Geotechnical monitoring of a residential building being erected on strip-shell foundations in "Aquarel" housing complex in Tyumen

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Abstract. The actual issues of expanding the field of application of foundations for high-rise buildings are considered. Traditional foundations are not always effective on construction sites composed of normally and strongly compressible soils; thus, in certain conditions thin-walled strip-shell foundations (TSSF) can be used for cost reduction of the zero cycle. The paper describes the geotechnical monitoring program for GP1.1 built on TSSF - an alternative type of foundation as compared to traditional shallow foundations. The paper presents theoretical modelling of interaction of soil with the building; it also describes the technology of its manufacture using standard materials of STO SRO 001-2015 - "Design requirements for strip-shell foundations". The modelling results have shown the efficiency of foundations with a curvilinear convex upwards contact surface. The increased stiffness of the soil bed is caused by additional side squeezing of soil due to the specific features of the contact surface shape. The paper discusses the efficiency of TSSFs as to material consumption and cost as compared to pile-slab foundations. The paper gives the comparative data of geotechnical monitoring and approximate analysis.

1. Introduction
At present, the tendency to use the systems with controlled properties is appearing in geotechnical engineering [1-6]. Thin-walled strip-shell foundations (TSSF) are an example [7, 8]. TSSFs make it possible to form the given stress in the soil bed under strips and shells during the design phase and improve soil behavior under loading [9-11]. The curvilinear shape of the loaded surface of TSSF affects the characteristics of soil deformability by additional horizontal squeezing of soil in the upper zone of the bed resulting in stiffness increase [12, 13].

TSSF consists of the elements differing in stiffness and peculiar features of behavior (Fig. 1). The first one - strip foundation – is a supporting structure for bearing walls or columns possessing a certain stiffness in the longitudinal direction, taking random eccentricities of load transfer [14-16] and forming the desired cantilever widening on the outer contour of the building. The second one is a flexible element in the span joining the nearest supporting strips as a slab foundation. It functions as a transmitter of the certain (given) load from the building on the soil bed in the span. The convex upwards shell which is good in tension is considered to be the most appropriate flexible connection between strip foundations. The shell may actively influence the soil "response" due to such parameters
as: width ratio of the strip and span, shell height, tensile stiffness, flexural stiffness, fixing in the strip foundation (flap hinge, embedding), function curvature of the shell.

\[ \begin{align*}
N_1 & \quad R & \quad b_1 \\
N_2 & \quad R & \quad b_2 \\
p_1 & \quad p_2 & \quad p_3 \\
p_1 & \quad p_2 & \quad p_3
\end{align*} \]

**Figure 1.** Strip foundations with flexible connection.

Structurally, the convex upwards shell is good in central tension as energetically the most favorable state of stress [17, 18] and possibility to form almost equal-stress state in length; technologically, it is good because of small thickness, single-layer reinforcement and low complexity of work.

The economic advantages, as compared to slab foundations, are as follows: significantly smaller amount of concrete, slightly smaller amount of reinforcement and hence, lower cost.

The building for geotechnical monitoring - GP 1.1 is located in “Aquarel” housing complex in Tyumen. The 22-storeyed monolithic frame building with dimensions of the main axes to be 25.75 m x 26.0 m in plan is a space-planning solution for GP 1.1. The structural diagram of the building is a frame. The monolithic reinforced concrete frame is made of concrete of class B25 in strength. Spatial rigidity and stability of the building is provided by the joint performance of the monolithic stiffening diaphragms (the walls of the staircase-elevator unit) with horizontal stiffness discs, monolithic slabs and columns of the building strictly interconnected with foundations and slabs.

Engineering-geotechnical elements XV-XV (Figure 3) are shown in Table 1. The laboratory data and engineering-geological section indicate the complexity of the geological structure; that is necessary to be considered in designing.

**Figure 2.** General schematic view.

**Figure 3.** Schematic view of drill holes location.
Table 1. Engineering-geological conditions of the construction site.

| Description of soil | Depth $H$, m | $\gamma_I$, kN/m$^3$ | $\phi_{II}$, degrees | $C_{II}$, kPa | $E$, MPa |
|---------------------|-------------|----------------------|---------------------|--------------|----------|
| 1. Semi-solid-solid loam with clay and sand layers | 0-1.7 | 19.3 | 21.6 | 24.3 | 22.58 |
| 2. Stiff loam with sand layers | 1.7-4.9 | 17.31 | 16.1 | 26.4 | 11.32 |
| 3. Semi-solid-solid clay with sand layers | 4.9-7.7 | 17.31 | 19.4 | 63.1 | 22.29 |
| 4. Sandy, medium-density, water-saturated silt with loam layers | 7.7-14.8 | 19.32 | 29.3 | 4.4 | 12.56 |
| 5. Semi-solid-solid clay with sand layers | 14.8-23.5 | 17.31 | 19.4 | 63.1 | 16.34 |

Groundwater is considered to be glacial-lacustrine deposits of the IV terrace above the flood-plain. The depth of groundwater occurrence is 3.9-5.2 m. The depth of the standing-groundwater level is 3.2-4.8 m.

The standard depth of the seasonal freezing of soils calculated after [19] is 1.8 m.

2. Methods

The analysis was carried out in SCAD software programme using the contact model based on Winkler's hypothesis; the 3D analysis took into account all the major elements of substructure and superstructure [20]. The analysis was performed according to the algorithm presented in STO SRO 001-2015 "Design requirements for strip-shell foundations" [21].

The average settlement of TSSF was analyzed according to [21] and Building Regulations 22.13330 [22] using the total calculated deformation modulus.

The total calculated deformation modulus $E_j^*$ needed to find $k_d$ (see. p. 6 of the Algorithm [21]) was determined in PLAXIS 2D AE geotechnical software programme; the 2D analysis was done on the basis of numerical implementation of the problem of soil-shell interaction, the soil being simulated with elastic half-space [23-25].

In order to determine $k_1$ (see. p. 6 of the Algorithm [21]), the layering summation method dividing shear and bulk soil deformations [23] was taken for analysis. Poisson's ratios for soils within the depth of the compressible strata were taken in accordance with the Building Regulations [19] due to lack of the source data. Limits of the depth of the compressible strata were also taken according to the Building Regulations [19]. The total specified characteristic load was 230 kPa.

Implementation of the analysis algorithm [18] for interaction of the foundations under study with the soil bed is given further.

1. The foundation depth is taken according to the architectural solutions, taking into account the requirements of p. 5.5 [22], equal to 2.1 m from the level of the design elevation.

2. Preliminary determination of TSSF dimensions (including consoles) is done. The outline of the shell is compatible to a square parabola and criterion of flatness (Figure 4). As a result, the following TSSF active design parameters are obtained: $f = 0.75$ m, $EA = 4.6 \cdot 10^6$ kN/m, $EI = \text{const} = 17050.3$ kN·m.
3. Let us determine the depth of the compressible strata \( H_{\text{comp}} \) according p.5.6.41 [22]. Given the total specified characteristic load \( H_{\text{comp}}^{230} = 18.2 \) m.

Let us evaluate the initial coefficient of soil bed reaction \( k_i \), by the formula:

\[
k_i = \frac{p_{\text{aver}}}{s_{\text{aver}}}
\]

where \( p_{\text{aver}} \) – average pressure under the TSSF footing, kPa; \( s_{\text{aver}} \) – average settlement of TSSF bed, m, determined according to p.5.6.31 [22].

Thus, \( k_i^{230} = 1.084 \text{MN/m}^3 \)

4. Let us simulate the foundation and the building superstructure frame in 3D and perform the static analysis of the bed-foundation-superstructure system in the total specified characteristic load with the specified number of floors for adequate consideration of the superstructure stiffness. Let us determine the settlements \( s_i \), m, and contact pressures \( p_i \), kPa under the individual elements of TSSF in normal loading. The average settlement values are \( s_{\text{aver}}^{230} = 137.0 \text{ mm} \). The reactive pressure diagram in normal loading is shown in Figure 5.

5. Let us check the contact pressure values under the TSSF strips in accordance with the condition (2) in the final stage of loading:

\[
p_{sf} = (0.5 + 0.9)R
\]

where \( R \) – calculated resistance of soil under the strip foundation, kPa, determined in accordance with p.5.6.7 [22], taking into account the counterweight from pressure of the neighboring shells for average strip foundations and the counterweight from the shell and backfill soil for edge strip foundations.

In non-compliance with the condition (2), the geometric dimensions of the strips and shells in plan and the design parameters are adjusted: shell depth \( f \), shell tensile stiffness \( EA \), its bending stiffness \( EI \) and the system is reanalyzed.

Thus, in the last stage of loading \( p_{sf,\text{aver}} = 0.3R_{sf,\text{aver}} \), \( p_{sf,\text{edge}} = 0.61R_{sf,\text{edge}} \).
Changes in the geometry of the system are not preformed.

6. Let us integrate the static analysis and evaluate the corrected values of the coefficients of soil bed reaction \( MN/m^3 \), under the separate TSSF elements by the formula:

\[
k_{i+1} = \frac{p_i}{S_i}
\]

(3)
where \( p_i - \) pressure under the separate zones of loading – TSSF elements in the first approximation, kPa; \( S_i - \) average settlements under the separate zones of loading – TSSF elements in the first approximation, m.

If the difference between \( k_i \) and \( k_{i+1} \) exceeds 10\%, the system is reanalyzed with the corrected values of the coefficients of soil bed reaction – the second approximation, etc. \( n \) times up to conformance with pressures \( p_n \) arising from the defined coefficients of soil bed reaction \( k_n \) and settlements \( s_n \) under the separate zones of TSSF loading.

7. Let us perform the static analysis taking into account the deformable characteristics of soil bed loaded along the curvilinear convex surface in normal loading.

The calculated values of the coefficient of soil bed reaction, \( k_n^* \), MN/m\(^3\), are evaluated by the formula:

\[
k_n^* = k_n \cdot k_f
\]

(4)
where \( k_n \) – coefficients of soil bed reaction under the individual TSSF elements, MN/m\(^3\), determined by the iterative approach in the last approximation; \( k_f \) – coefficient considering soil bed stiffness increase under the shells of TSSF due to the curvilinear form of loading is evaluated by the formula:

\[
k_f = 1 + \frac{k_d A_{shell}}{A \cdot k_1}
\]

(5)
where \( k_1 \) – the settlement ratio of soil bed layers from additional vertical stresses \( \sigma_z \) as a result of soil bed loading by the entire area of TSSF and a separate shell within the limits of the \( j \)-layer of soil; \( k_d \) – coefficient taking into account compressibility decrease of the soil bed loaded along the curvilinear surface with respect to loading along the flat surface:

\[
k_d = 1 - \frac{S_{shell}}{S_{flat}}
\]

(6)
where \( S_{shell} \) – soil bed settlement along the shell axis, loaded along the curvilinear surface, with the distribution diagram of the contact pressures specified from the approximate analysis considering \( E_j^* \); \( S_{flat} \) – soil bed settlement along the load axis on the flat surface, with a uniform distribution diagram of the contact pressures considering \( E_j \).

Thus, the coefficient of the form for normal loading is \( k_f^{230} = 1.192 \)

8. Let us check the conditions:

\[
S_n < [S]
\]

(7)
\[
\frac{\Delta S_n}{L} < \left[ \frac{\Delta S}{L} \right]
\]

(8)
where \( s_{nt} \) – average settlements under separate zones of loading – TSSF elements in the last approximation, m; [\( S \)] – ultimate limit settlement, m, after [22]; \( \Delta S_n \) – difference in settlements of the neighbouring strip foundations, m; \( L \) – distance between the axis of the principal strip foundations, m;

According to the analysis done after the geomechanics model using \( E_j^* \), the settlement is 135 mm, after the contact model using \( k_n^* \) the settlement is 124 mm, which is less than the ultimate limit value – [\( s \)] = 150mm.

Unevenness of settlement is almost 1.6 times less as compared to the standard settlement – 0.0018 < [\( \Delta s/L \)] = 0.003, \( L \) – distance between the axis of the principal strip foundations, m.

It should also be noted that the standard analysis for settlements by the method of summation by layers [22], using the total calculated deformation modulus \( E \) expects the settlement to be within 21.2 cm; this exceeds the ultimate limit values and makes it difficult to use shallow foundations in these conditions.

9. The checking analysis is required for the soil bed under strip foundations in order to prove the validity of using the theory of linear deformability of soils in normal loading in accordance with (9), taking into account the last iteration considering soil bed stiffness increase under the shells.

\[
p_{st} \leq R
\]  

where \( p_{st} \) – average pressure under strip foundations of TSSF, kPa, considering reanalysis; \( R \) – the same as in formula (2).

Due to some decrease in pressure under the strip foundations, the analysis can be ignored and taken as the final results presented in p. 5.

10. Let us perform the analysis of the edge uncompensated strip foundations for possible torsion. The analysis is made for displacement of the resulting load vector from the section gravitational center in order to eliminate torque, taking into account the whole of forces acting on strip foundations [14-16]. The side reaction of the natural bed at the contact zone with the side surface of the strip foundation (Figure 6):

\[
q_L = q_{vert} \cdot \xi = 250 \cdot 0.54 = 135 \text{ kPa}
\]

\[
\xi = \frac{v}{1-v} = \frac{0.35}{1-0.35} = 0.54 \quad \text{coefficient of side soil pressure; } q_{vert} = 250 \text{ kPa} \quad \text{vertical pressure under the shell at the contact point with strip}
\]

Force due to friction of soil along the strip foundation footing:

\[
\tau = \frac{Q_{soil}}{b} \cdot \tan \phi + c = \frac{1201.5}{2.67} \cdot \tan 16.1^\circ + 26.4 = 156.3 \text{ kPa}
\]

where running reactive soil pressure under the strip foundation:

\[
Q_{soil} = p_1 \cdot b = 450 \cdot 2.67 = 1201.5 \text{ kN/m}
\]

Longitudinal stress in the shell:

Figure 6. Design diagram for the edge strip foundation of 22-storeyed building.
The resultant of the active pressure: \( E_a = 16 \text{ kN/m} \).

If the value of eccentricity is \( e_1 = 0.38 \text{ m} \), \( h_e = 0.52 \text{ m} \), the torsional load is \( \sum M_0 \approx 0 \). The maximum deflection of the edge strip foundation in maximum distance of the transverse edges – \( l = 5.8 \text{ m} \) is 0.15 cm, which is considerably less than the limit value - \( [1 / 150 l] = 3.86 \text{ cm} \).

Strip foundations are constructed in the zones of load-bearing walls and columns and take the load from the building. The shells are formed in the span between the loaded zones and represent gently sloping cylindrical convex upwards reinforced shells (Figure 7).

The thickness of the strip foundations is taken to be 1200 mm, and the thickness of shells - 200 mm. The shells are reinforced with a single layer of A500C reinforcement, 20 and 25 mm in diameter. The strip foundations are reinforced with A500C - 20, 22 and 25 mm in diameter, cross reinforcement - A240, 8 mm in diameter. Heavy concrete - B30, F200 and W8 was taken for concrete works.

The foundation for GP-1.1 was carried out through line system. Construction of each section was conditionally divided into 5 phases: ground excavation (Figure 8), reinforcing and timbering (Figure 9), concrete works (Figure 10).

**Figure 7.** Schematic illustration of the foundation.
Table 2 shows materials consumption and the estimated cost for raw version of GP-1.1 erected on pile-slab foundation with driven piles - 300x300 mm, 12 m in length, incorporated with a slab - 1m in thickness and TSSF.

| Kinds of works                        | GP-1.1 | Pile-slab | TSSF |
|---------------------------------------|--------|-----------|------|
| Ground excavation, m³                | 3200   | 2740      |      |
| Fine excavation, m³                   | 97     | 191.1     |      |
| Driven piles, 1 m³ of piles          | 348.8  | -         |      |
| “Cutting” of pile heads, 1 pile      | 320    | -         |      |
| Crushed stone bedding, m³            | 137    | 64.3      |      |
| Concrete bedding, m³                  | 86.7   | 43.3      |      |
| Heavy concrete, m³                    | 841    | 581.1     |      |
| Reinforcement, kg                     | 57720  | 74501     |      |
| Estimated total labour costs, man/hours | 5550  | 3448      |      |
| Estimated cost in prices of the 3d Q, 2013, thous.rub. | 15154  | 7674      |      |

3. Results

The settlement points (Figure 11) were installed on perimeter in the base course of the building [27]. The stationary part of the settlement points is rigidly fixed in the base course, and the inventory part with a truncated spherical head makes it possible to install the leveling board at the middle of its toe when re-leveling strictly on the same point. Monitoring was carried out with a high-precision level - H-0.3 using a barcode leveling board, thus the accuracy of measurements (0.5 mm) was ensured at the given facility [28].

Monitoring was being carried out from commencement of construction. The foundation was concreted in October, 2014. In December 2015, construction of GP 1.1 was approximately 90% complete based on 100% of the specified characteristic load - \( P_{\text{aver}} = 230 \) kPa (Figure 12). According to the developed algorithm (p.2) the calculated value of the final settlement is within 12.5-14.0 cm for the considered facility, in ultimate limit values - 15cm [22].

In the middle of August 2016, the average settlement was 83.5 mm, the maximum one – 97.9 mm, the minimum one - 64.3 mm. The maximum settlement unevenness was \( \Delta s/L = 0.00093 \). The diagram
of settlement points located on perimeter of the building and diagram of settlements are shown in Figure 13.

![Installation of a leveling board on the settlement point](image1)

![General view of the residential unit – GP 1.1 under construction](image2)

**Figure 11.** Installation of a leveling board on the settlement point.  
**Figure 12.** General view of the residential unit – GP 1.1 under construction.

**Figure 13.** Diagram of settlements developing in time for the residential unit – GP 1.1.

Taking into account the results of monitoring, soil formations in the bed of the given construction site, it is needed to predict the final settlement value within 11-13cm, i.e. up to 10-15% in accordance with the calculated data [29-31].

4. Conclusion
The modeling results have shown the efficiency of foundations with a curvilinear convex upwards contact surface. The increased stiffness of the soil bed is caused by additional side squeezing of soil due to the specific features of the contact surface shape.

Reasonable use of effective types of shallow foundations will reduce the basic normalized the design parameter - settlement, up to several tens of percent, increasing the carrying capacity of the ground base, while reducing the consumption of materials, namely steel and concrete by 20-50%, with respect to the traditional, such as slab or pile-slab foundations.
References

[1] Goncharov B V, Galimnurova O V, Gareeva N B and Bashlykov A V 2011 Estimation of bearing capacity and settlement of shell foundations on tamped beds Soil Mechanics and Foundation Engineering Vol 48 No 2 pp 62-66 (rus)

[2] Ter-Martirosyan Z G, Pronozin Ya A and Stepanov M A 2012 Feasibility of pile-shell foundations with prestressed soil beds. Soil Mechanics and Foundation Engineering Pp 1-5 (rus)

[3] Samokhvalov M, Zazulya J, Melnikov R and Mironov V 2016 Design Calculation of Drill-Injection Piles with Controlled Broadening and Silty-Clayed Soil Foundation Basic Interaction Parameters MATEC Web of Conferences 73 2016 01009 (rus)

[4] Kiselev N, Pronozin Y, Stepanov M, Bartolomey L and Keck D 2016 Theoretical and Experimental Substantiation for Applicability of a Damping Layer in a Foundation Slab Placed on Soil Bed MATEC Web of Conferences 73 2016 01017 (rus)

[5] Ikonin S V and Suhoterin AV 2015 Konstruksia fundamentnoy plity c reguliruemymi svoistvami [The design of the foundation plate with adjustable effort] Magazine of Civil Engineering No 3 (55) pp 10-20 (rus)

[6] Goncharov Yu M 1990 Experience with construction and operation of residential buildings on shell-foundations in Igarka Soil Mechanics and Foundation Engineering 27 No 3 pp 91-96 (rus)

[7] Goncharov B V and Rybakov A V 2001 Shell foundations on tamped soil beds Soil Mechanics and Foundation Engineering 3 pp 167-171 (rus)

[8] Luo Y F and Teng J G 1998 Stability analysis of shells of revolution on nonlinear elastic foundations Computers & Structures 69 No 4 pp 499-511

[9] Hong T, Teng J G and Luo Y.F 1999 Axisymmetric shells and plates on tensionless elastic foundations International Journal of Solids and Structures 36 No 34 pp 5277-5300

[10] Paliwal D N, Pandey R K and Nath T 1996 Free vibrations of circular cylindrical shell on Winkler and Pasternak foundations International Journal of Pressure Vessels and Piping 69 No 1 pp 79-89

[11] Kolesnikov A G 2015 Issledovanie napryazhenno-deformirovannogo sostoyaniya polohih obolochek na uprugom osnovanii [Investigation of stress-strain state of shallow shells on elastic foundation] Science and Peace vol I pp 83-85 (rus)

[12] Abdel-Rahman M 1996 Geotechnical behaviour of shell foundations Ph. D Thesis, Department of Civil Engineering, Concordia University (Montreal, Canada)

[13] Bogomolov A N and Ushakov A N 2013 Stress-strain state of an elastic half plane under a system of inclined piecewise-linear loads Soil Mechanics and Foundation Engineering 50 No 2 pp 43-49 (rus)

[14] Bogomolov A N, Ushakov A N and Bogomolova O A 2014 Stress Distribution in the Bed of an Inclined Absolutely Rigid Plate with Consideration of Friction Along the “Plate-Soil” Contact Soil Mechanics and Foundation Engineering 51 No 4 pp 165-172 (rus)

[15] Ter-Martirosyan Z G, Pronozin Y A and Kiselev N Y 2014 Shallow Strip Foundations Joined by Gently Inclining Envelopes on Highly Compressible Soils Soil Mechanics and Foundation Engineering 51 No 4 pp 157-164 (rus)

[16] SNiP 2.02.01–83* Osnovaniya zdaniy i sooruzheniy [Soil bases of buildings and structures] (Moscow: Stroyizdat) 1995 (rus)

[17] Utkin V S 2016 Raschet nadezhnosti gruntovyh osnovaniy fundamentov zdaniy I sooruzheniy po kriteriu deformasii pri ogranicchennoy informatsii o nagruzkah I gruntah [Calculation of the reliability of soil basement foundations of building and structures by deformation criterion with limited information about the load and soil] Magazine of Civil Engineering pp 4-13 (rus)
[18] Standart organizasii STO SROP 001-2015 Trebovaniya k proektirovaniyu I ustroistvu lentochno-obolechechnykh fundamentah [Requirements for the design and construction of strip-shell foundations] (Tyumen) p 43 (rus)

[19] SP 22.13330.2011 Osnovaniya zdaniy i sooruzheniy. Aktualizirovannaya redaksia SNiP 2.02.01-83* [Building Regulations 22.13330.2011 Foundations of buildings and structures. Updated version of SNiP 2.02.01-83*] (Moscow: OAO "SPP" 2011 p 161 (rus)

[20] Goncharov B V, Gareeva N B, Galimnurova O V and Bashlykov A V 2010 O raschete fundamentov-obolechek na vytrambovannom gruntovom osnovanii [About calculation of shell foundations on compressed building base] Proceedings of the Kazan State Architectural University No 2 (14) pp 143-148 (rus)

[21] Bashlykov A V, Bogomolov A N and Goncharov B V 2007 Issledovanie raboty fundamentov-obolechek na vytrambovannom osnovanii I metod oseki nesushey sposobnosti [Research of the shell-foundations work on compressed building base and method for estimating the bearing capacity] Herald of Volgograd State University of Architecture and Civil Engineering. Series: Construction and architecture No 8 pp 18-21 (rus)

[22] Emelyanov I G and Kuznetsov V Y 2002 Oprdelenie uprugoplasticheskogo napryazhennogo sostoyania kontaktiruyushe s osnovaniem silindricheskoy obolochki [Determination of elastic-plastic stress state in contact with the base of the cylindrical shell] Herald of Russian Peoples Friendship University. Series: Engineering studies No 1 pp 42-46 (rus)

[23] Ukhov S B, Semenov V V, Znamenskiy V V and Ter-Martirosyan Z G etc. 2005 Mechanika gruntov, osnovaniya I fundamenty [Soil mechanics, foundation and Basement: Textbooks for high schools building specialties] (Moscow: ASV Publishing) p 528 (rus)

[24] Bogomolov A N and Ushakov A N 2013 Stress-strain state of an elastic half plane under a system of inclined piecewise-linear loads Soil Mechanics and Foundation Engineering 50 No 2 pp 43-49 (rus)

[25] Bogomolov A N, Ushakov A N and Bogomolova O A 2014 Stress distribution in the bed of an inclined absolutely rigid plate withconsideration of friction along the plate-soil contact Soil Mechanics and Foundation Engineering 51 No 4 pp 165-172 (rus)

[26] Ter-Martirosyan Z G, Pronozin Y A and Kiselev N Y 2014 Shallow strip foundations joined by gently inclining envelopes on highly compressible soils Soil Mechanics and Foundation Engineering 51 No 4 pp 157-164 (rus)

[27] Ter-Martirosyan Z G, Telchenko V I and Korolev M V 2006 Problemy mechaniki gruntov, osnovaniy I fundamentov pri stroitelstve mnogofunktionalnykh vysotnyh zdaniy I kompleksovv [Problems of soil mechanics, bases and foundations in the construction of multifunctional high-rise buildings and complexes] Herald of MGSU No 1 pp 18-27 (rus)

[28] Yun H-B and Reddi L N 2011 Nonparametric Monitoring for Geotechnical Structures Subject to Long-Term Environmental Change Advances in Civil Engineering 275270

[29] Pronozin Y A, Naumkina U V and Epifantseva L R 2015 Lentochnye fundamenty, obedinennye poloigimi silindricheskimi obolochkami, dlua zdaniy povyshenno etazhnosty [Strip foundation, combined shallow cylindrical shells for high-rise buildings] Industrial and civil construction No 12 pp 58-62 (rus)

[30] Huat B B K and Mohammed T A 2006 Finite Element Study Using FE Code (PLAXIS) on the Geotechnical Behaviour of Shell Footings Journal of Computer Science vol 2 No 1 pp 104-108

[31] Rinaldi R 2012 Inverted Shell Foundation Performance In Soil A Thesis in the Department of Building Civil & Environmental Engineering (Montreal: Concordia University)