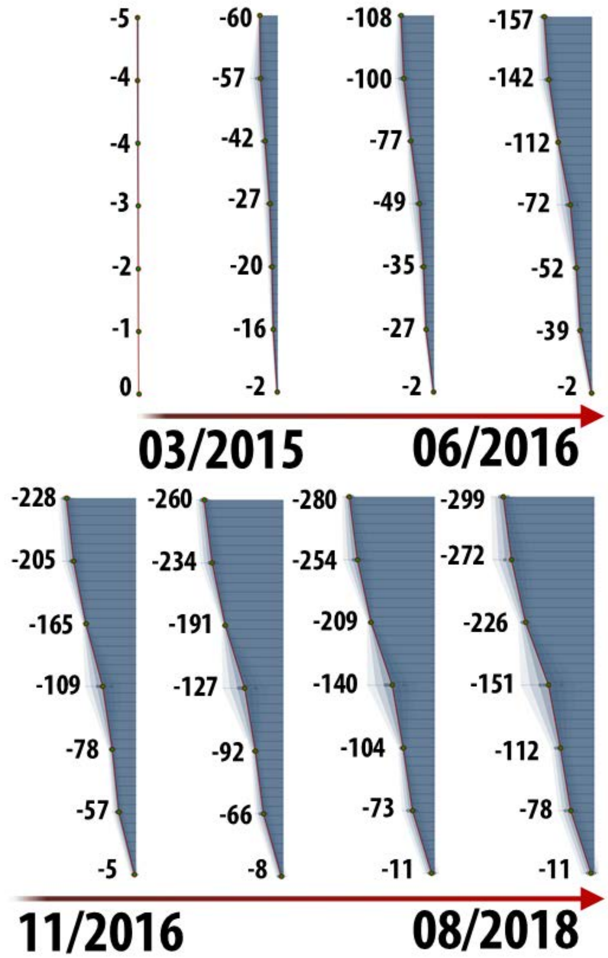

Fig. 27: Average strain (microstrain) diagram of pile PL4

According to the results of strain measurement in BF plates, stresses and forces in the three most loaded reinforcement meshes of the bottom plate and two meshes of the top plate were determined, stresses in the concrete body were also calculated. By integrating the normal stress distribution diagram over the cross section height, the bending and membrane forces acting in the plates were calculated (Fig. 32) and compared with the results of the finite element modeling.

The monitoring results for the BF bottom plate, presented in Figure 30, showed that the predicted basement stiffness differs from the real one (this is indirectly confirmed by the distribution of load on the piles, shown in Figure 28), which leads to differences in the nature of the force distribution over the plate area. At the same time, the amplitude values of the forces differ slightly from the predicted ones.

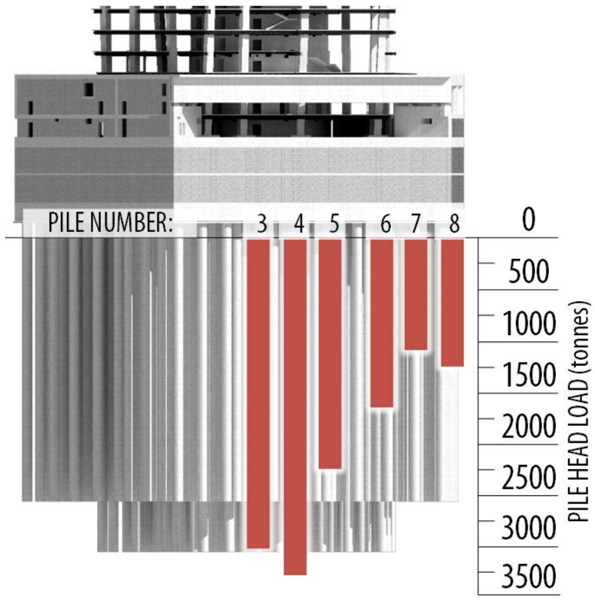

Fig. 28: Monitoring results. Pile heads force distribution on 01/08/2018 (tonne-force)

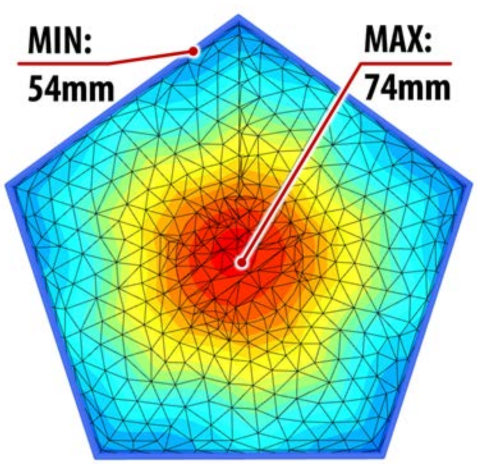

Fig. 29: Predicted tower foundation settlement on 01/08/2018

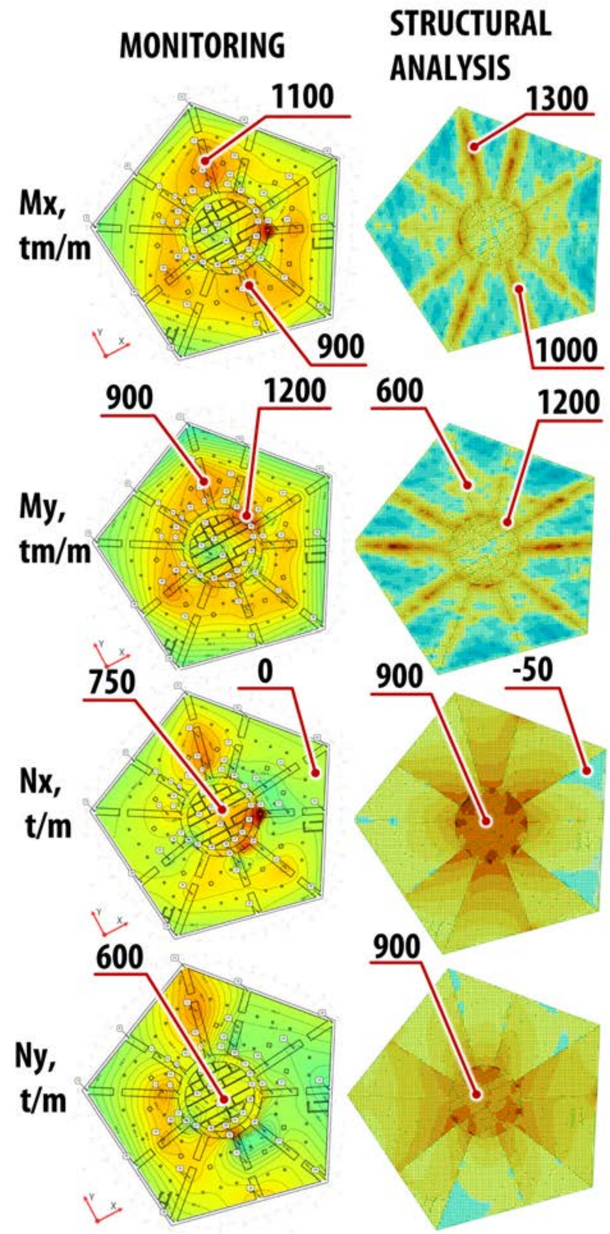

Fig. 30: Isofields of BF bottom plate internal forces (bending moments - M_x, M_y and axial forces - N_x, N_y). Comparison of monitoring data with predicted (FE structural analysis) values.

Design solutions for the BF bottom plate strain monitoring allowed us to determine the real stresses in the main reinforcement, in particular, tensile stresses in the most loaded lower reinforcement mesh do not exceed 80 MPa (Fig. 31), except for a small number of local concentrations.

In general, the monitoring system covers the whole complex of main bearing structures, providing the possibility of monitoring the actual values of stress-strain parameters during construction and future operation. Disagreements of the calculation results with the monitoring data in some cases illustrate the need to make adjustments to the analysis model based on the results of processing the specified data.

10 Conclusion

The automated structural health monitoring system of the Lakhta Center load-bearing structures is an example of the successful implementation of such

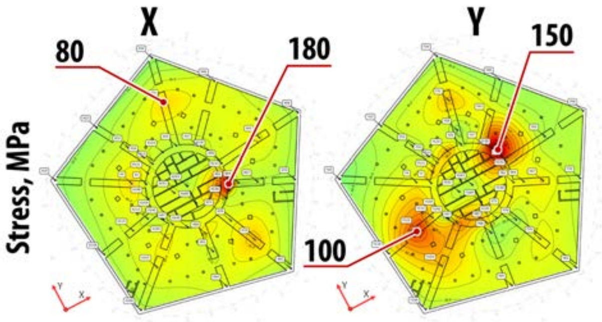

Fig. 31: Monitoring results. Isofields of stresses in bottom reinforcement of bottom BF plate

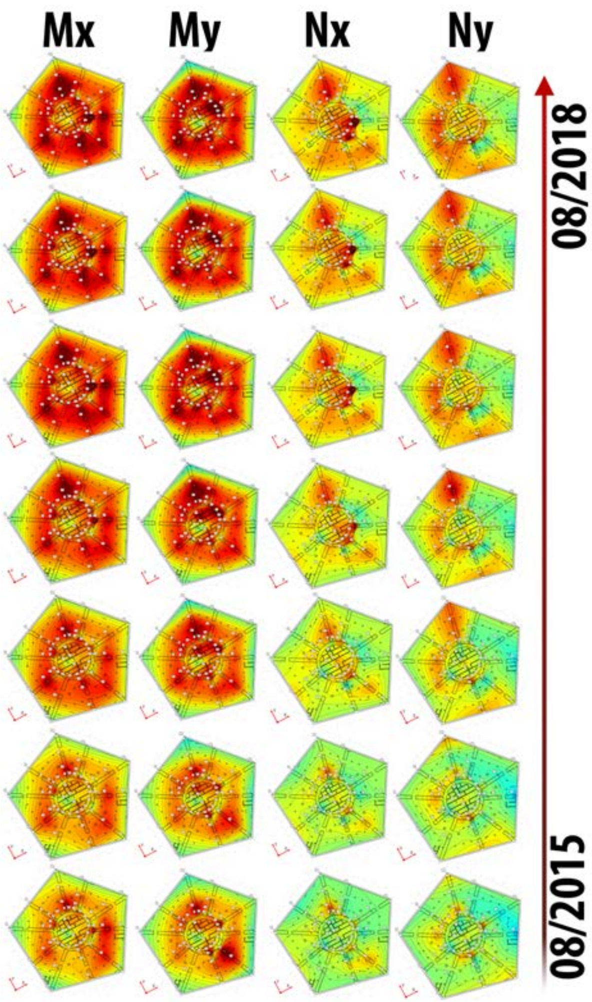

Fig. 32: Monitoring results. Internal forces (bending moments - M_x, M_y and axial forces - N_x, N_y) in BF bottom plate vs. time of construction

systems from an engineering point of view, which made it possible to solve a number of tasks in conducting scientific and technical support for construction of unique object: clarify the loads and deformation characteristics of materials, determine the actual stiffness of the basement, calculate the applied internal forces in the structural elements and thereby confirm the reliability and safety of the erected structure.

The processing of data on the deformation of the pile field, box foundation, composite columns and the central core revealed a number of unexpected effects: unloading columns by outrigger trusses, higher load on central piles in comparison to the border ones, etc.

Comparison of the strain monitoring results with calculated values showed a good agreement between the integral characteristics of the stress-strain state of most of the structures. But on closer examination significant differences between the expected and observed stresses and forces were found for some structures, indicating a lack of accuracy in modeling the behavior of these elements under load. It is difficult to imagine that such phenomena can be detected using any other source of information besides the monitoring system. Thus, for the qualitative solution of the design problems, the analysis model in the construction process should be constantly refined and adapted, and monitoring should be an obligatory measure in the adaptation process.

At the same time, the monitoring system should not be designed nominally, as is often the case in the practice of construction in Russia. The system design should strictly rely on the provisions of the monitoring program, in which the controlled structural elements, their parameters and the methodology of using the obtained values of the monitored parameters to calibrate the design models of the basement, foundations and above-ground parts of the structure must be defined.

It is advisable to carry out more intensive work on the development of a methodology for calibrating FE models of structures based on instrumented data provided by automated monitoring systems.

The implemented system is the most comprehensive monitoring system in civil engineering in Russia, comparable in its scale with the world's leading analogues [8]. At the same time, according to our estimates, the total cost of the system did not exceed 0.25% of the total cost of the Lakhta Center construction. Automated SHM system, some parts of which have been operated for more than four years, has shown excellent sustainability and maintainability in difficult and harsh construction environment.

During the construction of the Tower a huge array of data was accumulated. Some of this data has yet to be subjected to in-depth analysis by competent experts. The most interesting problems of interpreting the results of monitoring and using them to solve actual structural design problems will be covered in separate articles.

The use of automated monitoring systems will significantly increase the reliability of unique buildings and increase the safety of long-term operation.

The authors are grateful to all organizations participating in the project for their help in implementing such a large-scale automated monitoring system.

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