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Approximate solution of the problem on pressure of cohesionless soil on the displacing retaining wall

Titre en français. Une solution approximative au problème de la pression d'un sol non cohésif sur un mur de soutènement déplaçable

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ABSTRACT: the authors have developed an approximate engineering method of assessing the lateral pressure of non-cohesive soil on a displaced retaining wall, based on experimentally determined in laboratory conditions in the prelimit and limiting state under plane strain conditions and some simplifying assumptions. A method for determining of the lateral pressure of a sand filling on an absolutely rigid retaining wall, when it moves both away from the filling and the side of the filling is considered. The correctness of the obtained ratios was checked by comparing the experimental and some field data on the lateral soil pressure on the displaced retaining walls with the calculated values.

RÉSUMÉ : Les auteurs ont développé une méthode d'ingénierie approximative pour évaluer la pression latérale d'un sol non cohésif sur un mur de soutènement déplaçable, basée sur des dépendances déterminées de manière expérimentale (dans des conditions du laboratoire) entre les contraintes (tensions) et les déformations du sol de remblayage (à l'état pré-limité et limité dans des conditions de déformation plane) ainsi que quelques hypothèses simplifiées .Un procédé pour déterminer la pression latérale d'un remplissage de sable sur un mur de soutènement absolument rigide, lorsqu'il se déplace à la fois du remplissage et vers le remplissage, a été observé. L'exactitude des corrélations obtenues a été vérifiée en comparant des données expérimentales et certaines données de terrain concernant la pression latérale du sol sur les murs de soutènement déplaçables avec les valeurs calculées.

KEYWORDS: active, passive and rest pressure, retaining wall.

1 INTRODUCTION

Soil pressure on the retaining walls depends on numerous factors:

-structural features of the retaining wall itself, physical and mechanical properties of the soil, lying behind the wall and at the base, the wall movement as well as its rotation.

-simultaneous accounting for the influence of all factors for the formation of the soil pressure does not seem possible. The theory of soil mechanics shows that the soil located behind the retaining wall can show an infinite number of equilibrium state. Typical are those, that give the lowest and highest values of soil pressure. This condition of the soil occurs when the retaining wall, located in front of the soil is moved or titled. The state of equilibrium of the soil formed without any movement or inclination of the wall determines the resting pressure.

At present, when designing the retaining wall to determine forces arising from the equal action with the soil of the backfill, the solutions obtained by coulomb are mainly used. The lateral soil pressure on the displaceable retaining wall is determined by the coulomb without reference to the displacement value. Active and passive pressures are determined on the assumption that the backfill soil is in its ultimate strength state. It should be noted that, when the wall moves from the ground, the limiting state is achieved at sufficiently small relative displacements (displacements Δl, related to the height H of the wall Δl/H=0,001+0,01). Thus, the active pressure, corresponding to the limiting state of the backfill is practically always realized when the wall can be displaced away from the backfill, and in this case the Coulomb formulas can be used.

At the same time as numerous experiments show, the full passive pressure is realized with a significant relative displacement of the wall towards the ground (0,01+0,1). Displacement of such a value under natural conditions is never achieved and therefore limiting state of the backfill soil practically does not occur. In this regard, the values of passive soil pressure determined by the coulomb calculation as a rule, are significantly overestimated and this overestimation is greater, the less the displacement of the wall towards the filling. The exact solution of this problem is difficult, since it is associated with the study of the behavior of the backfill soil in the elastic-plastic stage of its operation.

1.1 *Initial regularities of the deformation of cohesionless soils.*

The proposed engineering method for assessing the lateral pressure on cohesiveness soil on a displaceable retaining wall is based on the relationships between stresses and deformations of the backfill soil determined experimentally in laboratory conditions and some simplifying assumptions.

The relationship between stresses and deformations used in calculations were obtained from our experiments (Ikramov, 1982, carried out in conditions of plane deformation along trajectories I and II (See Fig. I)). The choice of these trajectories is due to the nature of the retaining wall. The results of experiments on trajectory I(σ_1 =const; d σ_3 <0) were used in this calculation method to determine the passive soil pressure on the retaining wall and the data on trajectory II (σ_1=const; dσ_3<0) were used to determine the active soil pressure.

Initial regularities of deformation of cohesiveness soils obtained on the basis of experimental data (Ikramov 1982) data (fine Lyubertsy sand with the following physical characteristics was used at the test soil: specific gravity of soil particles γ_s=26.6 kN/m³; specific gravity of dry soil in extremely loose state γ_d=14.1 kN/m³ and in the extremely dense state γ_{dmax}=17.1 kN/m³; initial relative density I_D =0.8 while γ_d =16.4 kN/m³, e=0.63; at I_D=0.55 , γ_d=15.6 kN/ m³, e=0.71), can be represented as a dependence:

$$f=\frac{\sigma_1-\sigma_3}{\sigma_1+\sigma_3}=\Phi\,(\gamma_{\max})\,f^* \quad (1)$$

where γ_{max} is the maximum shear deformation, equal to $\gamma_{max}=\frac{e_1-e_3}{2}$.

Relationship (1) can be written:

$$f=\frac{\sigma_1-\sigma_3}{\sigma_1+\sigma_3}=f^{*}\cdot\Phi(\gamma_{max}) \qquad (2)$$

where f^{*} is the function value, corresponding to the ultimate strength state of the soil. The function f can be written otherwise:

$$f=\frac{\sigma_1-\sigma_3}{\sigma_1+\sigma_3}=sin\varphi_1 \qquad (3)$$

In the limiting state, the angle φ_1 will be equal to the angle of internal Mohr friction φ^{*}. Using ratios (1) and (3), as well as the main dependence (2), it is possible to represent the dependence of the current angle φ_1 on the value of γ_{max} (see Figure 2) as follows:

$$sin\varphi_1 = sin\varphi^{*}\cdot\Phi(\gamma_{max})$$

or (4)

$$\varphi_1 = arcsin\,[sin\varphi^{*}\cdot\Phi(\gamma_{max})]$$

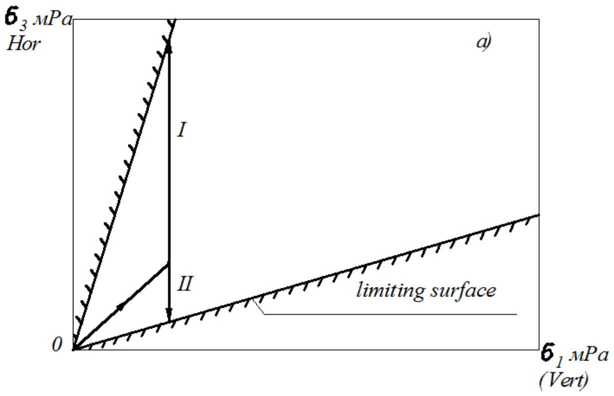

Figure 1. Investigated loading paths

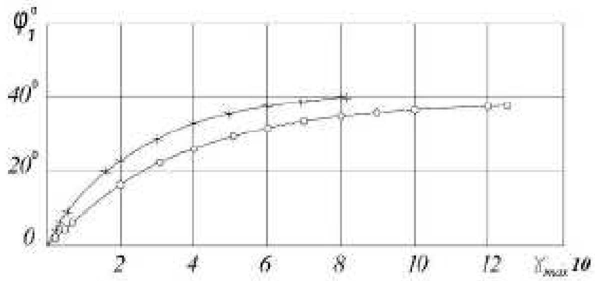

Figure 2. Dependence φ; from γ_{max}

x-x-x I_D=0,8
0-0-0 I_D=0,55

The function Φ (γ_{max}) should have the property, that when γ_{max}= 0 it goes to zero and when γ_{max} tends to the limiting value γ^{*}_{max} corresponding to the destruction of the soil, the function $\Phi(\gamma_{max})$ tends to one. This means that the angle of deflection at γ_{max}=0 is equal to zero, and at $\gamma_{max}\rightarrow\gamma^{*}_{max}$, the deflection angle φ_1 must coincide with the angle of internal friction φ^{*}(see Figure 3).

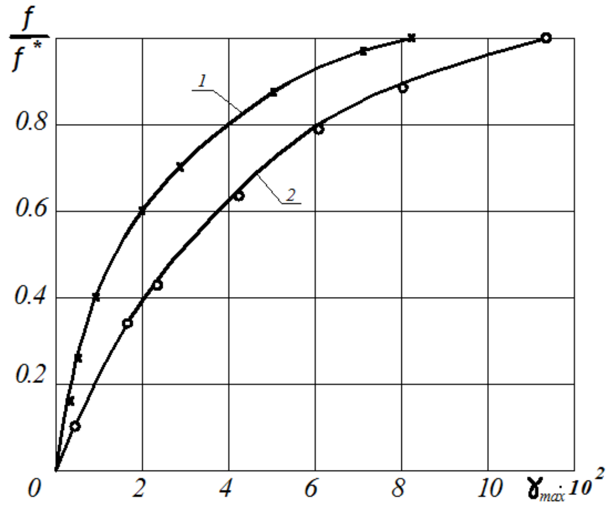

Figure 3. Dependence $\frac{f}{f^{*}}$ from γ_{max} 1-I_D=0,8; 2-I_D=0,55;

1-x-x-x-x I_D=0,8
2-0-0-0-0 I_D=0,55

According to the results of experiments (trajectories I and II) (see Fig.1) under conditions of plane deformation, the strength properties of the studied soil (Lyubertsy sand) are characterized by the angle of internal friction according to Mohr, and obtained by experiments: for γ_d=15.6 kH/m³ (I_D=0.55), angle φ^{*}= 39°; for γ_d=16.4 kN/m^3 (I_D=0.8) the angle φ^{*}=41°.
To describe the shear strain, the function $\Phi(\gamma_{max})$ is taken in the form of fractional linear dependence, similar to those used by A.I.Botkin (Botkin 1939):

$$\Phi(\gamma_{max})\frac{\gamma_{max}}{A+B\gamma_{max}}=\frac{e_1-e_3}{2A+B(e_1-e_3)} \qquad (5)$$

Here, A and B are dimensionless parameters for Lyubertsy sand, their value for the studied loading trajectories (see Figure 1) and graphs (see Figure 4.5) are given in Table 1.

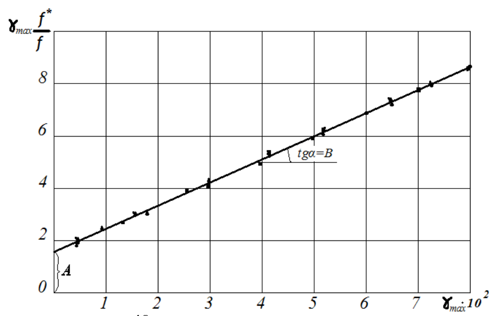

Figure 4. Dependence $\frac{f^{*}}{f}\gamma_{max}$ from γ_{max} trajectory I, I_D=0.8

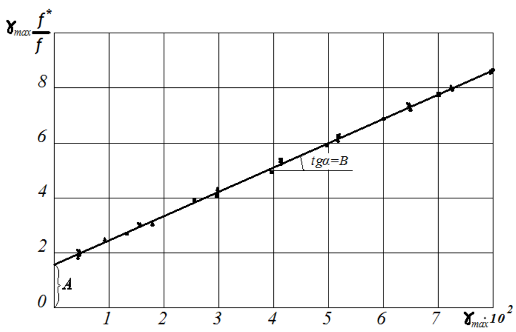

Figure: 5. Dependence $\frac{f}{f^{*}}\gamma_{max}$ from γ_{max} trajectory II, I_D=0.8

Table 1

Loading path (Fig. 1)	Specified loading condition	A		B	
		I_D=0,8	I_D=0,55	I_D=0,8	I_D=0,55
I	σ_1=const $d\sigma_3$>0	1,70	2,75	0,8	0,78
II	σ_1=const $d\sigma_3$<0	0,15	0,30	0,97	0,97

The second main dependence, which is used in the proposed method for calculating the lateral soil pressure on retaining walls is the parameter dependence ν (in the deformation theory of plasticity, it is analogous to Poisson's ratio) on the maximum shear strain $\gamma_{\max}$ (see Figure 6). In accordance with the deformation theory of plasticity, this parameter can be expressed on the basis of the generalized nonlinear Hooke's law (provided e_2= 0) in the form:

$$\nu = \frac{\sigma_2}{\sigma_1+\sigma_3} \tag{6}$$

$$\nu' = \frac{e_1\sigma_3 - e_3\sigma_1}{(e_1-e_3)(\sigma_1+\sigma_3)} \tag{7}$$

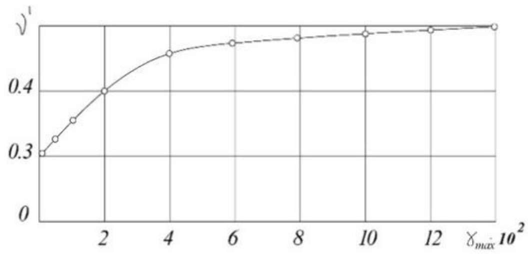

Figure 6. Dependence γ from γ_{max}

and is determined from experiments with known experimental values of σ_1, σ_3 and the corresponding values of e_1, e_3. Based on the found values of the coefficient ν ', the dependence is easily identified (see Figure 7):

$$\frac{\nu'}{1-\nu'} = F(\gamma_{\max}) \tag{8}$$

The dependence (8) can be approximated in the form:

$$F(\gamma_{\max}) = \xi_0 + (1-\xi_0)(1-e^{-\alpha\gamma_{\max}}) \tag{9}$$

where α=0.60 and ξ_0=0.42. The value of the coefficient of lateral soil pressure at rest, at $\gamma_{\max}$=0, obtained experimentally, is close to the data of K.Terzagi (Tertsagi 1958).

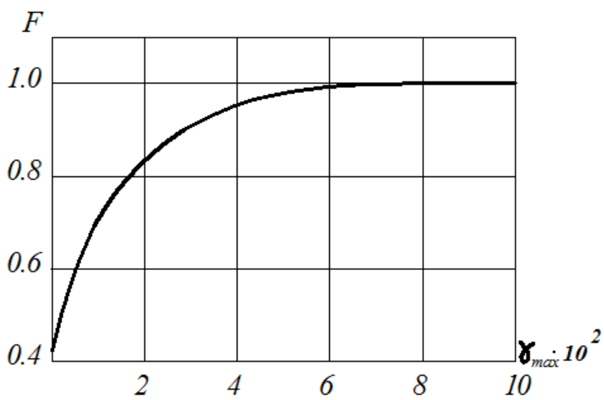

Figure 7. Dependence $F = \frac{\nu^1}{1-\nu^1}$ from the maximum shear

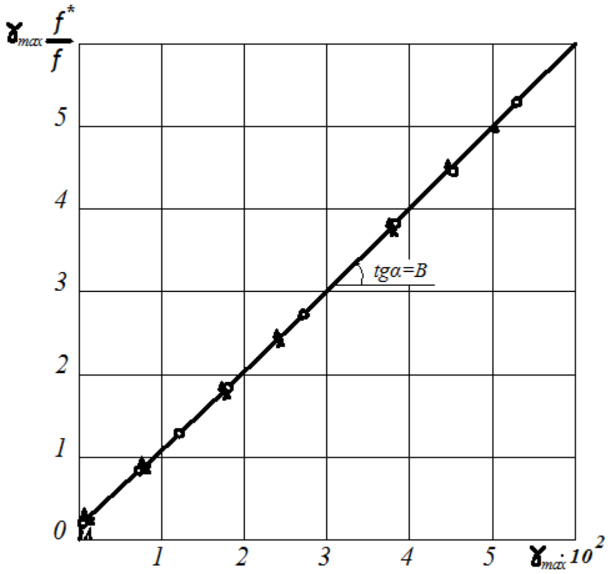

Figure 8. Dependence ln $\frac{(1-\xi_0)}{1-F}$ from $\gamma_{\max}$

The dependencies (1), (5), (9) are taken as the initial dependences for constructing a method for calculating the lateral soil pressure on the displaced retaining walls.

1.1.1 M*ethod for calculating soil pressure on retaining walls*

The method for calculating soil pressure on retaining walls is based on the assumption of interaction between the structure (see Figure 9)

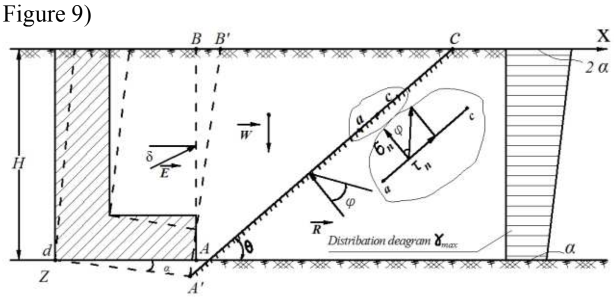

Figure 9. The calculated scheme of a retaining wall

of the retaining structure and the "wedge-shaped body" of the backfill soil. The outline of this "wedge-shaped body" is determined based on the fact that both in the prelimit and ultimate strength state, soil deformation occurs along the areas determined by the values of the function f, the soil prism (see Figure 9) is under the action of a vertical force equal to the weight of the selected prism $\vec{W}$, the resistant forces along the slip line $\vec{R}$, and the reactive pressure from the wall $\vec{E}$.

The enveloping slip line of the AC (see Figure 9) in the pre-limiting state of the soil is actually some fictitious line and does not physically determine the discontinuity of the medium; it represents the locus of points at which maximum plastic shear deformations are realized and it is more correct to call it the line of maximum shears. In the ultimate strength state, the soil prism ABC (see Figure 9) has the ability to move as a whole, and in this case, the balance of the prism under the action of the forces $\vec{E}$*,$\vec{R}$* and $\vec{W}$* can be considered. The AC plane becomes a real slip plane, the deflection angle φ 'in this case is equal to the angle of internal friction φ^*.

If, in the pre-limiting state of equilibrium of the ABC prism, it is determined that the sum of the forces $\vec{E}$,$\vec{R}$ and $\vec{W}$ is equal to zero, then we obtain an overestimated value of the lateral pressure on the retaining wall. Indeed, in the limiting state, the ABC prism cannot be regarded as a displaced rigid nondeformable region, and in this case the following inequality holds:

$$\vec{E} < \vec{R} + \vec{W}, \tag{10}$$

With the introduction of a certain decreasing coefficient æ inequality (10) can be transformed into the equality:

$$\vec{E} = æ(\vec{R} + \vec{W}) \tag{11}$$

The coefficient æ is limited, it is within $0 \le æ \le 1$, it depends on the realized shifts and, by assumption, is determined by the values of the function $F(\gamma_{\max})$ taken in dependence (11). So, for shifts $(\gamma_{\max})$, tending to the limiting value, the coefficient is equal to 1, and at rest, it should be close to the coefficient of rest pressure ξ_0. The angle of friction along the wall at rest is equal to zero. Then assumption (11) can be written in the following form:

$$E = F(\gamma_{\max}) W \frac{\sin(\theta - \varphi_1)}{\sin\left(\frac{\pi}{2} + \delta + \varphi_1 - \theta\right)} \tag{12}$$

The weight of the displaced prism ABC in accordance with the geometric shape of the section (see Figure 9) is:

$$W = \rho^{\text{ск}} g \frac{H^2}{2} \sin\left(\frac{\pi}{2} - \theta\right) \tag{13}$$

Taking into account dependencies (5), (9) and (11), formulas for determining the lateral soil pressure on retaining walls are as follows:

- for active pressure

$$Ea = \frac{F^{\text{ср}} \cdot \rho^{\text{ск}} g \cdot H^2}{2\cos\delta} \left[\frac{1}{\left(\frac{1}{\cos\varphi_1}\right) + \sqrt{tg^2\varphi_1 + tg\varphi_1 \cdot tg\delta}} \right]^2 \tag{14}$$

- for passive pressure

$$E\text{п} = \frac{F^{\text{ср}} \cdot \rho^{\text{ск}} g \cdot H^2}{2\cos\delta} \left[\frac{1}{\left(\frac{1}{\cos\varphi_1}\right) - \sqrt{tg^2\varphi_1 + tg\varphi_1 \cdot tg\delta}} \right]^2 \tag{15}$$

where δ is the angle of surface friction of the backfill material against the wall;
$\rho^{\text{ск}}$–is the density of the backfill material;
g–acceleration due to gravity;
H–is the height of retaining wall;
θ–is the angle of inclination of the AC plane;
$F^{\text{ср}}$–is the value of the friction F, taken by the mean value $\gamma_{\max}$ along the height of the retaining wall.

Thus, for engineering calculations of the total soil pressure on rigid retaining walls, depending on the displacement, one can use the well-known formulas (BCR=Building Codes and Regulations. 2.06.07-87.“Retaining walls, navigational locks, fish pass and fish protection structures”. Moscow 1989), if at an angle φ^*, we mean the angle of deflection φ_1 and introduce the function of lateral pressure $F(\gamma_{\max})$, and φ_1 and $F(\gamma_{\max})$, should depend on shear deformations.

To verify the proposed method, the active and passive pressures on a rigid retaining wall were calculated (see Fig. 10) with a height of H=20 m filled with Lyubertsy sand at two relative densities.

The influence of various factors (the density of the backfill soil, testing the soil trajectory to determine the function F, wall friction, taking into account the undercompacted layer of the backfill, etc.) is analyzed.

The proposed engineering method for calculating the lateral soil pressure on the retaining wall was used to determine the distribution of contact pressures along the height of the wall depending on its displacements. A method for determining the lateral pressure along the height of the wall has been developed; the transformation of the diagram of lateral pressure from the angle of rotation of the wall in the pre-limiting state is shown.

The results of the calculations performed, according to the proposed method were compared with the experimental data (see Fig. 11 a, b) (L.M. Emelyanov and others 1971).

The comparison indicates a good agreement between the results and the possibility of using the proposed method for determining the pressure on rigid retaining walls.

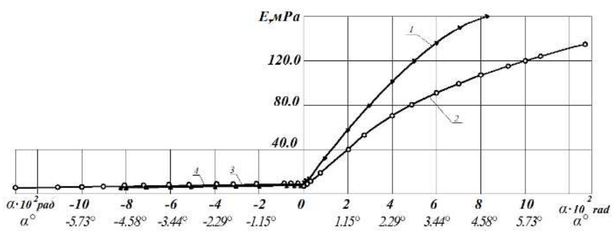

Figure 10. Full active dependence E_{acr}(3,4) and passive E_{pas}(1,2) pressures from α.

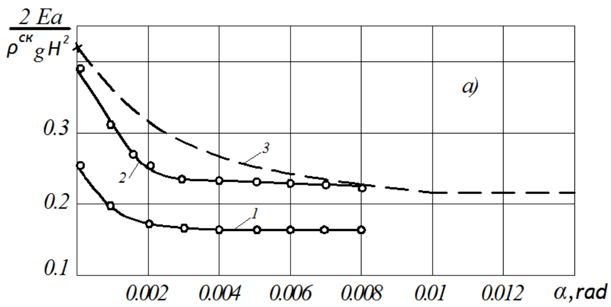

Figure 11 (a). Reduce values of active pressure $\frac{2Ea}{p^{ck} g H^2}$ from γ based on the experiments A.E.Emelyanov as calculated by the authors’ method

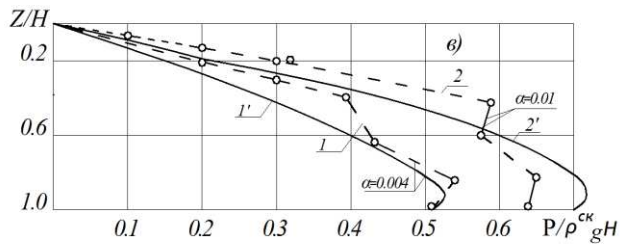

Figure 11 (b). Distribution of the reduced passive pressure $\frac{P}{p^{ck} g H}$ by the wall height Z/H from α (1,2….. based on the experiments of A.M. Emelyanov; 1; 2; as calculated by the authors’ method)

2 CONCLUSIONS

An engineering method has been developed for determining the soil pressure on a displaceable retaining wall. When turning it relative to the sole in the direction of the backfill and from it, it was established:

- active pressure is less than the resting pressure and the limiting state of the filling (when the wall deviates from the ground) occurs at small angles of rotation ($\Delta l / \text{H} \approx 0.01$);
- passive pressure formed when the wall tilts towards the backfill soil depends on the angle of inclination of the wall, and the limiting state is established at its significant slope equal to ($\Delta\ell/$H≈0.1). In real conditions, the value of passive Coulomb pressure is not realized;
- passive pressure, in contrast to active pressure, significantly depends on the density of the backfill soil both in the pre-limit and limit states;

- in the pre-limit state, the forecast of active and passive pressures depends on the loading path along which the backfill soil was tested. Comparison with the data of wall tests shows that the prediction of the value of passive pressure should be made using the results of studying the deformation and strength properties of soils, along the trajectories of "crushing" (σ_1=const; $\Delta\sigma_3$>0), and active pressure along the trajectories at which (σ_1 =const; $\Delta\sigma_3$<0);
- the diagrams of passive pressure in the pre-limiting state of the backfill soil are significantly differ from that determined by Coulomb, and in the lower part of the wall there is a decrease in pressure in comparison with the triangular one;
- the diagrams of active pressure in the pre-limit and limiting states of the backfill soil practically coincide with the diagram according to Coulomb;
- with an increase in the angle of rotation of the wall towards the backfill, the point of application of the resultant lateral soil pressure moves up.

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