Збірник наукових праць. Галузеве машинобудування, будівництво
Academic journal. Industrial Machine Building, Civil Engineering
http://journals.nupp.edu.ua/znp
https://doi.org/10.26906/znp.2020.54.2271

UDC 624.131: 624.154

Stress strained state change in the «deformed building – pile foundation – base» system resulting from supplying the slab under the grilles

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Presented features of a new analytical model of the "deformed building - driven prismatic piles as a part of a continuous grille - soil base with a weak underlying layer" system before and after supplying the monolithic reinforced concrete slab under the existing foundation grilles. Also, this system's stress strained state (SSS) simulation results by the finite element method (FEM) for evaluation of the combined action features of the system's components were presented. It has been published the new empirical data on the SSS change in the "deformed building - driven prismatic piles as a part of a continuous grille - soil base with a weak underlying layer" system resulting from supplying the monolithic reinforced concrete slab under the existing foundation grilles.

Keywords: soil base, poor-bearing soil, driven prismatic pile, monolithic reinforced concrete grille, settlement, crack, stress-strained state, monolithic reinforced concrete slab

Зміни напружено-деформованого стану системи «деформована будівля – пальовий фундамент – основа» внаслідок підведення під ростверки плити

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Викладено особливості нової розрахункової схеми системи «деформована будівля – забивні призматичні палі у складі стрічкового ростверку – ґрунтова основа зі слабким підстильним шаром» до та після підведення під існуючі ростверки монолітної залізобетонної плити й результати математичного моделювання з використанням методу скінченних елементів напружено-деформованого стану (НДС) цієї системи для оцінювання особливостей спільної роботи її складових. Розрахунок виконувався методом скінченних елементів у просторовій (3D) розрахунковій схемі з урахуванням спільної роботи надземних і підземних конструкцій, пальового фундаменту та основи під ним. При оцінюванні НДС будівлі ґрунтова основа існуючих фундаментів умовно замінювалася відповідними коефіцієнтами. Посилення полягало в підведенні під ростверки залізобетонних балок L-подібного обрису, об'єднаних поперечними балками, а зверху – монолітною плитою товщиною 200 мм. Отримано ребристу плиту підсилення з ребрами до низу, основою якої є пісок намивний, дрібний, середньої щільності. Ця конструкція добре перерозподіляє напруження від нерівномірних деформацій основ і має значну жорсткість за мінімального об'єму земляних робіт. Оприлюднено нові дослідні дані про зміну НДС системи «деформована будівля – забивні призматичні палі у складі стрічкового ростверку – ґрунтова основа зі слабким підстильним шаром» унаслідок підведення під існуючі ростверки монолітної залізобетонної плити. Моделювання НДС системи після посилення фундаменту показало, що фактичне створення плитно-пальового фундаменту значною мірою прибрало нерівномірний характер розподілу напружень і наблизило його до початкового стану. Доведено достатньо високу ефективність та надійність способу посилення пальових фундаментів у складі стрічкового ростверку підведенням плити.

Ключові слова: ґрунтова основа, слабкий ґрунт, забивна призматична паля, монолітний залізобетонний ростверк, осідання, тріщина, напружено-деформований стан, монолітна залізобетонна плита

Introduction

The choice of specific constructive-technological decisions of strengthening (reconstruction) of each deformed building on the pile foundation is carried out only after careful estimation of the technical condition of load-bearing building structures and investigation their bases and foundations parameters, by engineering inspection and establishing the causes of excessive deformation of the foundations' base [1 - 4].

Previously tested constructive-technological decisions of the increasing the load-bearing capacity of pile foundations of deformed buildings and structures by pushing piles to strong soil and installation of offset piles with sufficiently high reliability of their use results have a very high hand labor-intensity and require a long period of work. Therefore, it is advisable to improve adequately efficiency for conditions of sizable uneven deformations of buildings, and, at the same time, less labor-intensive and more prompt decisions to increase the bearing capacity of pile foundations with their or soil base SSS change, such as strengthening of driven prismatic piles foundations in a continuous grille by supplying a monolithic reinforced-concrete slab under the existing grilles [3 – 6].

Review of the research sources and publications

The correctness of soil state models and geomechanical models of 2D and 3D versions of FEM regarding calculations of combined action of piles as a part of continuous and plate grilles with the base is justified [5 – 14]. Thus, the 3D PLAXIS version of the complex has a whole library of nonlinear models of soil mechanical behavior (ideal elastic-plastic with the Mohr-Coulomb strength criterion; isotropic compaction (strengthening); weak creeping soil, etc.), makes it possible to vary the geometry of foundations, track the SSS stages of the base at different paths of loading, has a convenient interface for outputting results. It is well tested for SSS estimation of pile-slab foundations, pile groups combined by a grille of strip pile foundations [7 – 16].

In particular, Professor I. Boyko and his school [13] solved several problems of modeling SSS of the "plate-pile-base" system in the VESNA complex using an elastic-plastic soil model based on the dilatancy theory of V. Nikolaevsky, with the criterion of the plastic flow condition of Misesap-Schleicher-Botkin in Boyko's modification. It has been proved that taking into account the mutual influence of neighboring pile foundations, the deformation behavior of the base significantly changes, increasing the settlement of the foundations of the sectional high-rise building up to 30%, and the values of bending moments in the slabs joint area of the sections increase by 1.5-2 times.

I. Mayevska and N. Blaschuk [14] in the 3D version of PLAXIS investigated the influence of several factors on the bearing capacity and deformability of the system "continuous grille – driven piles of constant cross-section – soil" before and after strengthening by piles jacking: the piles spacing; distances between pile rows; pile lengths; type of piles (with and without removal of soil); soils. It was defined as the fractions of the load perceived by a new grille - from 5 to 65% and the grille, installed in strengthening by piles of the continuous footing - from 30 to 72%. The bearing capacity of pile foundations was estimated by the criterion of the limit value of a settlement.

Specialists of the Poltava geotechnical school [15, 16] by modeling in the 2D and 3D versions of PLAXIS the system "continuous grille – cast-in-place piles in drilled wells – wet loess base" using an elastic-plastic soil model and step-iteration procedures obtained a relative error of 15% compared to long-term natural object leveling data. For both tasks, the possibility of correct accounting for the soil's heterogeneity in the "zone of influence" of piles has been proved. Both simulated and experimental "load - settlement" graphs are curvilinear, that is, with the achievement of a second critical force on the system, the soil around the piles, its broadenings, and under the grille works in the plastic stage. It has also been established that the deformation modulus by the compression tests of wet loess soils should be taken into account during simulation without increasing factors.

Definition of unsolved aspects of the problem

The FEM has the greatest value in solving the so-called "complex geotechnical problems" – with the complex geometry of the design configuration [2, 5, 7, 9, 13], in particular, for estimating the SSS of the "soil mass - existing surrounding buildings - new structures" system in dense urban development conditions and taking into account the stages and technology of operations (fragments of the pit, fixing of its walls, building of the underground part of the structure, loading of it or its components during the erection of surface structures, etc.) as well as complex engineering and geological conditions and adverse effects (overwhelmingly use the software products PLAXIS, DIANA, FLAC, VESNA, FEM models and several others in the 3D version of FEM). Therefore, according to Professor V. Ilichov, a new direction of soil mechanics has already been developed - "technological soil mechanics" [5].

Therefore, it makes sense to test modeling with the 3D version of the FEM and the elastic-plastic soil model (Cam-Clay, Soft soil creep model, etc.) for the SSS estimation of the "deformed building – pile foundation (before and after reinforcement) – soil-based" systems.

In addition, when estimating the SSS of the "deformed building – pile foundation – base" systems, significant systemic discrepancies were established in the values of the pile`s bearing capacity calculated by the standard method, according to which soil resistance under the toe and along the side surface is obtained depending on the depth of their immersion, the liquidity index of clay soil or sand`s grading, and by the results of static tests of piles [16].

Problem statement

Therefore, the purpose of the actual work was: to develop a calculation scheme of the "deformed building – driven prismatic piles as part of a continuous grille – a soil base with a weak underlying layer" system before and after supplying a monolithic reinforced concrete slab under the existing grilles and to perform this system's SSS mathematical modeling using the FEM to estimate the features of combined action of its components.

Basic material and results

Put in commission in 1977, the end block of the five-story residential building with a basement in Horishni Plavni of Poltava oblast is resting on driven prismatic piles (9 m length, 350x350 mm section), joined by a continuous grille. The end block has sizable fractures in bearing masonry walls because a part of the piles is driven higher than the designed depth, and its toes are found in fluid sandy loam with silt layers [3].

Aside from that, due to the negative skin friction effect caused by self-compacting and mechanical suffusion in the upper layers of the bulk sands after rupture of the main thermal pipeline, which was intensified by inertial forces from the explosions in the quarry, the design load on pile dropped to $N = 268$ kN, which is less than the stress from the building 404.5 kN. Therefore, the pile foundation base's settlement was already in the nonlinear stage, which led to the emerging and development of respective non-uniform limit-exceeding deformations in it [4].

In structural regard, the building is a structure with longitudinal load-bearing brick-built walls. The height of the floors is 2.8 m, and the basement is 2.2 m. Its spatial rigidity is provided by transverse walls of a stairwell and floor slab disks (fig. 1).

The building's structural scheme cannot be considered rigid, and the technical condition of its pile foundation is qualified as insufficient [6].

The reinforcement consisted of the supply under the grille monolithic reinforced concrete L-shaped beams (900 mm height), joined by transverse reinforced concrete beams, and on top – by a monolithic 200 mm thick slab. The concrete for grille elements strengthening is of C20/25 strength grade.

A ribbed strengthening plate was obtained, which was based on the alluvial fine sand of medium density. Its ribs were directed to the bottom [6]. This design effectively redistributes stress from uneven deformations of the base and has considerable rigidity at the minimum volume of groundworks.

The design was simulated by FEM in spatial (3D) configuration with consideration of combined action of underground and superstructure, pile foundation, and base beneath it. In the evaluation of the building's SSS, the soil base of the existing foundation was conventionally replaced by the respective coefficients.

Spatial FEM configuration of the building block's foundation before its strengthening is shown in fig. 2, and after the strengthening – in fig. 3 and fig. 4 (schematic view fragment).

A simulation was carried out for the following phases:

1) the building's completion moment;
2) after the base wetting (flooding) from the pipeline malfunction;
3) after the foundation strengthening.

As initial conditions, the settlement factor was set to be k=7000 kN/m (fig. 5), for the flooding state the reduction of its load-bearing capacity was considered in certain areas [3], where k=5000 kN/m (fig. 6).

The respective analytical 3D model of the building is shown in Fig. 7 and fig. 8. It includes the building's load and influences data and its physical model (3D system of walls, slabs, beams, its joints, foundation, and base, as well as data on the physical and mechanical properties of materials).

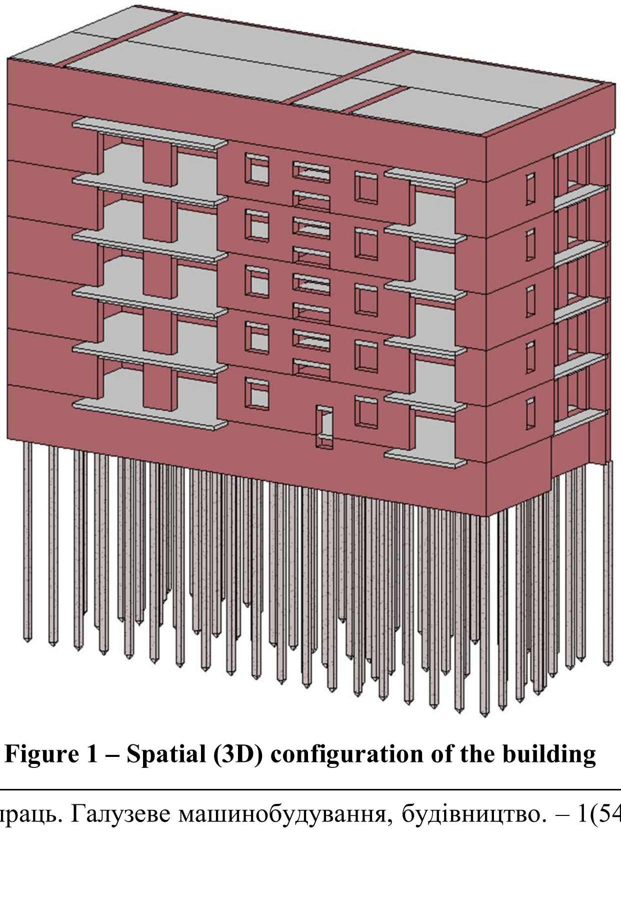

Figure 1 – Spatial (3D) configuration of the building

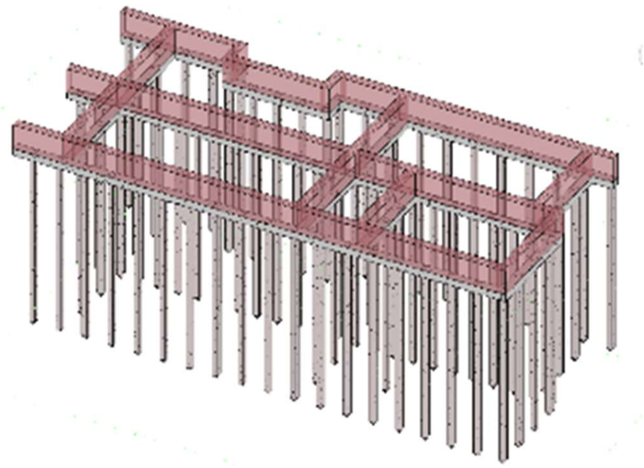

Figure 2 – 3D scheme of foundation before the strengthening

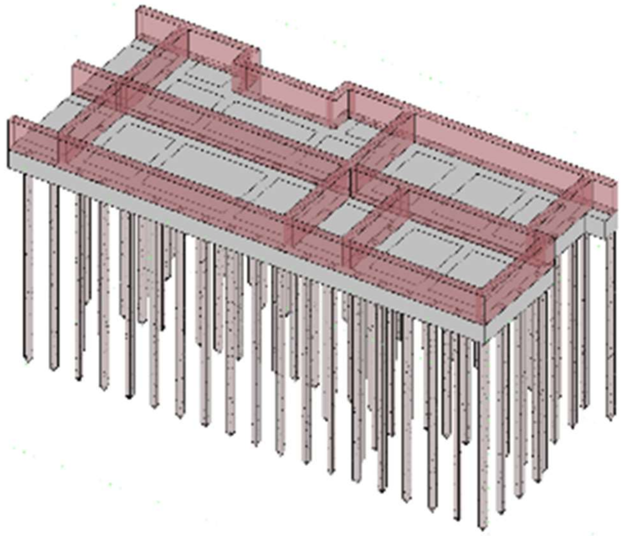

Figure 3 – 3D scheme of the foundation after the strengthening

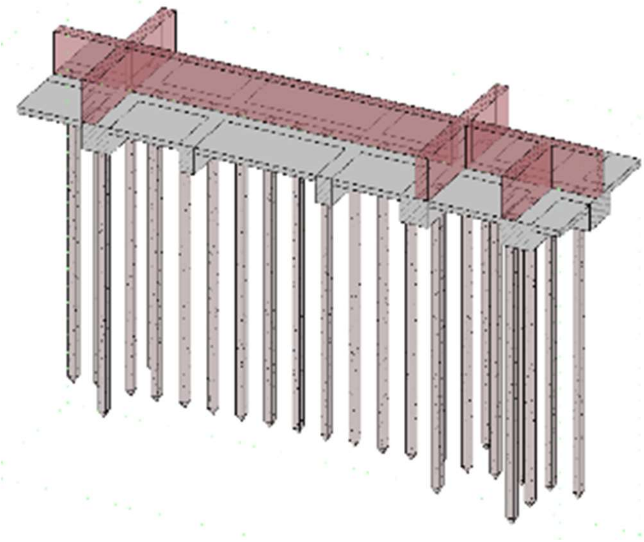

Figure 4 – View fragment of 3D foundation scheme after strengthening (along the inner longitudinal bearing wall)

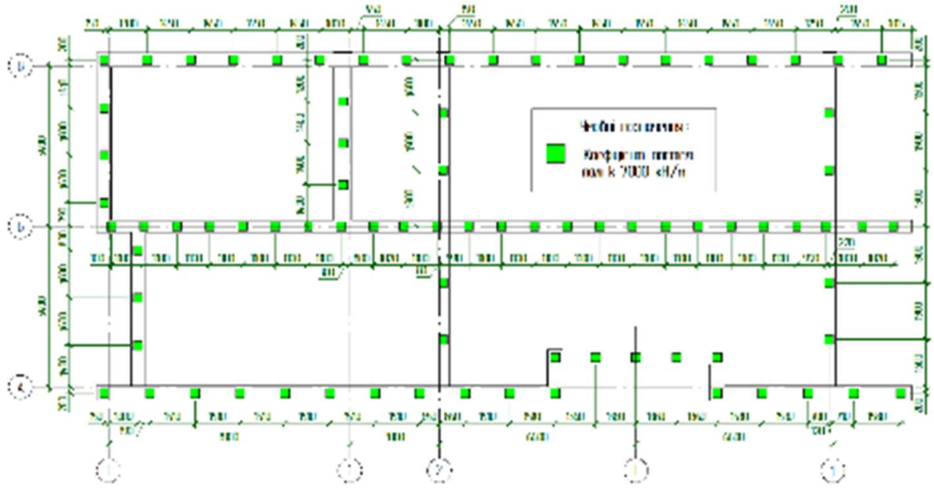

Figure 5 – Piles' initial rigidity, adopted in the simulation

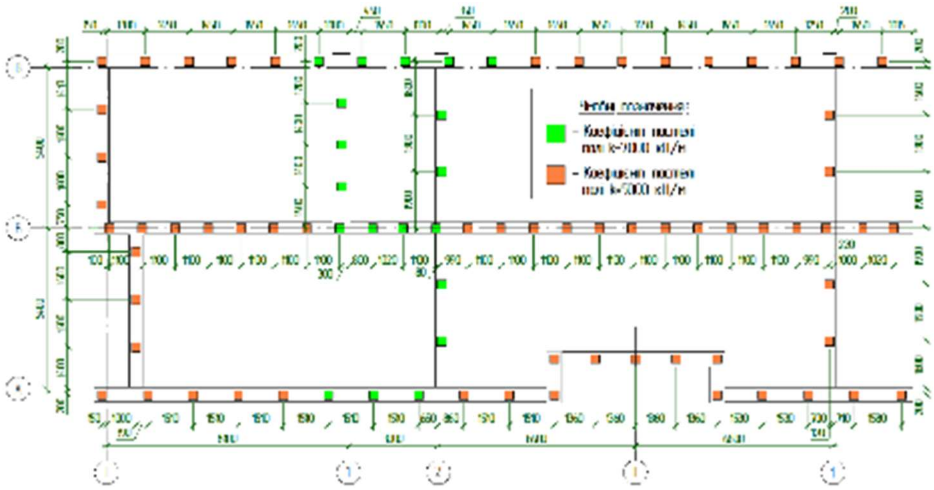

Figure 6 – Piles' rigidity, adopted in the simulation, after the base flooding

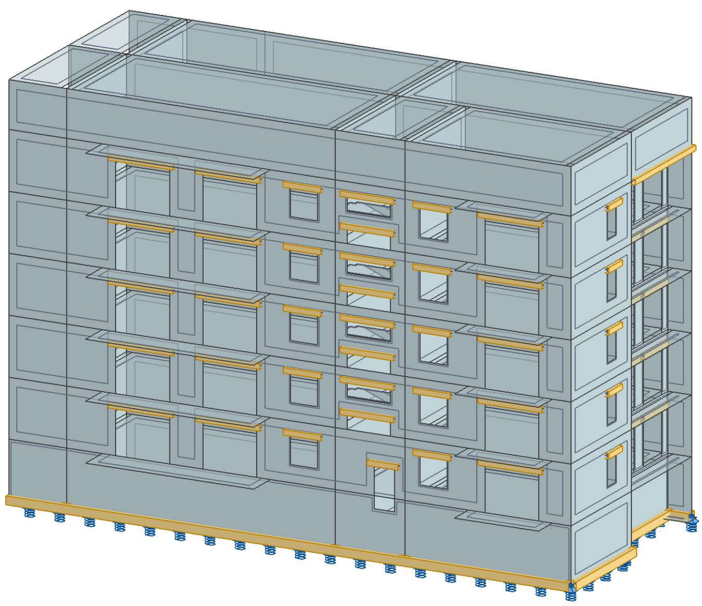

Figure 7 – 3D building's analytical model (pile foundation is set as elastic bearing)

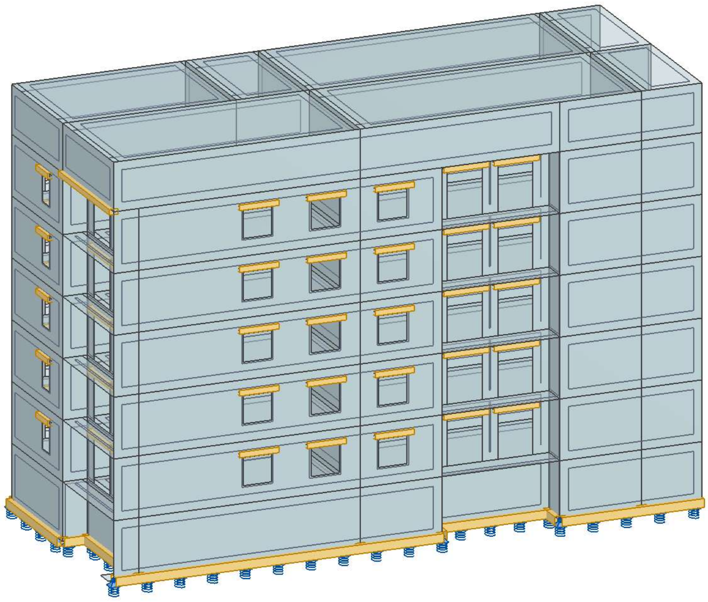

Figure 8 – 3D building's analytical model (pile foundation is set as elastic bearing)

The model of the building foundations before and after their strengthening is given in Fig. 9 and fig. 10.

The scheme of loads application for this building is shown in fig. 11. In this model for each floor, the characteristic values of the permanent load were set to be 4.9 kPa (floor weight, floor slabs, and parting walls), the characteristic values of variable sustained load – 1.5 kPa. The characteristic values of permanent load on the fifth-story flooring are set to be 4.84 kPa (floor weight, floor slabs), the characteristic values of variable sustained load – 0.7 kPa. The characteristic values of permanent load on the covering are 3.94 kPa (ruberoid sheet, roofing slabs, etc.), characteristic values of snow load – 1.28 kPa.

3D building's finite element model is shown in figure 12.

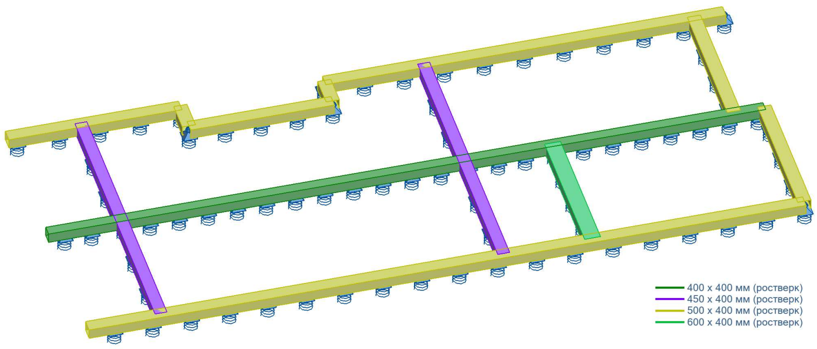

Figure 9 – Building's foundation model before the strengthening

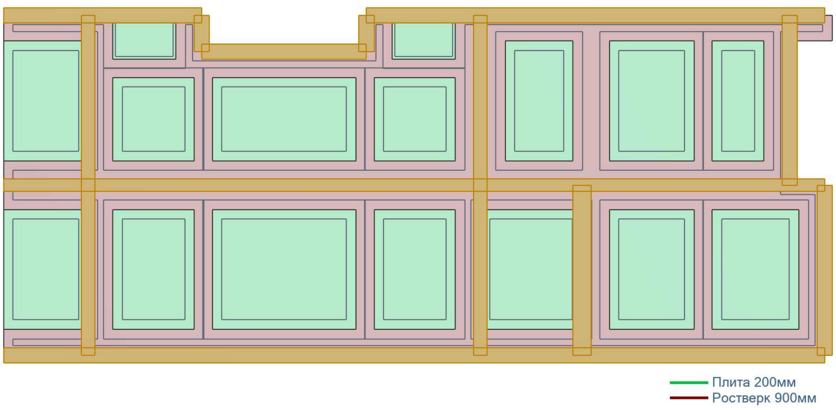

Figure 10 – Building's foundation model after its strengthening

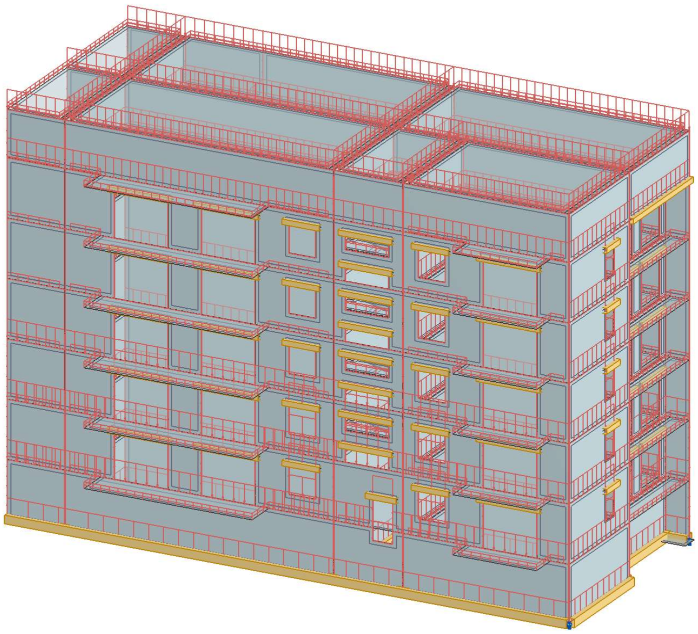

Figure 11 – 3D load application configuration for the building

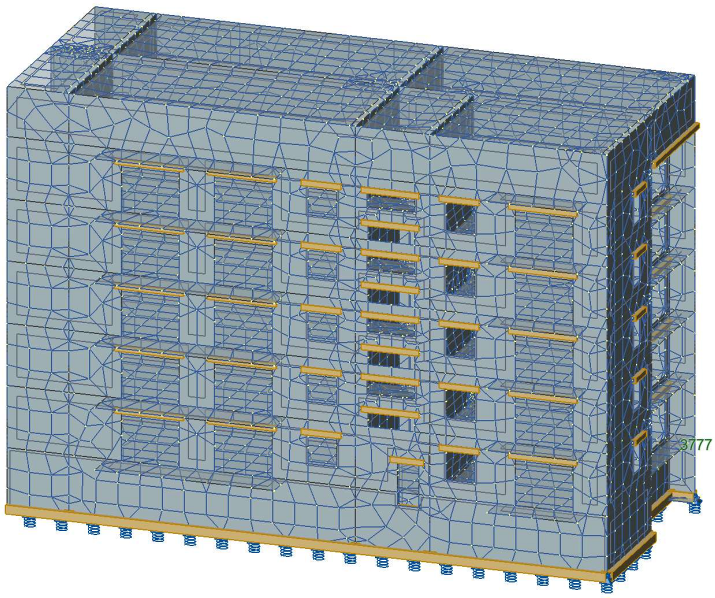

Figure 12 – 3D building's finite element model

Thus complex spatial geometrical schemes are simplified by replacement of a real design by the conventional scheme, for example, beams were approximated by bars, reduced to the axis, and slabs with walls were replaced with disks, reduced to the middle plane.

Calculation of structural configuration was carried out in spatial structural design with account for combined action of underground and superstructure, foundation and its base. In the building's SSS evaluation, the soil base of the existing foundation was replaced by elasticity factors, obtained in the spatial simulation of the building's structures. In structural configuration calculation, the actual reinforcement of reinforced concrete units and thus its non-linear operation was disregarded.

In the result of a structural configuration of the building calculation it's been defined the following: in the flat floor, covering, and foundation slabs – values of torsional and bending forces, shear and axial forces; in walls – values of normal and tangential axial forces, torsional and bending forces, and shear forces, etc.

In the building's numerical model, there were the following preconditions adopted: walls and floor slabs had hinge joints, as well as walls and grille joints. In the frame's numerical model, there were design parameters of strength, rigidity, and geometrical parameters of structures used. Floor slabs, stairwell, walls, and foundation reinforcement were simulated by disk elements, continuous grilles, and beams – by bar elements with respective axial and bending rigidity. The remaining structures, that had no impact on the spatial rigidity of the building (partying walls, floor, ceiling, roof, etc.), were represented by the equivalent load, applied in respective areas of the numerical model.

Using the evaluation results of the SSS of the "deformed building – driven prismatic piles as a part of a continuous grille – soil base with a weak underlying layer" system before and after supplying the monolithic reinforced concrete slab under the existing foundation grilles by FEM, let's analyze mainly the distribution of normal stress (to the horizontal plane) in-wall elements against the combination of existing loads (that include permanent and temporary with the limit design values).

The obtained stress values were compared to the design strain strength of brickwork in the traced section and compressive strength. Aside from that, it was considered that the brickwork was made of silicate brick M75 and grout M25.

According to table 9 (DBN V.2.6-162:2010, Appendix R) design strain strength of brickwork in the traced section is f_{bk2}=0.11 MPa. According to table 1 (DBN V.2.6-162:2010, Appendix R) design, the compressive strength of brickwork is f_d=1.10 MPa.

At the first stage of the simulation, the initial nature of the strains was determined in the brickwork at the time of the building`s construction before defects occurred for the faces 2-I and I-2 (respectively Fig. 13–14).

The compression strain does not exceed the maximum (f_d=1.10 MPa), and the tensile stress in the horizontal plane in some areas (for example, the places where the balconies are installed) exceeds the maximum values (f_{bk2}=0.11 MPa), which may be the result of a mismatch between the design model and between the real stress distribution scheme in the brickwork. But also in the walls around the stairwell, the tension of brickwork stressing is traced, that in some areas reaching the point of brickwork`s break by tensioning along a bandaged section, which may be the result of unsuccessful volumetric planning decisions of the building or errors in the design of the building foundation.

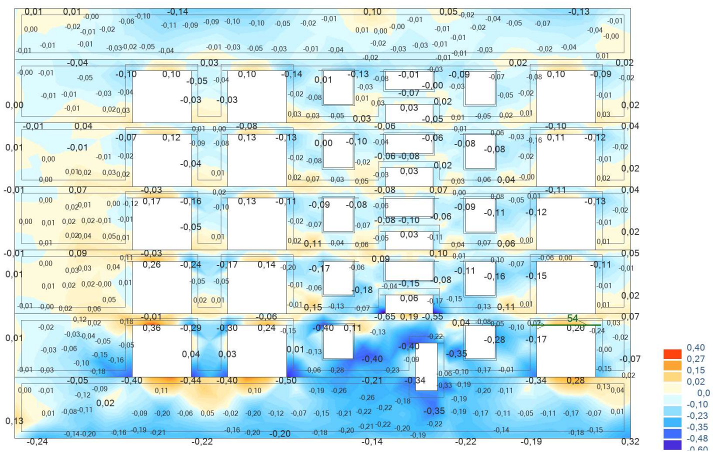

Figure 13 – Normal stress distribution view in the brickwork of outer walls on the 2-I face before the defects occurring

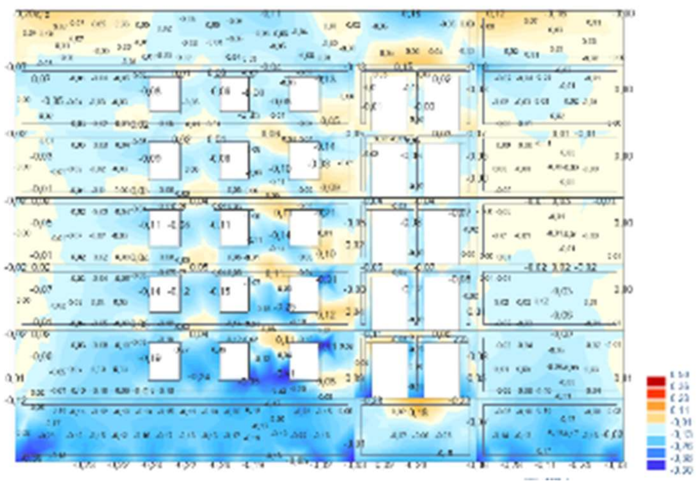

Figure 14 – Normal stress distribution view in the brickwork of outer walls on the I-2 face before the defects occurring

At the second stage of the calculation, the changes in the pile`s bearing capacity as a result of heating pipeline malfunction were considered, and the new behavior of the brickwork`s stress distribution behind the main faces was determined. By the nature of the stress distribution, it is possible to generalize that due to the flooding of the base, as a result of a pipeline accident, there was an uneven settlement of the left and right parts of the building around the stairwell (which because of the actual location of the pile as part of pile field has high rigidity). Therefore, most cracks are concentrated around the stairwell. On the stress distribution views (Fig. 15 and Fig. 16), the lines emphasized the places where the strength of masonry by tension is exceeded (f_{bk2}=0.11 MPa), and therefore the most possible occurrence of vertical cracks along a traced section. Values of compression strains do not exceed permissible values (f_d=1,1 MPa).

Figure 17 and Figure 18 show the layout of defects (cracks) of external bearing walls by the geotechnical monitoring data [4]. As can be seen, the locations and behavior of the theoretical (modeled by FEM) vertical cracks along the traced section are practically the same as the results of the surveys.

The third stage of the FEM modeling additionally takes into account the work of foundation reinforcement elements. The relevant stress distribution schemes (Figures 19 and 20) show that the foundation`s reinforcement has significantly helped to remove the uneven behavior of the stress distribution and bring it closer to the initial state. At the same time, the features of the tension stress distribution in the brickwork around the stairwell are saved, and their values either do not exceed the ultimate tensile strength of masonry (f_{bk2}=0.11 MPa) or exceed them in places similar to the initial state of the building.

The foundation reinforcement project was carried out [6]. A ribbed reinforcement plate is obtained, the base of which is the wet sand, fine, medium density. The ribs of the foundation slab are directed to the bottom. It efficiently redistributes stress from uneven deformations of the foundations and has significant rigidity with a minimum amount of groundworks, that is, in the process of strengthening a slab-pile foundation was arranged.

As geotechnical monitoring showed, new cracks in the building`s walls at the time of strengthening and the building's subsequent operation did not occur [4].

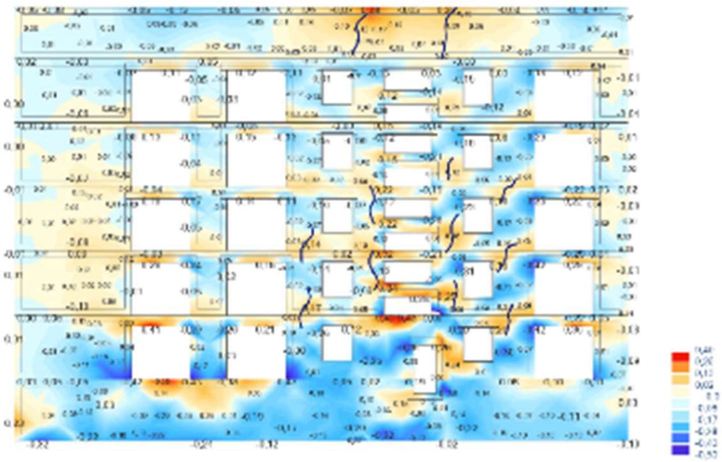

Figure 15 – Normal stress distribution view and respective fractures in the brickwork of outer walls on the 2-I face after the base flooding

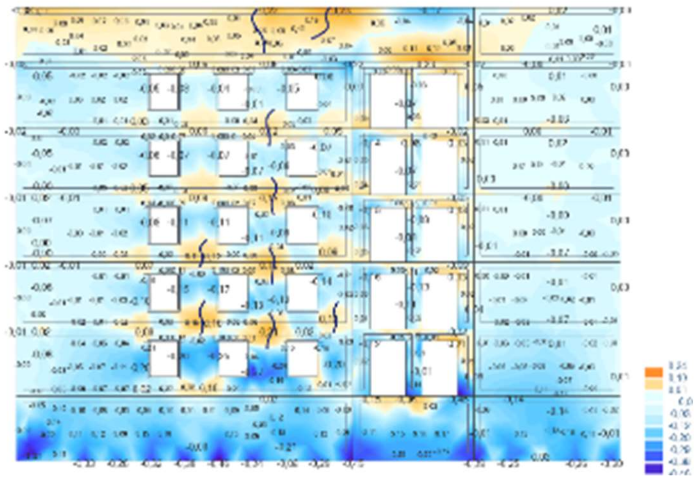

Figure 16 – Normal stress distribution view and respective fractures in the brickwork of outer walls on the I-2 face after the base flooding

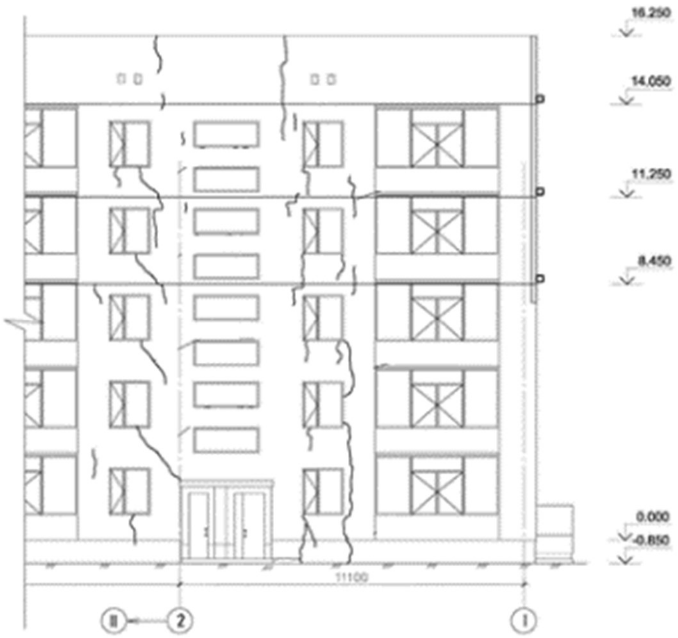

Figure 17 – Scheme of defects and damages in the brickwork of the outer walls on the 2-I face, along B axis (left end block section I-II) by the geotechnical monitoring data

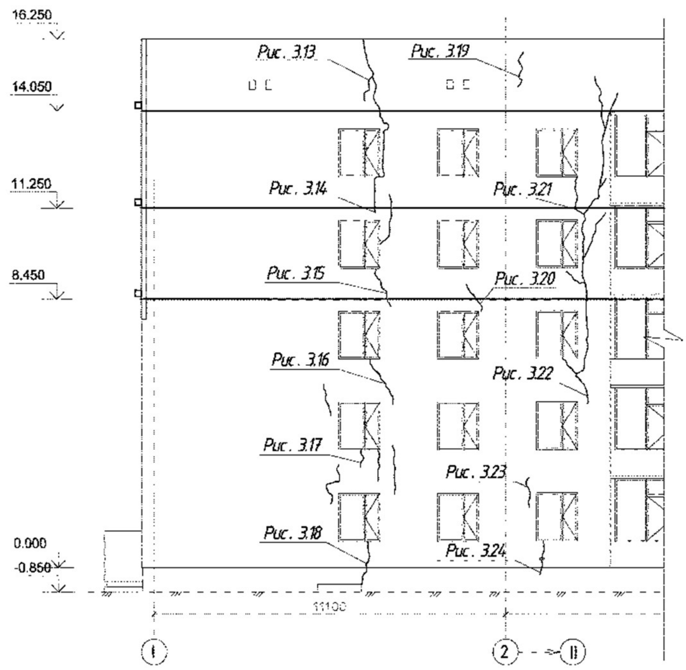

Figure 18 – Scheme of defects and damages in the brickwork of the outer walls on the I-2, face, along A axis (left end block section I-II) by the geotechnical monitoring data

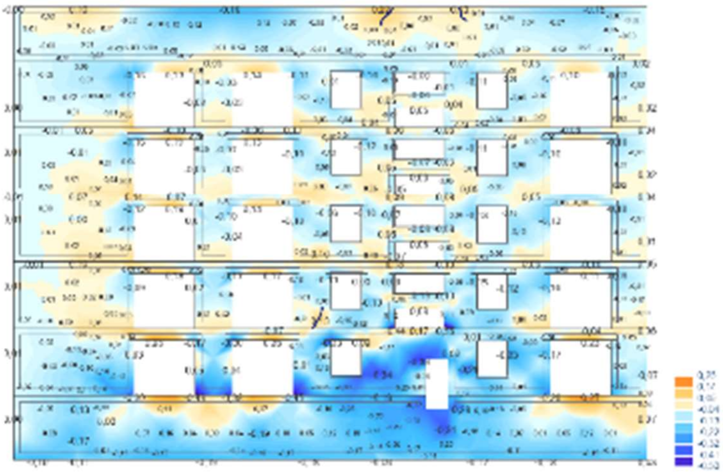

Figure 19 – Normal stress distribution view in the brickwork of the outer walls on the 2-I after the foundation strengthening

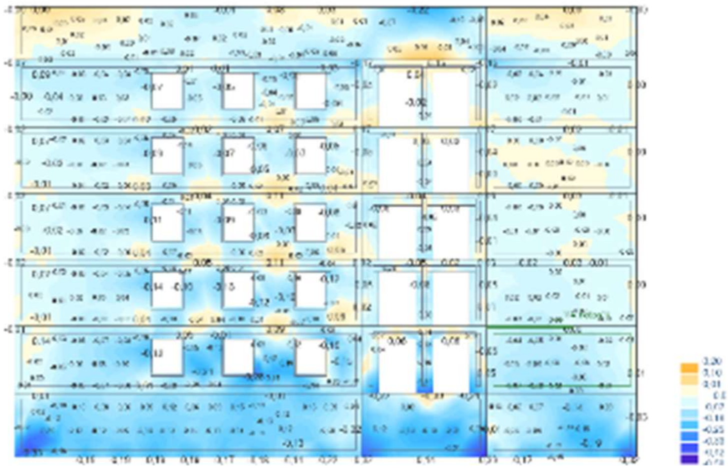

Figure 20 – Normal stress distribution view in the brickwork of the outer walls on the I-2 face after the foundation strengthening

Conclusions

Thus, a new 3D design scheme of the "deformed building – driven prismatic piles as part of continuous grille – a soil base with a weak underlying layer" system was developed before and after supplying monolithic reinforced concrete slab under the existing grilles, and mathematical modeling was carried out using the FEM of this system's SSS to estimate the features of combined action of its components.

New experimental data were obtained about the SSS change of the "deformed building – driven prismatic piles as part of a continuous grille – soil base with a weak underlying layer" system due to the supplying of a monolithic reinforced concrete slab under the existing grilles. The simulation of the SSS system after strengthening the foundation showed that the actual creation of the slab-pile foundation significantly mitigated the uneven behavior of the stress distribution and brought it closer to the original state. Sufficiently high efficiency and reliability of the method of the pile foundations strengthening as a part of the continuous grille by supplying the slab has been proved.

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