DETERMINING QUASI-SINGLE-PHASE SOIL SYSTEM SETTLEMENTS TAKING INTO ACCOUNT THEIR CREEP

DOI: 10.31618/ESU.2413-9335.2020.3.74.745

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ABSTRACT

This article discusses determining foundation settlements taking into account the creep of quasi-single-phase and two-phase soil systems of the bases. There is presented distribution of clay soils in depth in the territory of Central Kazakhstan. The analysis of the stratification results shows that physical properties of clays are heterogeneous and have sufficiently large scattering of values in depth. It has been experimentally established that clay soils selected at different depths are characterized by shear decaying creep. Methods of determining the coefficients of consolidation and creep, the deformation modulus from experimental curves are described. The calculation formulas are given for determining quasi-single-phase and two-phase soil systems taking into account their creep. The calculation formulas obtained on the basis of the results of experimental data indicate the novelty of the method.

Keywords: quasi-single-phase, deformation modulus, creep, Eurocode, foundation, base, clay soils.

1. Introduction

The problem of predicting base settlements is reduced to solving complex engineering problems associated with determining experimental parameters (the deformation modulus, the consolidation and creep coefficients), and selecting a rheological model of clay soils.

A complete analysis of the base behavior during consolidation and creep of solid soil particles over time in the course of construction and operation is possible only if the results of experimental studies are reliable and accurate. In this article the authors have found that physical properties of clay soils of the construction site of Central Kazakhstan are characterized by heterogeneity and sufficiently large scattering of the characteristic values in depth. Moreover, regulatory documents on the design of bases and foundations were developed almost 40 years ago for the entire vast territory of the USSR and were adopted without taking into account the peculiarities of soil conditions in Kazakhstan. Introducing new regulatory documents for design in the Republic of Kazakhstan, taking into account the principles of the Geotechnics-7 Eurocode in 2020 year determines the review and clarification of the main provisions, taking into account engineering and geological features.

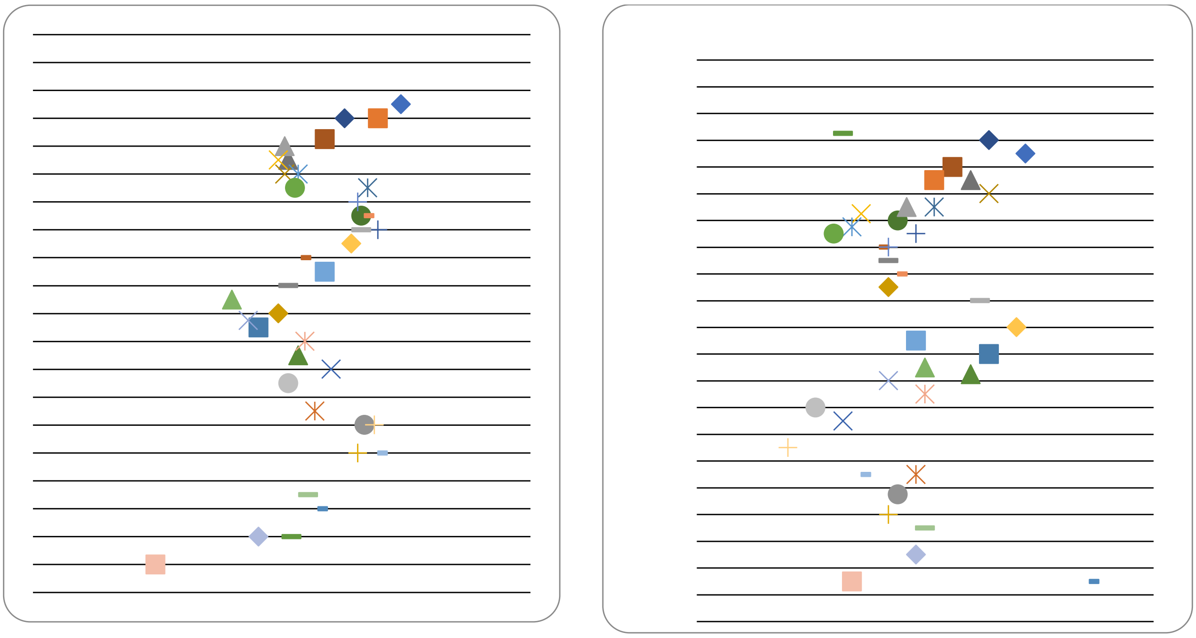
Comparative results given by the authors show that there are discrepancies in the calculation and design of the foundation bases:

- when selecting safety factors for foundation soil;
- when determining estimated resistance of the foundation soil;
- when selecting methods of determining bearing capacity;
- when determining methods of foundation settlements.

Thus, in order to move to the main provisions of international norms (Eurocode), it is necessary to adapt the basic principles of designing bases and foundations, as well as the design characteristics of the foundation soils, taking into account peculiarities of the engineering and geological conditions of the vast territory of Kazakhstan.

2. FEATURES OF ENGINEERING AND GEOLOGICAL CONDITIONS OF CLAY SOILS

Clay soils are widespread in the territory of Central Kazakhstan [1]. In particular, during intensive construction of the capital, the city of Nur-Sultan and industrial complexes of Karaganda, designers and builders are faced with clay soils in the foundations of buildings and structures. Figure 1 (a and b) shows graphs of changing the density and porosity coefficient of clay soils in depths up to 38.0 m. Soil densities in depth are variable: at the level of 9.0 m and 25.0 m the value is 1.7-1, 8 g/m^3, and at the depth of 3.0 m and 16.0 m it reaches 2.05-2.08 g/m^3 [1-5]. The porosity coefficient at depths from 7.0 m to 11.0 m and 24.0-26.0 m exceeds unity, confirming the heterogeneity of the formation. This circumstance makes it difficult to make calculations when designing foundations. In clay soils with full or partial water saturation, there is observed their changing from solid to plastic state, the loss of initial structural strength, and significant deterioration of the design parameters values: adhesion, the angle of internal friction and the deformation modulus.



a *b*

Fig. 1 Graphs of changing the density (a) and the coefficient of porosity (b) of clay soils

3. Results of studying the creep nature

The results of studies have established that structural strength of clay samples with undisturbed structure of natural moisture and density is from 25 kPa to 100 kPa. It was experimentally established that the process of consolidation of soil samples of undisturbed structure taken at various depths differs significantly in time and the nature of the deformation development.

The consolidation intensity for the studied soils at axial stresses from 25 kPa to 100 kPa practically does not change and is determined by the process of structural strength of the soil destruction, and at 100-400 kPa by the development of creep of the soil skeleton over time. The ultimate shear resistance of clay soils determined using different instruments practically does not coincide, for example, the values of the internal friction angles (according to More-Coulomb) for natural moisture clays W_0=13÷23%, determined on a triaxial compression device for various types of the stress state, turned out to be uneven and ranged from 13,200 to 19,680, and shear tests of the same soil on a single-plane cut-off device of the Maslov-Lurie design ranged from 22^0 до26^0.

There was established the effect of the stress state on strength and deformability of clay soils under shear creep. The largest value of the experimentally determined strength parameter (according to Mises-Botkin) corresponds to the Lode parameter $\mu_\sigma = -1$, the smallest μ_σ=0. The dependences of the strain intensity on the stress intensity during shear creep of clays were nonlinear and determined by the average normal stress. Figure 3 shows the test data for volumetric creep of the loam of the city of Karaganda. The value of comprehensive pressure in the experiments was: σ_m= 0.10; 0.20; 0.3 MPa. As the research results show, with increasing σ_m, the stabilization time of volumetric deformations increases. It should be noted that in experiments with a sample of natural density-moisture, decaying volumetric deformations was observed within 2.0-3.0 days. Moreover, with changing σ_m from 0.1 MPa to 0.3 MPa, volumetric deformations increased 2.1 times. In studying volumetric deformations of clays, there was also observed increasing the time of their stabilization with increasing pressure of comprehensive compression. So, if, at σ_m= 0.1 MPa, the full stabilization of volumetric creep deformations was achieved within 3.0-3.5 days, then at σ_m= 0.3 MPa it was 5-7 days. The analysis of the results of the study shows that clay soils of the base have a pronounced decaying creep.

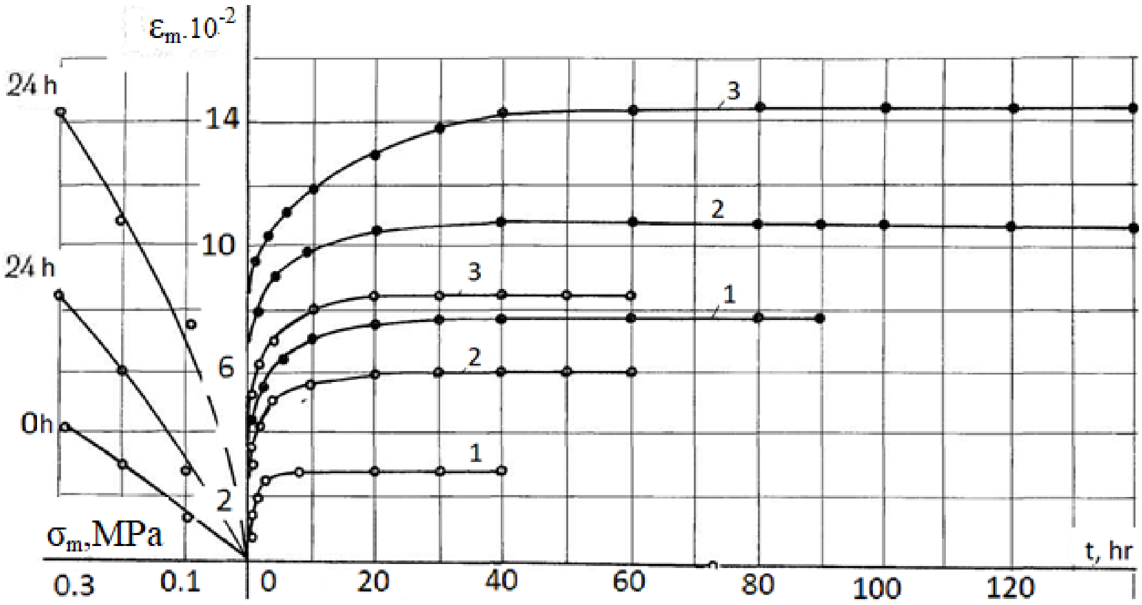


Fig. 2 Changing volumetric creep over time

It should be noted that the regulatory literature of the Republic of Kazakhstan does not take into account the base soil skeleton creep in time when calculating foundation settlements.

4. Determining the parameters of consolidation and creep

When testing clay soils to determine their structural compressive strength P_{str} the first and subsequent pressure steps are taken to be 0.0025 MPa until the compression of the soil sample begins. The onset of compression should be considered at a relative vertical deformation of the soil sample $\varepsilon > 0.005$, and the subsequent pressure steps P_1 are taken equal to 0.0125; 0.025; 0.05; 0.1 MPa and further with the interval of 0.1 MPa to the specified load value. For clay water-saturated soils, in case of partial softening after sampling and lifting the sample to the surface, the relative softening ε_1 should be calculated to determine P_{str} by the formula:

$$\varepsilon_1 = \Delta h_H/h = e_0(1-S_y)/1+e_0, \qquad (1)$$

where Δh_H is increasing the sample height in the course of decompaction, cm;

h is the sample height before testing, cm, см;

e_0 is the initial coefficient of the soil porosity after lifting the sample to the surface;

S_r is the soil moisture degree after lifting the sample to the surface.

The criterion of conditional stabilization of deformation is taken to be the strain rate of the sample not exceeding 0.01 mm over the last 16 hours for clay soils.

To determine C_v and C_a, a consolidation curve is constructed by the logarithmic method in the coordinates: the relative strain ε (ordinate) is the logarithm of the time *lg t* (abscissa). From the curve, one should find the deformation corresponding to 100% initial compression at a given load; to do this, a tangent to the final section of the curve $\varepsilon = f(lg\ t)$ is drawn. Then a tangent to the steepest part of the curve is drawn (Figure 3).

The intersection point of these tangents corresponds to 100% initial soil compression. The compression following 100% initial compression is defined as secondary compression caused by the creep strain. On the $\varepsilon = f(lg\ t)$ curve, one should find the value of the relative deformation corresponding to zero initial compression.

To determine the filtration consolidation coefficient by the logarithmic method for given pressure, the time required for the 50% initial compression is determined. To do this, the strain corresponding to the 50% initial compression is calculated, which is equal to the arithmetic mean between the strains corresponding to zero d_0 and 100% compression ε_{100}.

The time required for 50% initial compression with given pressure is found graphically according to the direct relationship $\varepsilon = f(lg\ t)$.

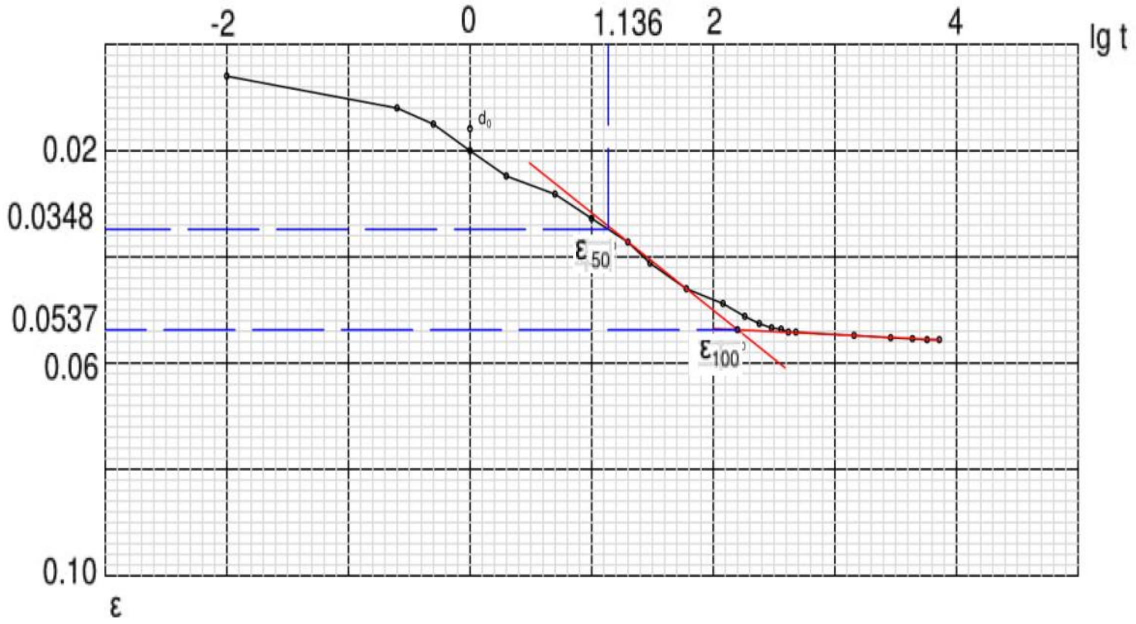


Fig. 3 Graphs of consolidation for determining the calculated parameters of consolidation and creep

The filtration consolidation coefficient C_V (cm^2/year), is calculated by the formula:

$$C_V = T_{50}h^2 / t_{50}, \quad (2)$$

where T_{50} is the coefficient (time factor) corresponding to the consolidation degree 0.5 and equal to 0.197;

t_{50} is the time corresponding to 50% initial compression, min.

The secondary consolidation coefficient C_a (dimensionless value) is determined by the tangent of the angle between the straight line parallel to the abscissa axis and the straight line segment of the curve in the secondary consolidation section using the formula:

$$C_a = tg_a = ε(t_2) – ε(t_1)/lg(t_2) - lg(t_2), \quad (3)$$

where $ε(t_2)$ and $ε(t_1)$ are the sample deformations in the secondary consolidation section; t_1 and t_2 is the time corresponding to deformations $ε(t_2)$ and $ε(t_1)$, min.

To study and determine the deformability parameters of water-saturated bases in assessing the stress state, a set of tests was carried out on samples of loamy soil of undisturbed structure on triaxial devices. Figure 5 shows that, according to the results of triaxial testing clays, there was obtained the deformation modulus, E = 19.3-23 MPa and the secant deformation modulus, E = 7.8-10.6 MPa.

The data obtained allow analyzing the prediction of foundation settlements taking into account creep.

5. Determining foundation settlements taking into account base soil creep

The Geotechnics-7 Eurocode recommends determining the foundation soil settlement by the formula:

$$S=S_0+S_1+S_2,< Su, \quad (4)$$

where S is the calculated (final) settlement;

S_0 is immediate elastic settlement;

S_1 is settlement caused by consolidation;

S_2 is settlement caused by creep (secondary);

Su are ultimate values of foundation absolute settlements (presented in the National Annexes).

Secondary (creep) settlement S_2 is caused by creep deformations. Deformations of this type arise in cohesive soils under loads exceeding a certain value, called the long-term strength limit. The development in time of deformations in solid, semi-solid and refractory clay soils occurs mainly due to the creep of their skeleton, practically without pressure in pore water. Within the construction period ($t<t_e$) the load on the foundation changes over time. To predict the creep settlement of the foundation under variable loads, it is advisable to use the Boltzmann-Voltaire theory of hereditary creep. Given the complex nature of the load growth and the complex form of the creep core, it is most convenient to solve the integral equations of the theory of hereditary creep using the Krylov-Bogolyubov numerical method.

For quasi-single-phase as well as single-component systems, determining the settlement of the soil layer is solved under the assumption that the settlement occurs only due to creep of the soil skeleton. The creep core is the creep rate of the soil at a constant unit stress. Creep will only affect the course of settlement over time, and a fully stabilized soil settlement in the case of a one-

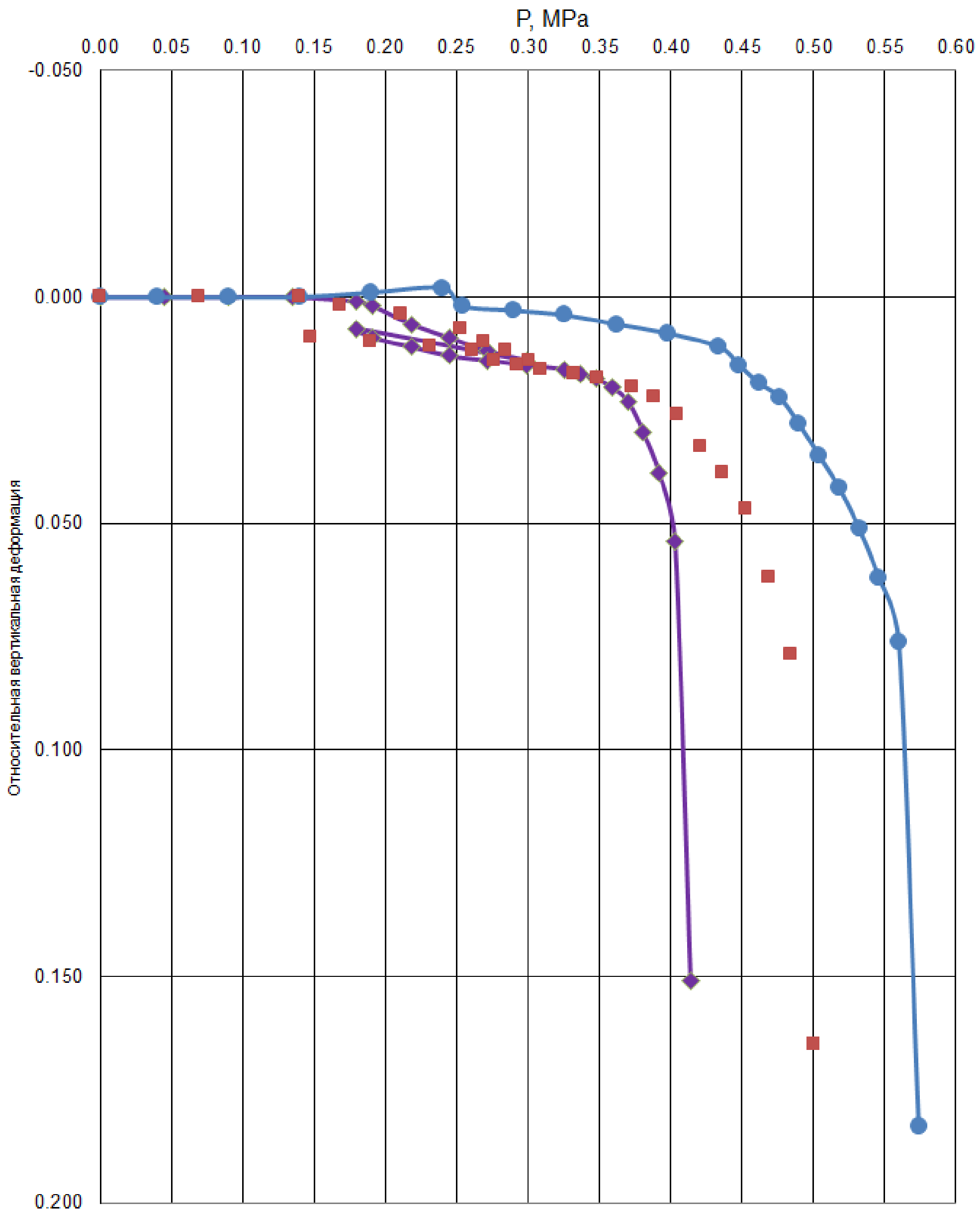


Fig. 4 Results of triaxial testing clays

dimensional problem will have the following expression:

$$S_2 = hm_sP, \quad (5)$$

where m_s is the coefficient of relative compressibility of the soil skeleton with creep.

It follows from the determining of decaying creep parameters that the coefficient of relative compressibility of the soil skeleton with creep m_s can be expressed by the equation:

$$m_s = C_V C_a (1-e^{-\delta_1 t}), \quad (6)$$

where C_V and C_a are coefficients of the initial and secondary soil consolidation,

$$m_s = m'\left[1 + \frac{Ca}{Cv}(1 - e^{-\delta_1 t})\right], \quad (7)$$

and since, according to the formula

$$\delta / \delta_1 = C_a / C_v, \quad (8)$$

and substituting the expression, we'll obtain that for the quasi-single-phase soil, creep settlements in time will be described by the expression:

$$S_2 = h_э C_v P[1 + C_a / C_V (1-e^{-\delta_1 t})]. \quad (9)$$

It should by noted that in case of applying a local load (from the structure foundation) there is accepted the value of the equivalent layer $h_э$.

Let's make a prediction of the foundation settlement with the help of an applied geotechnical program using the finite element method in the elastic-plastic formulation, taking into account the soil creep. To build a system of finite element equations, let's use the condition of equal power of the contour forces with displacement rates and internal stresses with deformation rates. In 1988-89, in the course of dismantling a blast furnace, elastic deformations of the base were recorded, the magnitude of which was 32 mm. After the construction of the blast furnace and its full load, its settlement stabilized after 2 months. The settlement was determined by the calculated load: for the blast furnace it was 300 t/m^2. The comparison of the calculated, actual, and maximum permissible settlements showed that the actual settlement of the foundations was several times smaller than the calculated settlements recommended by the regulatory literature. Actual settlements of industrial facilities are many times less than the calculated and limit values: for blast furnaces by 5-6 times.

The calculations of the foundation settlement prediction with the help of the applied geotechnical program using the finite element method in the elastic-plastic settlement, taking into account the soil creep showed that the discrepancies between actual and theoretical precipitations were no more than 15% (Figure 5).

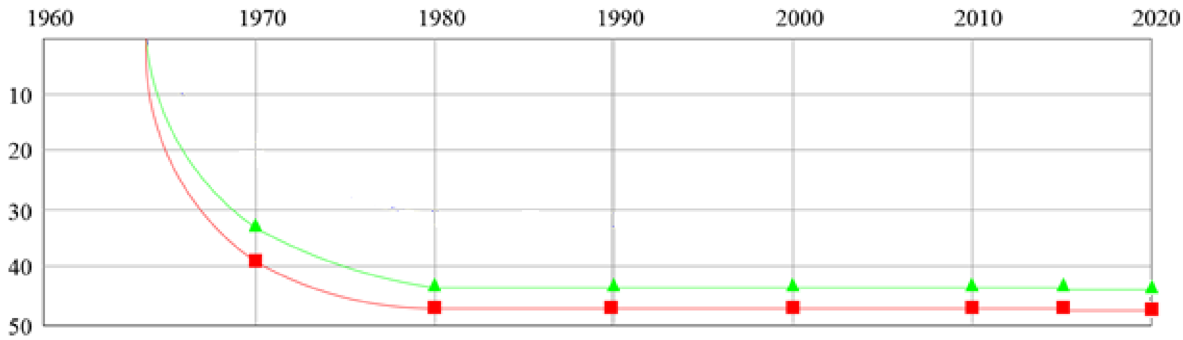


Fig. 5 Graphs of foundation settlements: experimental (squares) and calculated and theoretical (triangles) of the elastic-plastic model taking into account creep

Conclusion

The analysis of the results of methods for determining the foundation settlement shows that there is a difference in the approaches to solving problems and practical foundation analysis.

Accepting the basic principles of geotechnical design of the Geotechnics-7 Eurocode, we harmonize the methods of foundations analysis and put normative documents of the Republic of Kazakhstan to international standards.

In the geological-lithological structure, the construction site is composed of Quaternary sediments represented by clays.

As a result of laboratory studies, it has been established that with water saturation clays change the initial values of adhesion, the angle of internal friction, and the deformation modulus by 2–3 times.

The analysis of the results of the triaxial test study shows that clay soils of the base possess pronounced decaying creep.

According to the results of compression tests, there should be determined the coefficients of filtration consolidation and creep (secondary consolidation).

There has been proposed a formula for determining the foundation settlement taking into account creep for a quasi-single-phase soil base on the basis of experimental data.

References

[1] Ter-Martirosyan Z.G. Rheological parameters of soils and calculations of foundations of structures. M.: Stroyizdat, 2019. 200 p.
[2] Zaretsky Yu.K. Visco-ductility of soils and structural calculations. M.: Stroyizdat, 2014. 352 p.
[3] Fadeyev A.B. The finite element method in geomechanics. M.: Nedra, 2011. 221 p.
[4] Tsytovich N.A., et al. The forecast rate of foundation of structures settlements. M.: Stroyizdat, 2002. 239 p.
[5] Ukhov S.B., et al. Soil mechanics, bases and foundations. M.: Higher School, 2012. 566 p.
[6] Das, M. Braja. Principles of geotechnical engineering. Third Edition. 2003, PWS Publishing Company, Boston. P. 672.
[7] Paramonov V.N. The finite element method for solving nonlinear problems of geomechanics. 2012. Group of companies "Geoconstruction". St. Petersburg.
[8] Zhakulin.A.S. Deformability of water-saturated base soils. 2015. LAP Lambert Academic Pablishing. Saarbrucken.
[9] Zhakulin A.S., Zhakulina A.A Basics of geotechnical design. 2015. Karaganda, KSTU Publ. House, 162 p.
[10] Brinkgreve R.B. Jetal. PLAXIS 2004. Version 8. Balkema.
[11] Vyalov S.S. Rheological basis of soil mechanics. M.: Higher School, 2014, 447 p.
[12] Bulychev N.S. Mechanics of underground structures in examples and tasks. M: Nedra, 2014, 270 p.