Reinforcement of a deformed structure on the pile foundation

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Abstract. The results of geomonitoring of the technical state of a five-storeyed building erected on driven prismatic piles are presented. The cause of excessive deformations of the foundation base is the inability of the pile tip to reach the design depth, which led to a significant reduction in their load-bearing capacity. The constructive technological solution of strengthening of the bases from piles as a part of a tape grid by bringing under existing grids of a monolithic reinforced concrete slab is improved. A new analytical model of the system “deformed building – driven prismatic piles as a part of a tape grid – soil base with a weak underlying layer” before and after supplying the slab under the existing foundation grids. The stress-strain state (SSS) of this system is estimated using the finite element method (FEM). New data of the change in the SSS system after supplying the slab under the existing grids were obtained. The results of the project execution (without resettlement of the residents of the house) and the respective geomonitoring are presented.

1. Introduction

The causes of excessive settling of structures are errors in engineering and geological surveys, poor design, violation of the rules of construction work and operation, and for buildings on piles also the use of increasing factors on the data of laboratory tests of compressible soils; getting the tip of the piles into a layer of weak soil; immersion of piles above the design mark; overestimation of bearing capacity of piles due to non-compliance with the normative “rest”; misinterpretation of the schedule “load – pile settling”; too close placement of piles; uneven loading of them in a grid; disregard for the loading of adjacent areas under conditions of compact planning; driving in piles near buildings, development of ditches, etc. [1-3].

Reliable taking into account the influence of these factors, as well as components of absolute and relative uneven settling (compaction, uncompaction, protrusion, restructuring and due to changes in stress or strain of the foundation base during operation) on the value of strains of pile bases foundations of buildings, especially using the methods of classical soil mechanics, is either difficult or requires time-consuming and labor-intensive field and laboratory studies of soils. [4].

The most proper way to estimate the efficiency of alternatives for strengthening of pile foundation for strain building is possible by combining an analytical model of the system “strained building - pile foundation – soil base” before and after foundation strengthening and executing mathematical modeling using FEM and SSS soil this system to estimate the features of combined action of its components. [5-8].

Design and technological solutions to increase the bearing capacity of pile foundations of strained buildings and structures by pushing piles to strong soil and installation of offset piles with sufficient reliability of their performance have a very high hand labor-intensity and require a long period of work [9]. Therefore, it is advisable to improve sufficiently effective for conditions of sizable uneven strains of buildings, and at the same time less labor-intensive and more prompt solutions to increase the
bearing capacity of pile foundations with their SSS changes or base soils, such as strengthening of driven prismatic piles foundations in a continuous grid by supplying a monolithic reinforced-concrete slab under the existing grids [10].

In the practice of reconstruction of buildings, including strengthening of their bases and foundations, the general system of geotechnical monitoring and scientific and technical support [11], which is even included in building norms, has arisen [12]. However, for objects with the system “strained building - driven prismatic piles as a part of tape grid - soil base with a weak subsoil” one of the most vulnerable spots is the difficulty of reliable determination the parameters of bases and foundations, in particular, the actual length of piles, and for follow-up computations – the complexity of combining the analytical model of this system before and after strengthening for reliable estimating the features of combined action of its components.

2. Purpose of the Work
The purpose of this study is to improve the structural and technological solution of foundation on prismatic cylindrical piles joint in continuous footing (pile cap) by the example of a distinctive actual object and assess the influence of supplying the monolithic reinforced concrete plate under the existing grids on the SSS of the “strained building – driven prismatic piles joint by continuous footing – soil base with loose underlying strata” system.

3. Methodology and Research

3.1. Geotechnical monitoring results
A typical full-scale object of this study, which was strengthened without the resettlement of its inhabitants during this period, is a five-storeyed residential building with a basement and crawl space, located in Horishni Plavni, Poltava region. It was built in 1977 according to a standard project. It consists of three-block sections, of which the left-handed end has undergone sizable strains (Figure 1). In terms of construction, the building is a structure with longitudinal load-bearing walls. The height of the floors is 2.8 m, and the basement is 2.1 - 2.2 m.

![Figure 1. End block section of the strained building.](image)

The walls are made of 1.5 sand-lime bricks on the cement-sand matrix. The thickness of the outer longitudinal load-bearing and end walls is 510 mm, and the thickness of the inner longitudinal load-bearing wall is 400 mm. The load-bearing walls in the basement are made of blocks and brickwork. Spatial rigidity is provided by transverse walls of a stairwell and disks of inserted floor overlapping.

The structural scheme cannot be considered rigid. The main heating system passes through the basement.

Investigating the strained block section established that during the construction and operation of the house, there were defects and damages that affected the load-bearing capacity and wearing qualities of individual elements and the building as a whole, in particular, vertical cracks with a width of up to 20 mm (Figure 2) and internal load-bearing walls mainly in the places of support of the spandrel and over (under) perforations. They are indicative of sizable strains of the base foundations of the section in the direction of partition off the main part of the house.
Figure 2. Vertical cracks with opening width: a – 7 mm; b – 15 mm; c – 12 mm; d – 4 mm (in the basement).

Their development significantly reduces the spatial rigidity of the section, because, for the outer and inner walls divided into parts, this rigidity is no longer provided by the transverse walls of the stairwell. There are no cracks on the outer and inner walls of the stairwell, which confirms the previous statements.

Minor cracks in the walls and between the floor slabs appeared immediately after the settlement of the house in 1977 but it underwent the largest deformations in 1993 after the heating main breakthrough.

At the same time, pretensioned reinforcement strands were arranged (Figure 1) from reinforcing bars with a diameter of 36 mm with tightening screws to create tension in them. Horizontal beams of the rectangular section from two welded channels No.24 established on face walls, fastening of rods to beams secured through the equilateral L-shaped member 140x10, which is installed at the corners of the building.

Since the free length of the strands between the fixing points should not exceed 15 - 20 m (in our case - 67.2 m) the cracks opened further. At the level of the third and fifth floors overlapping, there was a break in the tension bars.

At the level of the socle, marks were arranged around the perimeter of the building, which were leveled throughout the ge-monitoring period. Plaster beacons are set on the external and internal load-bearing walls. Cracks in them indicate the further development of deformations. There are no cracks on the end wall of the block section. The technical condition of the load-bearing walls is considered as unsatisfactory.

The foundations are driven prismatic piles 9-35 (9 m length, 350x350 mm section), combined by a strip grid of 400 mm height (Figure 3).

Figure 3. Spatial model of the foundation of the building section.
Under the inner load-bearing wall, the grid width is 400 mm, and under the outer wall, it is 500 mm. The increment of the piles under the inner wall is 1100 mm, under the outer wall is 1360 to 1530 mm, and under the end walls - 1590 to 1610 mm.

Within the boundary of the site under the bulk layer (Strata-1a) and fine sands (Strata-2c and Strata-2p, respectively), of medium density and dense with modulus of deformation, respectively, $E = 19.5$ and 35 MPa) with a total depth of about 7 m, there is a layer of buried soils (Strata-3) - layered sandy loams, with layers of silt and clay, fluxional ($E = 6.5$ MPa, organic matter content - 8%), which are underlined by medium-sized alluvial sands of 9-10 m deep (Strata-4, $E = 45$ MPa, and from a depth of about 18 m - by the clay.

Groundwater level (WL) at the time of the survey was 6.8 - 7.3 m from the earth surface.

Its annual and seasonal fluctuations reach 1.5 m from this level. Unfavorable engineering-geological processes have been identified within site dynamic impact on sandy soils from career explosions, which can lead to their dynamic liquefaction; mechanical suffusion during the operation of water-bearing communications; rather thick (up to 2.3 m) soil with impurities of organic matter.

The bearing capacity of the piles $F_d$ here was estimated using the well-known three component formula

$$F_d = \gamma_e \left[ l_{cr} RA + \sum h_i \left( \gamma_{cr} u_i f_i + \gamma_{ct} v_i E_i k_i \zeta_i \right) \right],$$

where $\gamma_e$ - coefficient of pile operation’s condition in soil; $\gamma_{cr}$ and $\gamma_{ct}$ - coefficient of soil operation under the foundation footing (bottom end) and along the lateral surface of a pile, which account for its manufacturing features; $R$ and $f_i$ - soil design strength under the foundation footing and along the lateral surface of a pile respectively; $A$ - pile support area on soil; $h_i$ - height of $i$-th soil strata, tangent to a pile’s lateral surface; $u_i$ - outer perimeter of $i$-th cross section of a pile; $v_i$ - side sizes sum of $i$-th cross section of a pile, which are deviated from vertical; $E_i$ - deformation module of soil $i$-th strata; $k_i$ - soil type dependent coefficient; $\zeta_i$ - rheology coefficient.

In the presented case for driven pyramidal piles the third component of the expression was equal zero. The building pile foundation settling was determined as for a reference soil massif at the pile tip level using a well-verified method of stepped summing.

$$S = \beta \sum_{1}^{n} \sigma_{ip} h_i E_i^{-1},$$

where $\sigma_{ip}$ - average value of additional stress in the $i$-th elementary strata; $h_i$, $E_i$ - height and deformation module of the $i$-th soil layer respectively; $n$ - number of elementary layers within the massif compressed under the reference foundation footing.

The project provided for penetrating the buried soil with 9-35 piles that reached Strata-4. In this case, the calculation revealed that: the load on the pile under the inner and outer bearing wall is 404.5 and 390.6 kN, respectively; pile's bearing capacity is $F_d = 1334.8$ kN; allowable design load $N = 953.4$ kN; the settling of the base of such foundation is $S = 1.44$ cm.

Considering the above and the worst possible scenario, under which accompanied by the rapture of the thermal pipeline could have the effect of “negative friction” on the lateral surface of the pile, the magnitude of which can reach 317 kN (soil layers Strata-2c and Strata-2p), allowable calculated load on the pile $N = 499.5$ kN which still exceeds the pile load from the structures 404.5 kN.

Therefore, the actual length of the piles in the foundation was checked. Control of continuity and length of piles is carried out by an acoustic method by means of Pile Integrity Tester PIT – W complex. For these tests, six trial pits were performed and the pile body was cleared by 20 cm. (Figure 4).

It was found that the actual length of the piles was 4.5 - 8.5 m. The results of the instrumental verification of the actual length of the piles and its integrity estimation are summarized in Table 1 (in particular, the pile # 119 operates on eccentric compression due to the considerable eccentricity of loading and pile's body deflection from vertical; the crack opening in the pile - 40 mm).
That is, part of the piles was not immersed in the design depth. The calculation was performed for pile # 119 (shaft # 1) of a 4.5 m length with a cross-section of 350х350 mm. A vertical tie of this pile to the geotechnical column is shown in Figure 5.

![Figure 4. Pile within the grid view from the trial-pit and the pile length control.](image)

**Table 1.** The results of the instrumental verification of the actual length of the piles and its integrity estimation.

| Number of pitch / pile | Actual length of piles (m)          |
|------------------------|-------------------------------------|
| 1/119                  | the pile is destroyed               |
| 2/105                  | 8.5                                 |
| 3/9                    | 4.5                                 |
| 4/77                   | 8.5                                 |
| 5/171                  | 6.0                                 |
| 6/33                   | 7.5                                 |

![Figure 5. The vertical tie of the 4.5-meter-long pile to geotechnical column according.](image)

The settling of the pile is \( S = 2.2 - 2.5 \) cm based on the calculation method. Thus, the difference between the settlement of piles with a length of 9 m and 4.5 m is about 1 cm, which could not cause actual over-deformation. However, given the occurrence of "negative friction" effect due to 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 bearing capacity of the pile decreased to \( F_d = 375.7 \) kN, it is up to \( N = 268.0 \) kN, which is less than the vertical force from the structures of the building 404.5 kN.
When the permissible load on the pile was exceeded, the pile foundation base settlement was already in the nonlinear stage, which led to the appearance and development of existing deformations of the building. Provided that if all the pile tips reached the design mark, then, of course, such deformation would not occur.

3.2. Simulation results of stress strain state of the “building – piles in continuous grid – base” system before and after supplying the slab under the caps

The technical condition of the pile foundations of the building was classified as unsatisfactory. A project of the pile foundations strengthening was developed. The strengthening meant the underlying of L-shaped monolithic reinforced concrete beams under the existing grids. The beams were joined with transverse reinforced concrete beams, and from the top – with the cast-in-situ 200 mm thick slab (Figure 6).

![Spatial model of the pile foundation underpinning.](image)

The reinforcement consisted of the supply under the grid of monolithic reinforced concrete L-shaped beams, which were joined with transverse reinforced concrete beams, and on top - with a cast-in-situ slab with a thickness of 200 mm.

To include the plate “in operation” right after it was set there was provided soil-base compaction under the plate by rubble and its ramming by vibrating plates.

When designing the reinforcement of the foundations of the building, the decisive criterion was the assessment of additional settling and its unevenness, which are realized during the works. They were taken into account depending on the category of the building according to the technical state, based on the conditions:

\[ S_d \leq S_{d,u} ; \]  
\[ (\Delta S/L)_d \leq (\Delta S/L)_{u} ; \]  
\[ i_d \leq i_{d,u} , \]

where \( S_d \) – designed additional settling of a building in reconstruction, cm; \( S_{d,u} \) – limit value of the additional settling of the building, cm; \( (\Delta S/L)_d \) – calculated distortion of adjacent foundations after reconstruction; \( (\Delta S/L)_{u} \) – the limit value of the distortion in the interval \( L \); \( i_d \) – designed additional heeling of a building; \( i_{d,u} \) – limit additional heeling of a building.

To confirm the effectiveness of the solutions made, FEM modeling was proposed. The calculation in the spatial scheme, taking into account the joint work of aboveground and underground structures, the pile foundation and soil base under it was performed. When assessing the SSS of the building, the soil base of the foundations was replaced by appropriate coefficients.

In the first stage, we could simulate the service period, taking into account the different vertical stiffness of the piles due to the reasons described above, as well as emergencies with water supply SSS to the ground part similar to that recorded during the survey.
To do this, the changed stiffness of the piles was taken into account as in Figure 7. The results of the simulation of the working period are shown in Figure 8.

**Figure 7.** Initial (top) and after the soil wetting (bottom) pile stiffness accounted for in simulation.

**Figure 8.** Map of the normal stresses distribution in the masonry of the outer walls (left) and cracks in the wall according to the results of supervision (right) after flooding the foundations.
According to the nature of the stress distribution, it can be concluded that due to flooding as a result of the accident on the pipeline, the left and right parts of the building collapsed around the stairwell, which due to the nature of the piles is more rigid. Therefore, most cracks are concentrated around the stairwell. At the third stage of the simulation the work of the elements of foundation reinforcement is taken into account (Figure 9). The stress distribution maps show that the reinforcement significantly helped to remove the uneven nature of the stress distribution and bring it closer to the initial state. The peculiarity of the distribution of tensile stresses in the masonry around the stairwell is preserved, but their values either do not exceed the tensile strength of the masonry ($f_{bk2} = 0.11$ MPa), or exceed it in places similar to the initial state of the building.

Figure 9. Map of the distribution of normal stresses in the masonry of external walls after strengthening the foundations.

3.3. Practical implementation of foundation underpinning

The underpinning operations were performed in six stages (Figure 10):

- dig out of trenches and concreting the transverse stiffening beams between the grids of the longitudinal load-bearing walls and under the grids of the transverse walls of the stairwells;
- underlying the L-shaped beams under the grids at the intersection of the longitudinal and transverse walls between the axes "2" and "1";
- the remaining L-shaped beams were underplayed under the grids in the intersection of longitudinal and transverse walls between the axes "2" and "1";
- underlying the L-shaped beams under the grids at the intersection of the longitudinal and transverse walls between the axes "1" and "2";
- underlying the L-shaped beams under the openings remaining under the grids in the intersections of longitudinal and transverse walls (between axes «1» and «2»);
- reinforcement and concreting of the upper monolithic plate, which unites all reinforcement beams into a continuous rigid spatial structure.

Figure 10. Building foundation underpinning: a – setting up of monolithic concrete reinforcement beams; b – reinforcing plate set up; c – basement view after the operations performance.
A ribbed reinforcement plate is obtained, which is based on Strata-2с (alluvial sand, fine, medium density). Its ribs are directed to the bottom. This design effectively redistributes stress from uneven deformations of bases and has considerable rigidity at the minimum volume of groundworks, that means that virtually slab-and-pile foundations was arranged.

As a result of geotechnical monitoring after strengthening the foundations of the object, which has lasted for more than 3.5 years, it was found that: additional settling of the foundation of the building did not exceed 1 - 2 mm and has already stabilized; after the reinforcement was completed, some of the previously destroyed gypsum beacons were restored, but no new cracks were formed in them.

4. Conclusions
Thus, the design technological solution of strengthening of the prismatic pile base as a part of a continuous grid by supplying a monolithic reinforced concrete slab under the grid is improved. Its rather high efficiency and reliability are proved. Also, the calculation scheme of the system “strained structure - prismatic piles in the continuous grid – the base with a weak underlying layer” before and after the slab supply under the grid and there was performed mathematical modeling using FEM and elastic-plastic model of soil SSS of this system to assess the features of combined action of this system's components.

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