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ANALYSIS OF THE NON-LINEAR METHODS FOR FOUNDATION SETTLEMENT ANESSMENT

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Abstract

The article discusses methods for calculating foundation settlement based on various approaches to solutions of the theory of limit equilibrium. The analysis of foundation settlement calculations in the linear and nonlinear stages of the subsoil reaction according to the Belarusian and Russian regulatory documents. It has been found that with a critical load equal to $0{,}85P_u$, the calculated draft most fully corresponds to the experimental one. Thus, when calculating settlement beyond the linearity limit, it is recommended to take into account not the limit load, but 0,85 of its value.

Keywords: foundations, soils, foundation settlement, nonlinear calculation methods, foundation depth.

АНАЛИЗ РАСЧЕТА ОСАДОК ФУНДАМЕНТОВ НЕЛИНЕЙНЫМИ МЕТОДАМИ

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Реферат

В статье рассмотрены методы расчета осадок фундаментов, основанные на различных подходах к решениям теории предельного равновесия. Проведен анализ расчетов осадок фундаментов в линейной и нелинейной стадии работы грунтового основания по действующим в Республике Беларусь и Российской Федерации нормативным документам. Установлено, что при критической нагрузке равной $0{,}85P_u$, расчетная осадка наиболее полно соответствует экспериментальной. Таким образом, при расчете осадок за пределом линейности рекомендуется принимать во внимание не предельную нагрузку, а 0,85 от ее значения.

Ключевые слова: фундаменты, грунты, осадки фундаментов, нелинейные методы расчета, глубина заложения.

Introduction

The methods used in practice for determining the foundation settlement, in the linear stage of the use of the foundation soil, are considered quite reliable today, because they guarantee the overall normal facilities management without reducing their durability. However, this does not mean that they do not contain some contradictions that reduce the reliability of the obtained results, and in this regard, they continue to be discussed and improved today.

The assessment of calculation methods

In accordance with [1] in the Republic of Belarus, the following methods for calculating the final absolute compaction settlement of the foundation should be used in the engineering:
- the method of layer-by-layer summation using the calculation scheme of a linearly deformed half-space;
- the method of a linearly deformed layer of finite thickness;
- the equivalent layer method.

At the same time, the scope of each of the above methods is clearly limited, since the total final settlement S of the foundation depends on a number of factors, such as: stresses in the foundation; the distributing ability of the foundation soil; the contact plane of the foundation and the soil conditions; the size of the foundation, its shape, stiffness and depth; the structure and texture of the soil; rate of loading and it's condition; manufacturing environment, etc. and all of them to a certain extent have an impact on the value of S. Nevertheless, the actual values of the foundation deformations in many cases are significantly less than the theoretically estimated settlement values. That is why, according to the current regulations of the Russian Federation [2, 3], the calculated soil resistance R, that limits the linear relationship between the pressure along the base of the foundation P and its settlement S, is recommended to be increased to R_n with the ratio of the calculated settlement of the base S (pressure $P = R$) and the maximum settlement S_u:
- if $S \le 0{,}4S_u$ $\quad R_n = 1{,}2R$;
- if $S \ge 0{,}7S_u$ $\quad R_n = R$;
- if $0{,}7S_u > S > 0{,}4S_u$ $\quad R_n$ is determined by interpolation.

With appropriate justification, it is allowed to take $S < 0{,}4S_u$, and, accordingly, $R_n = 1{,}3R$.

The specified increase in pressure should not cause an increase in deformations of the foundation over 80% of the limit and exceed the bearing test ultimate pressure value.

Obviously, this approach provides a more economical solution of foundation structures in the building design. According to the regulations [4] of the Republic of Belarus, the calculation of the foundations is based on any known models of foundations (linear and nonlinear) and methods (direct, indirect, empirical), including simplified ones that guarantee, with the necessary reliability, protection against the leading to destruction or ultimate limit states for foundations and safety requirements; ensure suitability for normal operation, durability and economic feasibility of the decisions taken.

The use of nonlinear methods for calculating the settlement of the foundations, which is actually a move beyond of the proportionality $S = f(P)$, i.e. in the region $R < P < P_u$ (P_u is the ultimate or maximum pressure on the foundation) clearly indicates an increase in the efficiency of the foundations design solution, but their reliability, normal operation and durability must be ensured.

The analysis of approaches to calculating the foundation settlement by nonlinear methods allows us to note that in almost any proposed method, it is necessary to know R and P_u to determine the final settlement.

As for R, for its determination the necessary coefficients are introduced into the formula: the operating conditions of the soil base γ_1, the building or structure service conditions within interaction with the foundation γ_2, the reliability k, which allow us to get a better reflect of the distribution capacity of the foundation soils, the influence of the structural features of the building and its three-dimensional rigidity on composite action with the foundation and the reliability of the calculated soil characteristics used. In addition, the calculated dependence also takes into account the depth of the foundation base, the average (by layers) calculated value of the specific weight of the soil lying above and below the base of foundation, the weighing effect of groundwater if the groundwater is located above the base of foundation, etc. All this contributed to a significant increase in R.

At the same time, the average pressure under the base of the foundation is limited by the value R according to [11] (linear phase of deformation), which means that the calculation of the zones of limit equilibrium is not taken into account, i.e. the calculation of settlement is based on the linearly deformable array model. Thus, the restriction $P \leq R$ indicates the absence of sufficiently developed calculation methods that take into account the presence of plastic zones in the foundation, i.e. methods based on the use of nonlinear models.

It should also be noted that there are some contradictions among the methods of calculating the settlement of foundations, in one form or another using the theory of a linearly deformable medium. For example, the calculation method proposed in [8], which allows taking into account the effect of horizontal stresses and approximately the stiffness of the foundation associated with deformations of compaction and shaping, is more accurate than the method of layer-by-layer summation, due to the fact that most of the foundation settlement (64 % or more), even at the initial stage of its loading, is associated with shear deformations of the soil.

The condition for achieving the maximum deformations of buildings is not fully used, since the determining factor is the inadmissibility of exceeding the pressure P of the value R or 1,2R. But if the value of R allows for the partial development of plastic deformation zones, then at a pressure P_1, these zones are only beginning to emerge under the edges of the foundation (Fig.1).

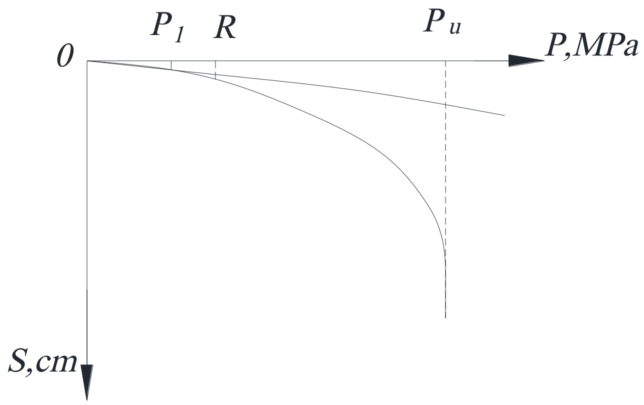

Figure 1 – Dependence of settlement S on pressure P

It is this load that is the critical initial, the value of which is determined by the formula of N. P. Puzyrevsky, N. M. Gersevanov [9, 10]. However, as M. N. Goldstein suggests [11], the methods do not take into account the actual distribution of stresses along the foundation base, at which their concentration occurs along its edges already at the lowest pressures, and, apparently, the limit equilibrium regions start to appear at these points, even when the pressure is far below than P_1.

As for the load capacity of the foundation, the most common methods are so-called rigorous methods, in which the sliding curve shape is found as a result of calculation (solutions of Sokolovsky V. V., Berezantsev V. G., Malyshev M. V., Golushkevich S. S., etc.) and methods in which the outlines of the sliding surface are given approximate, but quite well consistent with experience and theories (solutions of Berezantsev V. G., Terzaghi K., Meyergof G. G., TCP 45-5.01-67-2007).

Table 1 shows the results of calculations based on the methods for solving the theory of limit equilibrium to various degrees, performed by V. G. Berezantsev [5] and supplemented by the authors.

The calculations were performed for a non-buried foundation with a central load (plane problem case) using the values of a dimensionless coefficient equal to $P_u/\gamma b^2$ (γ – specific weight; b – width of the foundation), calculated by various methods for two values of the internal friction angle of the soil: φ = 30° and φ = 40°.

Table 1 – Values of dimensionless coefficients

The author of the method; a reference to a literary source; the formula given in [5]	$P_u/\gamma b^2$	
	φ = 30°	φ = 40°
Gorbunov-Posadov M. I., Mintskovsky M. I., formula (77)	6,1	62,3
Terzaghi K., formula (79)	20,0	50,0
Caquot and Kérisel, formula (80)	11,4	57,0
Meyerhof G. G. formula (79/)	12,5	60,0
Malyshev M. V., formula (75)	23,1	104,5
Zaharescu E., formula (78)	8,3	47,5
Gorbunov-Posadov M. I., [7]	-	95,5
Berezantsev V. G., formula (111)	10,8	50,1
TCP 45-5.01-254-2012 (02250)	12,39	66,01

Analysis of the calculation results shows that most of the methods, with the exception of the values obtained by M. I. Gorbunov-Posadov [7], M. V. Malyshev [6], and according to TCP 45-5.01-254-2012 [4], give solutions with a significant difference. $P_u/\gamma b^2$ at φ=30° variability is from 6,1 to 12,5, i.e. more than 2,0 times. In comparison with the solution of M. V. Malyshev, the coefficient increased almost 3,8 times. At φ=40°, the coefficient increases, and the difference is almost 2,2 times.

Thus, the smaller the value φ, the bigger coefficient $P_u / \gamma b^2$ variability is. The data obtained by M. I. Gorbunov-Posadov and M. V. Malyshev show that modern calculation methods P_u based on solving a mixed problem give results that provide a certain reserve of strength, but the issue of determining reliable values P_u requires detailed study.

While using nonlinear methods of calculating settlement there is a condition $R < P < P_u$. As shown by M. V. Malyshev [6], the load which is relevant to the plastic zone emergence under the edge of the foundation and its dimensions depend on the lateral pressure of the soil coefficient ξ_K. The value ξ_K = 1 corresponds to the maximum load. If ξ_K < 1,0 or ξ_K >1,0, then the load is lower and in the second case – significantly, and the strength characteristics of the ground play a decisive influence. They also revealed that the size of plastic zones depends on the depth of the foundation, the size of the foundation base, and the stiffness. If there is no loading, or if it is negligible, then zones of plastic deformations are formed even at low loads. At the same time, as noted by Elizarov S. A. and Malyshev M. V. [12], in the entire range of loads transmitted to the sandy foundation through a stiffened bandpass rugged stamp until the ultimate limited state is exhausted, the regions with the limiting state begin to emerge already at P > 0,25P_u and, despite their presence at the loading stage up to P = 0,5P_u, the dependence between the settlement and the load remains linear.

At the loading stage up to P = 0,7P_u, a new area was observed under the center of the stamp at a depth of about 0.6b (b is the width of the stamp). The reason for its occurrence is the presence of a compacted core under the stamp, at the top of which local destruction occurs [13].

The formed compacted core under the stamp digs into the foundation, and as a result the ground is destroyed along its entire surface. When loading P > 0,85P_u, the limiting state regions develop from under the edges of the foundation and merge at a depth, skirting both the elastic and plastic parts of the resilient core. With a further increase in the load, the base loses stability.

Thus, the destruction of the base occurs at loads exceeding (0,7…0,8)P_u.

The determination of settlement beyond the limit of the linear relationship between stresses and deformations $R < P < P_U$ based on the solution of M. V. Malyshev [14] is performed according to the formula:

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$$S_P = S_R\left[1+\frac{(P_u-R)(P-R)}{(R-\sigma_{zq,0})(P_u-P)}\right], \qquad (1)$$

where S_R – is the foundation settlement at a pressure of $P = R$ or $P = 1{,}2 \cdot R$;

$\sigma_{zq,0}$ – vertical stress from the soil own weight at the level of the foundation base;

P_u – the maximum resistance of the ground.

Therefore, if in (1) the value P_u is limited to 70...80% of the limit value, then the reliability of determining the value S_p increases due to the prevention of the foundation ground destruction, namely, limiting the appearance of plastic areas under the entire foundation and reducing the ground discharge to the sides from the edge of the stamp.

To a certain extent, the justification of this approach is confirmed by the studies of V. V. Lushnikov and A. S. Yardyakov [15], who came to the conclusion that the method of M. V. Malyshev allows us to obtain a fairly reliable forecast of settlement at pressures approximately corresponding to half of the interval $P_1 - P_u$, where P_1 is the critical initial load on the ground. In this case the accuracy in determining the values of settlement is estimated at 30-40%.

Figure 2 shows the graphs of the settlement of foundations on a sandy base, obtained from the results of calculations according to formula (1) and according to the test data of real foundations. Information about the soils and tested foundations is given in Table 2.

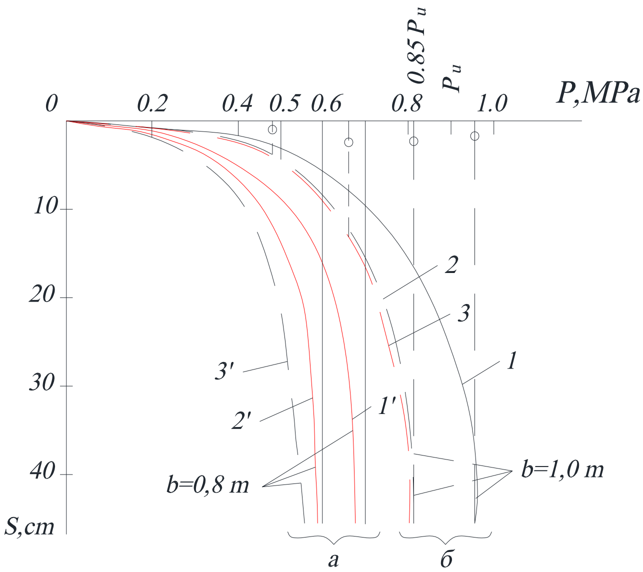
Table 2 – Value P_1, R, P_u and for various soils

Soil, dimensions of foundations	The value P_1, R, P_u, кPa and their ratio, values S_R, cm						
	P_1	R	S_R	R/P_1	P_u	P_u/P_1	P_u/R
1	2	3	4	5	6	7	8
Sand: E = 14,0 MPa; ν = 0,2; γ = 17,0 кN/m³; C = 2,0 кPa; φ = 30º; bxl = 0,8x4,8m; d = 1,5m	162	174	1,3	1,07	702	4,33	4,03
					597	3,69	3,43
Sand: E = 27,5 MPa; ν = 0,2; γ = 18,0 кN/m³; C = 1,0 кPa; φ = 32º; bxl = 1,0x4,8m; d = 1,5m	180	204	1,0	1,13	962	5,34	4,72
					818	4,54	4,01

The table and figure show that the calculated resistance differs by 14,7 %; P_1 – by 10 %, and the settlement obtained by layer-by-layer summation within the linear dependence $S = f(P)$ – by 23,1 %. The limiting pressures in medium-sized fractioned sand are 1,37 times higher than for small-sized fractioned sands and more than 4,0 times higher than the calculated soil resistance. If we take $P = 0{,}85P_u$, then this difference decreases from 3,4 to 4,0 times.

The graphs in Fig. 2 should be considered as linked to the actual loading of foundations with bxl =0,8x4,8 m and bxl = 1,0x4,8 m dimensions and a foundation depth d =1,5 m. Their analysis (curves 1, 2, 3) shows that the dependences $S = f(P)$ obtained from the test results and constructed according to the formula (1) at $P = 0{,}85P_u$, practically coincide within the entire range of operating pressures. The settlement determined for this case, at P_u = 962 kPa according to the formula (1), is considerably higher than the experimental ones. Approximately, up to P = 0,4 MPa, the experimental and calculated results are almost the same.

Moreover, the $S=f(P)$ relationship, while P in the pressure range from 0,0 to 0,4 MPa can be considered linear, since the settlement at P = 0,4 MPa is only 2,1 cm, which, however, is more than 2 times higher than S at $P = R$, but is only 26,3% of the maximum permissible settlement, assumed to be equal to 8,0 cm. And this indicates the presence of a major reserve in determining the size of the foundations in the plan. It should be noted that the pressure P = 0,4 MPa is 41,6 % of P_u.



a) with the width of the foundation b = 0,8 m;
b) with the width of the foundation b = 1,0 m;

1; 1' – according to the formula of M.V. Malyshev;
2; 2' – according to the formula of M.V. Malyshev, at $P = 0{,}85P_u$;
3; 3' – results of full-scale tests of foundations

Figure 2 – Graphs of the settlement of foundations on a sandy base

With an increase in P, settlement increases markedly in different ways: at $P = 0{,}5P_u$, the difference in settlement is 1,52 times; at $P = 0{,}7P_u$, this difference is already equal to 1,56; at $P = 0{,}85P_u$, it is more than 10,0 times. Curves 1', 2', 3' complies with the calculated data and test results of foundations with a width of 0,8 m on a sandy base.

The analysis of the curves $S=f(P)$ shows that the linear section here is shorter and can be taken at a pressure of P = 0,3 MPa. The difference in settlement for all three considered cases varies in the range of 2,53...3,92 cm, which is on average 40% of the maximum permissible settlement. The settlement at $P=R$ is 1,3 cm, which is equal to 51,4...33,2%. With an increase of the foundations external load, the settlement difference is intensifying. At $P = 0{,}5P_u$, the settlement determined by the formula (1) is 3,52 cm, and the settlement according to the test results is 5,98 cm.

The settlement determined by the formula (1), but at a limit pressure of $0{,}85P_u$ is equal to 4,28 cm, i.e. it is equal to the intermediate value between the S determined by the formula (1) and the results of full-scale tests. At $P = 0{,}7P_u$, respectively, S = 8,52 cm, S =24,56 cm, S = 13,54 cm, i.e., the growth of settlement at a pressure of $0{,}7P_u$, in comparison with the previous interval, was 2,42, 4,1 and 3,16 times, respectively. A similar character of settlement development persists at $P = 0.85P_u$.

This allows us to conclude that during the actual tests, the largest settlement increase is bigger than the foreseeable calculated one. If, according to the calculation, the maximum value of the settlement S is reached at $P = P_u$, then during the tests – at $P = 0{,}85P_u$, since the test results didn't reach the limit load.

Conclusion

1. The analysis of the existing methods for calculating the foundations settlement during the linear stage of the ground base work, regulated by the current normative documents, both in the Republic of Belarus and in the Russian Federation, shows the presence of a certain reserve, even if we allow an increase in the calculated resistance of the ground.
2. The use of nonlinear methods for the settlement calculating allows us to obtain fairly reliable results for determining settlement at pressures not exceeding 50% of the limit.
3. The results of the settlement calculations according to the method of M. V. Malyshev corresponded well with the experimental data, while the limiting pressure value being 85% of the maximum.
4. Theoretical solutions for the development or modernization of existing methods for calculating the settlement of foundations require clarification based on the experimental research data.

References

1. TCP 45-5.01-67-2007 (02250). Slab foundations. Design rules / Ministry of Architecture and Construction of the Republic of Belarus. – Minsk : Minarhstroy RB, 2008. – 138 p.
2. SP 50-101-2004. Design and construction of foundations and foundations of buildings and structures. – M., 2005. – 106 p.
3. SP 22.13330.2011. Foundations of buildings and structures (updated edition of SNiP 2.02.01-83). – M., 2011. – 262 p.
4. TCP 45-5.01-254-2012 (02250). Foundations and foundations of buildings and structures. Basic provisions. / Ministry of Architecture and Construction of the Republic of Belarus. – Minsk : Minarhstroy RB, 2012. – 104 p.
5. Berezantsev, V. G. Calculation of the foundations of structures / V. G. Berezantsev // Publishing house of literature on construction. – Leningrad, 1970. – 208 p.
6. Malyshev, M. V. The strength of soils and the stability of the foundations of structures. / M. V. Malyshev. – M. : Stroyizdat, 1980. – 137 p.
7. Gorbunov-Posadov, M. I. Stability of foundations on a sandy base / M. I. Gorbunov-Posadov. – M. : Gosstroyizdat, 1962. – 96 p.
8. Bronin, V. N. On the account of horizontal stresses at the base when determining the settlement of the foundation / V. N. Bronin, S. V. Tatarinov // Foundations, foundations and soil mechanics. – 1993. – Vol. 4. – P. 19–21.
9. Gersevanov, N. M. Collected works (T. 1 and 3) / N. M. Gersevanov. – M., 1958. – 516 p.
10. Puzyrevsky, N. P. The theory of tension in earthy soils / N. P. Puzyrevsky // Collection of works of LIPS. – 1929. – Vol. 99. –108 p.
11. Goldstein, M. N. Calculation of the settlement and strength of the foundations of buildings and structures / M. N. Goldstein, S. G. Kushner, M. N. Shevchenko. – Kiev : Budivelnik, 1977. – 208 p.
12. Elizarov, S. A. Carrying capacity criteria and different phases deformation of the base / S. A. Elizarov, M. V. Malyshev // Foundations, foundations and soil mechanics. – 1993. – Vol. 4. – P. 2–5.
13. Localization of deformations and destruction of sandy soil at the base of a strip stamp / S. A. Elizarov [et al.] // Interuniversity collection. – Yoshkar-Ola, 1989. – № 54. – P. 50–54.
14. Malyshev, M. V. Calculation of the settlement of foundations with a nonlinear relationship between stresses and deformations in soils / M. V. Malyshev, N. S. Nikitina // Foundations, foundations and soil mechanics. – 1982. – № 2. – P. 21–25.
15. Lushnikov, V. V. Analysis of sediment calculations in the nonlinear stage of soil work / V. V. Lushnikov, A. S. Yardyakov // Bulletin of the Perm National Research Polytechnic University. Construction and architecture. – 2014. – № 2 – P. 44–55.

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