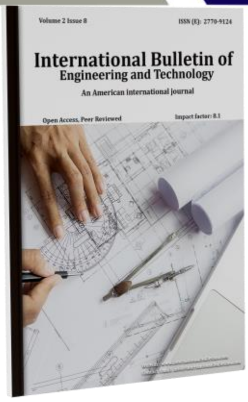

DETERMINATION OF THE CALCULATED (PERMISSIBLE) PRESSURE ON THE LOESS FOUNDATION OF BUILDINGS AND STRUCTURES IN SEISMIC CONDITIONS

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Annotation. This scientific article provides an improved method for determining the allowable (calculated) pressure on the loess foundation of buildings and structures during earthquakes. This improved method for determining the design pressure on the foundation is based on the condition that the average pressure from a structure erected in seismic areas should not exceed its design value, which is determined taking into account changes in the strength characteristics of soils during earthquakes. Obviously, in this case, when calculating the foundations for deformations, the additional seismic settlement of structures will be taken into account. In accordance with the proposed method, the design pressure on the base in seismic conditions is determined by the numerical value of the change in the stress state, soil connectivity and hydrodynamic gradient that occurs in the soil during its compaction. Therefore, the design pressure on the foundation in seismic conditions should be established based on the duration of the seismic load, which, as is known, depends on the soil cohesion and hydrodynamic gradient.

Key words: *design pressure; allowable pressure; seismic load; loess soils; seismic deformation; strength characteristics; soil connectivity; duration and intensity; hydrodynamic pressure gradient; stress state; destruction zones; load bearing capacity.*

Introduction. As is known, in accordance with regulatory documents, the calculation of foundations in seismic regions is provided for by the first limit state, which leads for all soils to the need to calculate their stability.

The main provisions for calculating foundations for a special combination of loads in the first limit state are based on determining the bearing capacity of the foundations in accordance with the approximate theory of the limiting equilibrium of soils, taking into account the eccentric effect on the foundation of the load and dynamic stresses in the foundation that arise during the propagation of seismic waves.

In this case, a partial separation of the base of the foundation is allowed, provided that the eccentricity of the load from the structures is limited, the reliability coefficient of the base in the compression zone under part of the base of the foundation is sufficient, and the marginal ordinates of the support exceed the limit value of the maximum contact stresses. As a result, a significant reduction in the size of the foundations is achieved.

The calculation is based on the possibility of deformation of the base (settlement, rolls, etc.), exceeding its limit value, permissible under the main combination of loads. Therefore, with a special combination of loads, taking into account seismic effects, the deformation of the bases is not calculated.

This basic provision for calculating the foundations for the first limiting state seems to us insufficient and far from the phenomena observed in natural conditions.

Materials and methods. To determine the calculated (permissible) pressure on the oscillating soil, consider the case of a foundation in the form of a strip with a depth of H, along the base of which a vertically distributed load of intensity Po acts (Fig. 1).

The main stresses at any point of the base from the load are expressed by the formulas:

$$G_1 = \frac{Po}{\pi}(1 + Kcl^{-\omega t})(\delta + sin\delta)$$
$$G_2 = \frac{Po}{\pi}(1 + Kcl^{-\omega t})(\delta - sin\delta) \qquad (1)$$

The limit equilibrium condition, expressed in terms of principal stresses, is described by the dependence:

where φ_(w-) is the angle of internal friction of the soil in natural occurrence ρ_(w-) is the average soil density at moisture content W;

C_(w-) ground adhesion.

The equation of lines limiting the area of limit equilibrium is obtained from expressions (1) and (2) in the form:

$$Sin\varphi = \frac{\frac{Po}{\pi}(1+Kcl^{-\omega t})sin\delta}{\frac{Po}{\pi}(1+Kcl^{-\omega t})\delta+\rho_w(z+H+\frac{C_w}{\rho_w tg\,\varphi_w})} \qquad (3)$$

From here:

$$z = \frac{Po}{\pi\rho_w}(1 + Kcl^{-\omega t})\left(\frac{Sin\delta}{Sin\varphi} - \delta\right) - \left(H + \frac{C_w}{\rho_w tg\,\varphi_w}\right) \qquad (4)$$

The maximum depth of development of the areas of limiting equilibrium is obtained from the condition H = 0, which can take place when:

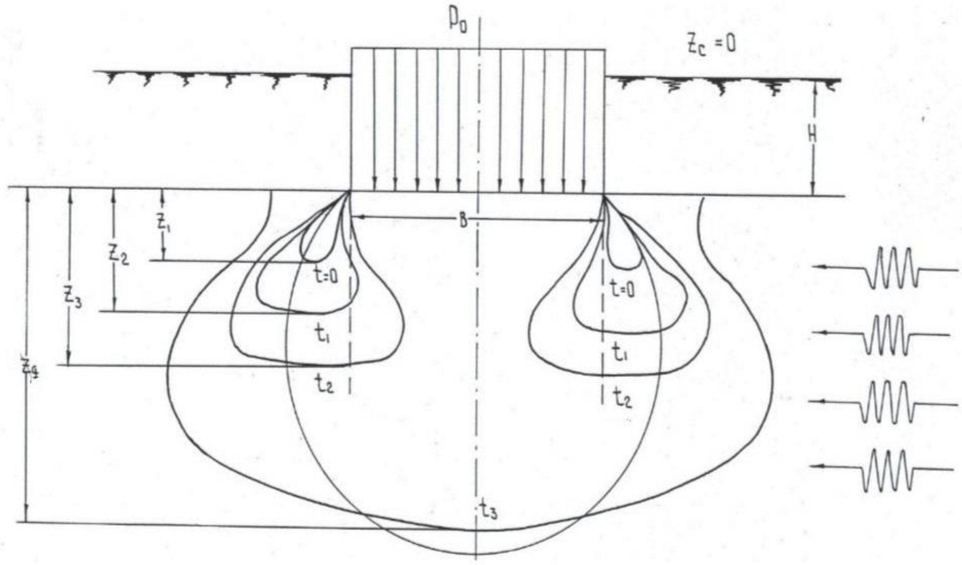

Fig.1 Calculation scheme for determining the design pressure on oscillating soil.

$$\frac{dz}{d\delta} = \frac{Po}{\pi\rho_w}(1 + Kcl^{-\omega t})\left(\frac{cos\delta}{Sin\varphi} - 1\right) = 0 \qquad (5)$$

where

$$\cos\delta = sin\varphi; \ \delta = \frac{\pi}{2} - \varphi \qquad (6)$$

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Taking into account dependencies (4) and (6), we have:

$$z_{max} = \frac{Po}{\pi \rho_w}(1 + Kcl^{-\omega t})\left(ct_g\, \varphi + \varphi - \frac{\pi}{2}\right) - \left(H + \frac{C_w}{\rho_w t_g\, \varphi_w}\right) \qquad (7)$$

According to this expression, the zone of limit equilibrium (destruction) z_{max} increases with increasing load Po. However, as noted above, for the dynamic conditions of the soil, this provision is valid in the case when the condition $\alpha_{\text{кр}} > \alpha_{\text{c}}$.

Otherwise (when $\alpha_{\text{кр}} < \alpha_{\text{c}}$), it is possible to increase in time the destruction zone under the acting load in the seismic conditions of the foundation. this is an increase, in accordance with the findings of Rasulov Kh.Z. depends on the change in time of the strength characteristics of the foundation soils.

This, in accordance with the studies carried out above, is associated with the duration of the seismic load, depending on which the active zone, which passes into a dynamically disturbed state, increases, within which a decrease in soil strength is observed.

Therefore, when determining the design pressure on the base, in addition to inertial forces, it is necessary to take into account a number of other factors that determine the dynamic regime of the soil in the boundary zone of the structure. It is obvious that with full weighing (liquefaction) of the soil layer in areas bordering the foundation, the depth will be lost.

In all cases, this is accompanied by a gradual weakening of the internal bonds of the soil during vibration. Therefore, the issue of reducing the role of deepening of the structure under these conditions must be made dependent on the duration of the dynamic impact. In this case, formula (7) takes the form:

$$z_{\text{c}} = \frac{Po}{\pi \rho_w}(1 + Kcl^{-\omega t})\left(ct_g\, \varphi + \varphi - \frac{\pi}{2}\right) - \left(H(t) + \frac{C_w(t)}{\rho_w t_g\, \varphi_w}\right) \qquad (8)$$

According to this expression, the size of the limit equilibrium zone will increase due to a decrease in the loess connectivity $C_w(t)$ in time and a decrease in the depth H(t) in seismic conditions.

A slight tolerance of the destruction zone z_c when determining the design load, as it is done in the static analysis of structures, already leads to a violation of the overall stability of the foundation. Hence the need arises to base the calculation

z_c =0 for dynamic operating conditions of foundations composed of soils capable of moving into a dynamically disturbed state. Based on this, it is possible to determine the value of the load P_o from expression (8), which in this case is considered as calculated:

$$P_{(t)} = \frac{\pi \rho_w \left[H(t) + \frac{C_w(t)}{\rho_w t_g\, \varphi_w}\right]}{(1+Kcl^{-\omega t})\left(ct_g\, \varphi + \varphi - \frac{\pi}{2}\right)} + \rho_w H \ldots \qquad (9)$$

where $\rho_w\, H$ - is the weight of the soil excavated from the pit.

In expression (9) C_w(t) corresponds to the weakening of the connection of the loess during oscillation and is determined by the formula of prof. Rasulova Kh.Z. as:

$$C_w(\text{t}) = C_w(K) + [C_w(\text{H}) - C_w(K)]\, l^{-\mu t} \ldots \qquad (10)$$

where C_w(t), $C_w(K)$ - respectively initial (before vibration) and final (after vibration) values of soil cohesion;

t- is the duration of the oscillation;

μ- is a parameter determined experimentally.

According to formula (9), the decrease in the design pressure on the base, in addition to the value of $C_w(t)$, will depend on the drop in the role of penetration H in the dynamic conditions of the structures.

In accordance with works /3,4/ this drop can be written as:

$$\rho_w H(t) = \rho_w H - \Delta b h(t) \qquad (11)$$

where, Δb is the density of water:

$h(t)$ - dynamic pressure arising in the thickness of water-saturated soil during its compaction, which is determined by the formula /9/:

$$h(z,t) = \frac{z^2}{2K\varphi} \vartheta_n (1 - l^{-\mu t}) \ldots \qquad (12)$$

here z-is the active zone, ϑ_n, μ-parameters determined empirically.

Expression (11) about taking into account (12) will be rewritten:

$$\rho_w H(t) = \rho_w H - \frac{z^2}{2K\varphi} \Delta b \, \vartheta_n (1 - l^{-\mu t}) \qquad (13)$$

Dependence (9) can be represented as:

$$\text{P (t)} = \frac{\pi \rho_w \{H - \frac{z^2}{2K_\varphi} \vartheta_0 (1 - l^{-\mu t}) + \frac{C_w(K) + [C_w(H) - C_w(K)] \, l^{-\mu t}}{\rho_w t_g \varphi_w}\}}{(1 + K_c l^{-\omega t})(ct_g \varphi + \varphi - \frac{\pi}{2})} + \rho_w H \ldots \qquad (14)$$

where, $\rho_w \approx \Delta b \approx 1.0$ т/м^3.

Considering that the value $\frac{z}{K_\varphi} \vartheta_0 (1 - l^{-\mu t}$ is the hydrodynamic pressure gradient $\Im(t)$, acting in the thickness of the core at time t, we finally obtain.

$$\text{P(t)} = \frac{\pi \rho_w \{H - \Im(t) \frac{z}{2} + \frac{C_w(K) + [C_w(H) - C_w(K)] \, l^{-\mu t}}{\rho_w t_g \varphi_w}\}}{(1 + K_c l^{-\omega t})(ct_g \varphi + \varphi - \frac{\pi}{2})} + \rho_w H \ldots \qquad (15)$$

Expression (15) makes it possible to determine the value of the design pressure on the base, taking into account the transition to a dynamically unstable state of the soil lying in the zones bordering the structure when seismic forces act on the base.

As follows from this expression, the value of P(t) will be less in seismic conditions than in static ones, due to the weakening of the cohesion $C_w(t)$ of the soil and the decrease in the role of penetration H(t).

In accordance with this formula, the design pressure on the base must be set taking into account the duration of the seismic load, on which the values C_w and $\Im$.

CONCLUSIONS

1. Among the physical and geological processes, the greatest danger to buildings and structures is the seismic subsidence of loess rocks. Any additional moistening of the base of buildings and structures erected on such soils. Under seismic impacts, it can lead to seismic subsidence. Seismic impacts can also cause a violation of the dynamic stability of soils filled in the areas bordering the foundations, which often causes a weakening of the bearing capacity of the foundation itself.

2. Uneven deformations of structures erected on moist loess soils are in most cases associated with a weakening of soil strength and a decrease in the overall stability of foundations during earthquakes.

3. When designing foundations in seismic regions, it is important to determine the expected additional deformation, taking into account the possible duration and intensity of an earthquake, changes in the strength characteristics of soils under these conditions.

4. The conclusions noted in paragraphs 1, 2 and 3 to a certain extent testify to the insufficiency of calculating the foundations of structures erected in seismic regions according to the first limit state, which does not take into account the seismic deformation of soils.

5. Subsidence of foundation soil, which manifests itself during earthquakes, often determines the degree of destruction of structures under these conditions. At the same time, along with seismic intensity, the magnitude of the static load from the structure plays an important role.

6. The developed method for determining the design pressure on the base is based on the condition that the average pressure from a structure erected in seismic areas should not exceed its design value, determined by taking into account changes in the strength characteristics of soils during earthquakes. Obviously, in this case, when calculating the foundations for deformations, the additional seismic settlement of the structures will be taken into account.

In accordance with the proposed method, the design pressure on the base under seismic conditions is determined by the numerical value of the change in the stress state, soil cohesion and hydrodynamic gradient that occurs in the soil during its compaction, and therefore P(t) should be set based on the duration of the seismic load, on which, as is known, the quantities C_w and $\mathfrak{I}$. depend.

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