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Development of sediment foundations of clay soils

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ABSTRACT: The article presents the results of long-term geodetic observations of deformations of blast furnace bases at the Karaganda Metallurgical Combine and the main provisions of the development of the elastic-plastic clay model for soils of Central Kazakhstan. The results of experimental studies of clay soils and numerical analysis of the forecast of basement sediments are presented.

1 INTRODUCTION

For more than 60 years, geodesic monitoring has been carried out for the precipitation of structures of the Karaganda Metallurgical Plant. During the construction of the foundations of blast furnaces No. 1-4, coke oven batteries No. 1-4 and chimneys with a height of 250 m, reference points were installed (according to the project requirements), which still record deformations of the foundations of structures. It is known that the main precipitation occurs during the construction of the structure and during the first few years of its operation, and in clay soils, precipitation stabilization occurs much later than in sandy soils. In this regard, it is very important to monitor the deformations of the base, starting from the moment of foundation construction, i.e. from the very beginning of load transfer to the base. It should be noted that the ground water level is at a depth of 23.0 meters and the ground at the base of the foundations is wet. In 1982, there was a need for a complete replacement of the blast furnace No. 1, its overhaul after 20 years of operation. During the dismantling period, elastic deformations of the base were recorded, the value of which was 32 mm. After the blast furnace was built and fully loaded, the sludge stabilized in 2 months. A similar pattern was observed during the reconstruction of all blast furnaces.

2 ANALYSIS OF THE RESULTS OF ENGINEERING AND GEOLOGICAL SURVEYS

Analysis of the results of engineering and geological surveys shows that most buildings and structures They are built on soils, mainly of Quaternary age (systems). Soils of the Quaternary system are characterized by the variability of their physical state, complexity, and variety of mechanical properties. Therefore, as part of the object (geotechnical system), the forecast of

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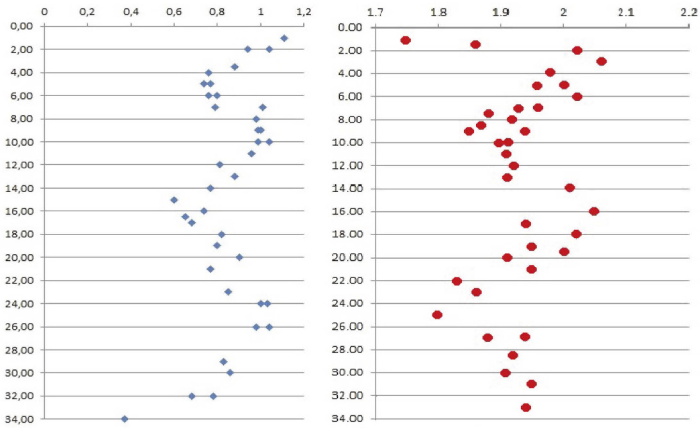
precipitation of industrial structures ‘foundations on clay soils of Central Kazakhstan was chosen as the subject of research. The main factors that lead to changes in the properties of clay soils of foundations are, first of all, man-made (technical and technological) and climatic (precipitation infiltration). The negativity of the process lies in the fact that the factors that lead to changes in the properties of the base soil are irreversible and are primarily associated with human engineering activities. Therefore, taking into account these factors that directly affect the stress-strain state of soils is necessary for a reliable forecast of foundation precipitation. Figure 1 shows the distribution of the porosity and density coefficients over depth on clay substrates. Analysis of the graphs shows that the porosity coefficients vary in depth from 0.6 (15.0 m) to 1.1 (1.0; 10.0; 25.0 m) and the density values - from 1.75 to 2.1 g/^{cm3}.



a) b)

Figure 1. Depth distribution of physical properties of clay soils. (a. porosity coefficient b. density).

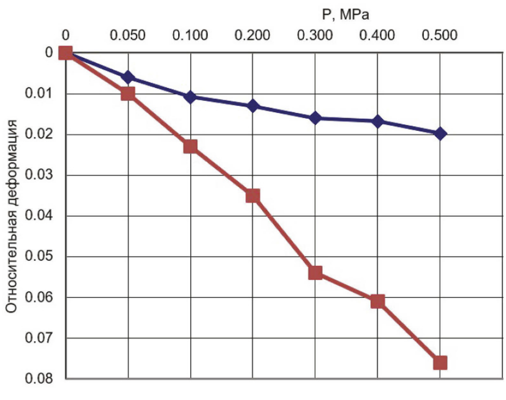


Figure 2. Compression graphs before and after soaking clays.

Figure 2 shows the results of testing clays in a compression device with the initial data: W=14.3, p=1.85 g/^{cm3}, e=0.695, While the results are obtained: natural humidity-m $_0$ = 0.045 MP a, E $_k$=17.8 MP a, E $_{od}$=44.4 MP a and at soaking – m0=0.254 $_{MP}$ a, ek=6.0 MP a$_,$ E $_{od}$=7.5 MP a. The analysis shows that the compression module is 2.9 times smaller when soaked than in the natural state, and the odometric module is 5.8 times smaller.

To study the stress-strain state and forecast the sedimentation of foundations of structures, an improved elastic-plastic model of clays based on the tent model of Professor T. Tanaka is presented [3,7,8]. Tent (deformation) models are the most developed for describing the deformation of clay soil bases [3]. When applying this model for clays, we consider the incremental relationship between stresses and deformations when testing soils on triaxial compression devices. The analysis of the results is carried out in stresses, since clay soils of the base are moist and not fully compacted in their natural state:

$$p = 1/3(\sigma_1 + 2\sigma_2) И \, q = \sigma_1 - \sigma_2, \tag{1}$$

and deformations:

$$_{ei} = {}_{e1+} 2e2 \text{ and } \gamma = 2/3 \, (_{e1} - e2), \tag{2}$$

where σ_1, σ_2, ε_1, ε_2 - stresses and deformations.

A graphic representation of the tent model is shown in Figure 3.

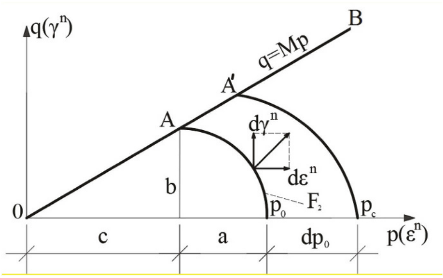


Figure 3. Graphic representation of the elastoplastic model.

Line with the equation:

$$Mp = q, \tag{3}$$

in Figure 3: the line with the equation $q = Mp$ is the limit line of this soil,

where M = constant is a coefficient that varies within 0.7÷1,2, which corresponds to the internal friction angles from 18 ° to 30 °.

When solving the problem, the following model assumptions are made::

– isotropic compression under pressure – P_o forms the elastic zone OAR_0 and within this zone the change in the stress state is accompanied only by small elastic deformations. Beyond the boundary of the yield surface AP_0 is accompanied by the appearance of plastic deformations dy^n and dfe^n;
– the yield surface AP_0, this is the surface of the plastic potential and, in accordance with the associated flow law, the total vector of plastic deformations is normal to the yield surface.

An increase in stresses, accompanied by the accumulation of plastic deformations, will lead to an expansion of the elastic region and the yield surface (tent) will occupy a new position A'P *C.* At the same time, the volume plastic deformation at any point of the new tent will be independently constant and equal to the deformation by which the soil is deformed under isotropic compression from pressure P_0 and P_C. Experimental compression parameters for implementing the model – λ and decompressions – k were obtained from compression curves in semilogarithmic coordinates (Figure 4). In this case, changes in the soil porosity coefficient linearly depend on the logarithm of the external pressure change.

Then the equation of the compression curve has the form:

for the load branch

$$e_i = e_0 - \lambda \ln\left(\frac{P_i}{P_0}\right), \tag{4}$$

for the unloading branch

$$e_i = e_0 - k \ln\left(\frac{P_i}{P_0}\right), \tag{5}$$

where e0 and P0 are the initial porosity and pressure coefficients; *ei* $_{and}$ Pi *are the porosity coefficient and pressure corresponding to the* $_{i\text{-}th}$ load stage; λ – compression ratio; k – decompression ratio.

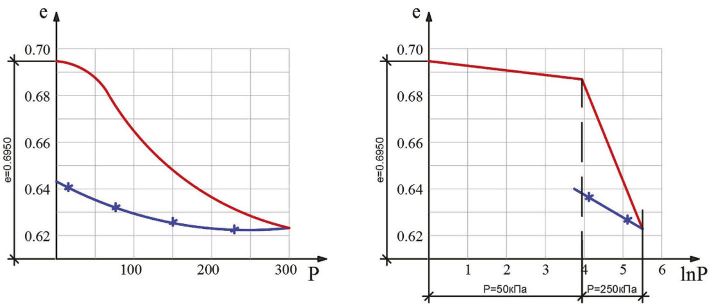


Figure 4. Compression graphs for determining load parameters-λ and unloading – k.

Compression ratios λ and decompression k is the tangent of the angle of inclination of the semi-logarithmic curve to the pressure axis, numerically equal to the difference in the porosity coefficients. If *Pi* = *e*, this condition should be taken into account in *Pi* = *e ln pi* = *1* [3,4]. These coefficients are dimensionless and characterize the compressibility of soils in a large pressure range [5].

Bearing in mind that the total increase in volume deformation consists of elastic and plastic parts, the total deformation is defined as:

$$d\varepsilon_{ij} = d\varepsilon_v = d\varepsilon_{ij}^y + d\varepsilon_{ij}^n = d\varepsilon_v^y + d\varepsilon_v^n. \tag{6}$$

Increments of the elastic strain component are related by linear relations of the generalized Hooke's law with stress increments:

$$d\varepsilon_{ij}^y = \left[(1+v)d\sigma_{ij} - 3v\delta_{ij}d\sigma_{cp}\right]/E, \tag{7}$$

where δ_{ij} is the Kronecker symbol.

The increment of plastic deformation components that occurs under active loading at a regular point on the loading surface is determined by the ratio:

$$d\varepsilon_{ij}^n = dk\frac{\partial \Phi}{\partial \sigma_{ij}}, \tag{8}$$

the use of which implies the construction of the loading surface *F*=*const,*
where *dk* is a scalar infinite multiplier; *F is* the loading function.
Loading function in the elastic domain at *F* < *0* in matrix form:

$$d\varepsilon = \ d\varepsilon^{y} + s \cdot d\varepsilon^{n} = D^{y-1} d\sigma + s\lambda B, \tag{9}$$

plastic deformations from experimental graphs are determined by:

$$\varepsilon_{ij}^{n} = \ \frac{\lambda - k}{1 + e_0} \frac{1}{2.3} \ln\left(\frac{-3P}{-3P_0}\right), \tag{10}$$

where: $-3P = P_s$
then:

$$\varepsilon_{ij}^{n} = -A \ln(-P_C) + B, \tag{11}$$

$$A = \frac{1}{2.3} \frac{\lambda - k}{1 + e_0} И \, B = -A \ln(-3P_0), \tag{12}$$

coefficients A and B are constant

$$P_c = \exp\left(\frac{B - \varepsilon_{ij}^{n}}{A}\right), \tag{13}$$

The yield surface equation for an isotropic medium in the stress space is written as:

$$f(\sigma) = 0, ij, \tag{14}$$

where f *is* the yield function in the elastic region $f < 0$.
In the case of an elastic-plastic medium $F = f$, then (8) takes the form:

$$d\varepsilon^{y} = dk \frac{\partial f}{\partial \sigma_{ij}}, \tag{15}$$

It should be noted here that if the guiding cosines of the normal to the loading surface are proportional, then the ratio (8) means the vector of increment of plastic deformation at a regular point directed along the normal to the loading surface-F_2 in any direction of the loading vector. In this case, only the value of the vector depends on the loading vector . The loading surface is used to establish the relationship between stresses and deformations in the pre – limit state of the soil, and the yield surface is used in the limit state.
In general, the yield function is defined as:

$$f = \alpha I_1 + \frac{J_2^{1/2}}{g(\theta)} - K = 0, \tag{16}$$

At: $f(\sigma, k) = 0; \ df = 0;$
Then we have

$$\mathrm{a}^{t} \, d\sigma - A\lambda = 0, \tag{17}$$

$$A = -\frac{1}{\lambda} \frac{\partial f}{\partial k} dk, \mathrm{a} = \frac{\partial f}{\partial \sigma}, \mathrm{b} = \frac{\partial \Phi}{\partial \sigma}, \tag{18}$$

the final element area coefficient is defined as follows:

$$S = \frac{Fb}{Fe}, \tag{19}$$

where fb is the area of the final element after shear or deformation; $_{F}\,e$ $_{is\ the\ area}$ of the final element at the beginning of the calculation.

Formula (9) is an equation for the relationship between strain increments and stresses for the accepted model. And also the proposed model considers the *OM* line as the limit line of normally compacted soil and defines:

$$\alpha_{cs} I_1 + \frac{I_3^{\frac{1}{2}}}{g(\theta)} = 0 \text{ or } \alpha_{xs} P_0 + \frac{b}{g(\theta)} = 0, \tag{20}$$

where: $b = -\alpha_{cs} P_0 g(\theta)$ and $\alpha = R(-\alpha_{cs} P_0 g(\theta))$,

Then

$$P_c = P_0 - a = P_0[1 + \alpha_{cs} g(\theta) R], \tag{21}$$

where α_{cs} – critical condition coefficient;

In this model, the associated yield surface and plastic potential for normally compacted clays ($F_2 = f$ or AP_c) is a section of an ellipse described by the equation:

$$\frac{(I_1 - P_0)^2}{a^2} + \frac{I_2}{b^2} = 1, \tag{22}$$

where *a* and *b* are geometric parameters of the ellipse,*a*/*b* = *R* = *const;* P0 is the center of the ellipse.

The yield surface is described by the equation:

$$f \;=\; F_2 =\; b^2 (I_1 - P_0)^2 + \;\frac{a^2 I_2}{(g(\theta))^2} -\; a^2 b^2 =\; 0, \tag{23}$$

or

$$f \;=\; F_2 =\; b^2 (P_c P_0)^2 + \;-\; a^2 b^2 =\; 0, \tag{24}$$

Given the Mohr-Coulomb law, we define:

$$g(\theta) = \frac{\cos\frac{\pi}{6} - \frac{1}{\sqrt{3}} \sin\frac{\pi}{6} \sin\varphi}{\cos\theta - \frac{1}{\sqrt{3}} \sin\theta \sin\varphi} = \frac{3 - \sin\varphi}{2\sqrt{3}\left(\cos\theta - \frac{1}{\sqrt{3}} \sin\theta \sin\varphi\right)}, \tag{25}$$

where θ - parameter: e;

$$\sin 3\theta = -\frac{3\sqrt{3} J_3}{2 J_2^{\frac{3}{2}}}, \tag{26}$$

The given invariants are determined by the well-known formulas.

The position of the ellipse is determined by two constants *a* and *b* and a function that relates the position of the tent to the bulk plastic deformation-$e\ v^{P}$.

$$P_c = F_2(\varepsilon_v^n), \tag{27}$$

It should be noted that the theoretical stress increase is defined as:

$$n\ v\ \varepsilon^{\,Kd} - y\ dp_{=}{}^{dp} \text{ and } dq = dq^{\,y} - 2\ Gdq^{\,n}, \tag{28}$$

where *dqy* and *dpy* are the elastic stress increment; *K, G are* the secant modules of bulk strain and shear.

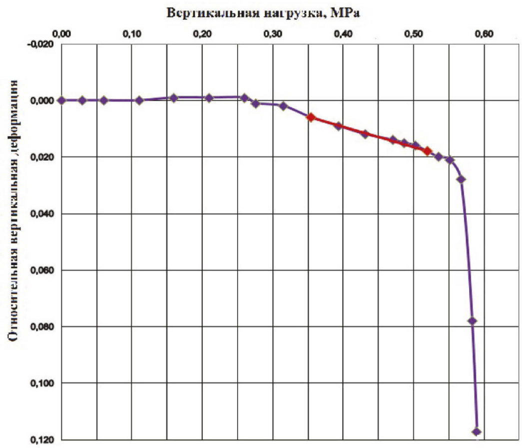


Figure 5. Results of triaxial clay tests.

Thus, to study the stress-strain state in an elastoplastic model, it is necessary to determine the strain modulus, Poisson's ratio, and secant strain modulus. Figure 5 shows that the results of triaxial tests of clays were obtained at the initial clay data-W=14.3%, p=1.85 g/^{cm3}, e=0.695, modulus of deformation-E = 13.8 MPa; Poisson's ratio-v = 0.21; secant modulus of deformation-E =7.2 MPa.

We will make a forecast of basement sediment using the Nonsolan applied geotechnical program (Japan) using the finite element method in the elastic-plastic formulation-application-Soil Creep. To construct a system of finite element equations, we use the condition that the powers of contour forces at displacement velocities and internal stresses at strain velocities are equal. Also, the forecast of basement sediment requires consolidation and creep parameters, which are determined using compression tests of clays (Figure 6).

To determine $_{C}v$ *and Ca* by the logarithmic method, we construct $_{a}$ consolidation curve in the coordinates: relative strain ε (ordinate) - logarithm of time *lg t* (abscissa). The curve should be used to find the strain corresponding to 100% primary compression at a given load. For this purpose, a tangent is drawn to the final section of the curve $\varepsilon = f\,(lg\ t)$. Then draw a tangent to the steepest part of the curve (Figure 6).

Figure 6. Consolidation charts for determining calculated consolidation parameters and creep.

Filtration consolidation coefficient-C_v (^{cm2}/year), calculated by the formula:

$$Cv = T_{50}h^2/t_{50}, \tag{29}$$

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

$50_{\ t}$ – time corresponding to 50% primary compression, min.

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

$$C_a = tg_a = \varepsilon(t_2\ ) - \varepsilon(t_1\ )/lg\ (t_2) - lg\ (t_2), \quad (30)$$

where $\varepsilon(t2)$ and $\varepsilon\ (t1)$ are the values of the sample deformation at the secondary consolidation site; $t1$ and $t2$ are the time corresponding to the deformations $\varepsilon(t2)$ and $\varepsilon\ (t1)$, min.

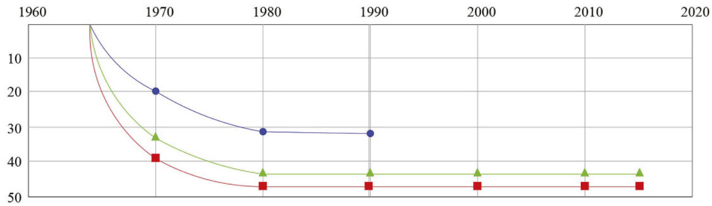


Figure 7. Graphs of basement sediments (experimental “round – elastic”, “triangular – full-scale” and theoretical ones using the elastic-plastic model).

3 CONCLUSION

It was found that the total draft for the blast furnace was 43.0 mm. Elastic precipitation due to soil compaction was 32.0 mm and creep precipitation – 12.0 mm.

To compare the calculated foundation precipitation with the actual and maximum permissible values, the foundation precipitation was recalculated. The draft was determined for the design load: for a blast furnace-300 $t/^{m2}$, with the dimensions of slab foundations: 27.0 m in diameter, with a depth of 3.5 m. Precipitation was calculated using the methods recommended by the regulatory literature. Comparison of calculated, actual, and maximum permissible precipitation showed that the actual precipitation of blast furnace foundations is several times less than the calculated precipitation recommended by the regulatory literature. The actual precipitation of the blast furnace is many times less than the calculated and limit values. Calculations of the forecast of basement precipitation using the Nonsolan applied geotechnical program (Japan) using the finite element method in the elastic-plastic formulation showed that the discrepancy between the actual and theoretical precipitation is no more than 15%.

Comparing the results obtained, we come to the conclusion that it is necessary to conduct additional studies to accumulate and systematize new data and make amendments to the generally accepted formulas for determining basement sediments.

Adjustments and corrections of the main calculation formulas will allow us to approximate the obtained data on the depth of propagation of the main compressive stresses, the amount of precipitation, and the calculated soil resistances under the base of foundations obtained theoretically to the practical values revealed by construction practice and long-term observations of deformations of buildings and structures in operation.

More accurate (real) calculated values will reduce the total cost of construction, since the design of building foundations is carried out with a large margin of safety, which entails unnecessary labor and material costs.

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