частей техники, уменьшить сложность текущего ремонта, лучше использовать запасные части на ремонт машины, определять остаточный ресурс и, таким образом, уменьшить общую стоимость ремонта сельськохозяйственной техники, тракторов и автомобилей.

Ключевые слова: ***голография, техническое состояние, диагностирование, микродеформировиние, поверхность деталь, физические размеры***

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CALCULATION METHODS OF RETAINING WALLS

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Abstract. *Calculations of envelope structures of deep pits with developing of universal design models for soil mass in a contact with rigid elements of pits and foundations using methods of nonlinear theory of elasticity and plasticity and their computer realization is the actual modern problem of buildings and structures design.*

Calculations of envelope structures of deep pits with developing of universal design models for soil mass in a contact with rigid elements of closures of pits and foundations using methods of nonlinear theory of elasticity and plasticity and their computer realization is the actual modern problem of buildings and structures design.

The basis of introduced method is the generalization of dependence of soil mechanics for getting rules that allow more precisely know the meaning of deflected mode of closures of pits, bases and foundations of adjacent buildings depending on the heterogeneity of soil base. This method of determination of rated characteristics of soil base differs from others because it allows take into account not only its heterogeneity but also anisotropy of physical-mechanical characteristics of soils, separate elements and their replacement.

Fulfilled preproject researches of the interaction of space-enclosing structures of deep pits with soil half-space that include the bases and

foundations of existing buildings indicate that erection of new administrative building practically doesn't violate the conditions of the equilibrium of the ground and underground parts of existing nearby building and doesn't cause any considerable internal efforts in their structure.

Key words: ***sheet-pile retaining wall, calculation methods, soil, stability, earth thrust, building, foundations, displacement, strains, bearing capacity of soil***

Analysis of recent research results. The research connected with calculation of sheet-pile retaining wall was started from the analytical review of available literature. There were chosen several basic works, among which are: Rengach V. N. "Sheet-pile retaining walls. Calculation and Design" [1]; Tsytovich N. A., Ter-Martirosyan Z. H. "Basis of applied geomechanics in construction" [2]; Iaropolskiy I. V. "Basis and foundations" [3]; Harr M. E. "Basis of soil theoretical mechanics" [4].

Purpose of research. After the analysis we make important decisions connected with design model configuration and choosing the right way of loading implementation.

Results of research. Retaining wall is the structure that is intended for supporting the soil mass from avalanche when the slope gradient is greater than limit. Under construction there is often happens the situation when the natural slopes are cut, their stability is saved under gradient slope Ψ_0, that is named as natural gradient slope. New slope with angle Ψ that is greater than Ψ_0 cannot be stable and it will slough if will not be supported by the retaining wall (Fig. 1).

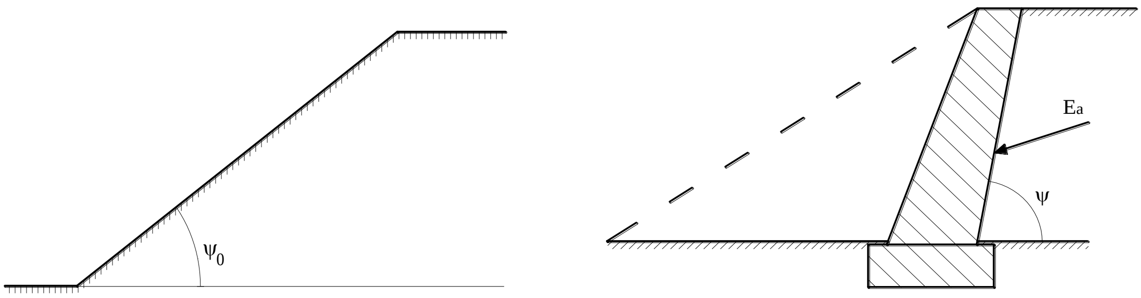


Fig. 1. Natural state of the soil and supported by the wall state.

In such case the retaining wall will be under the action of soil that is the result of soil weight and its dispersion. Retaining walls are subdivided into gravity, flexible and sheet-pile retaining walls.

The stability of gravity walls is provided by their own weight, and the stability of flexible walls – by the own weight and the soil weight that is lying on thin back slabs.

The stability of sheet-pile retaining walls are provided by binding to the soil mass in the combination with tension bars fixed to the anchor structure (pile) or by distance bars installation.

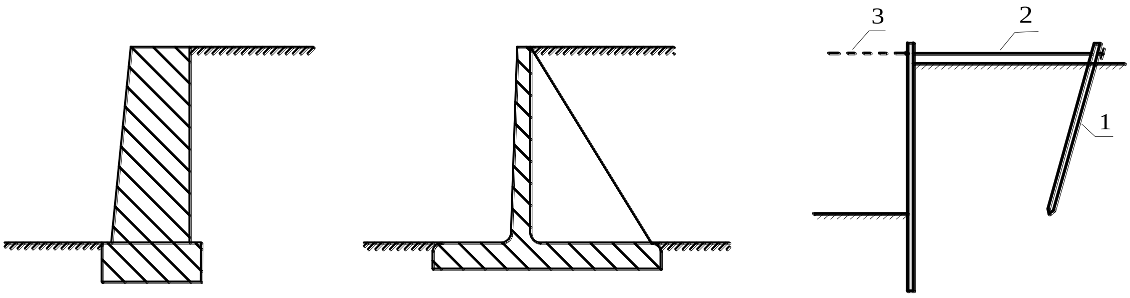


Fig. 2. Gravity, flexible and sheet-pile retaining walls: 1 – anchor pile, 2 – tension bar, 3 – distance bar.

Theoretical base for retaining wall calculation is the Coulomb hypothesis based on next statements:

1) In the backfilling soil under specific condition there is the prism of the sliding ABD, limited from the other soil that is in the before-limit state by the sliding plane AD (Fig. 3);

2) The inclination angle of the plane of sliding AD should be so that the active pressure E_a should be maximal;

3) Reaction R from the soil in the before-limit state is inclined from the normal to the plane of sliding AD on the angle of internal friction φ to the side opposite to the prism movement;

4) Force of active pressure E_a (reaction of the active pressure), acting on the back side of the wall AB inclines from the normal to the wall on the angle δ. δ is the friction angle of the soil and wall material. Sliding wedge is in the equilibrium under the action of forces G (dead weight), R and E_a.

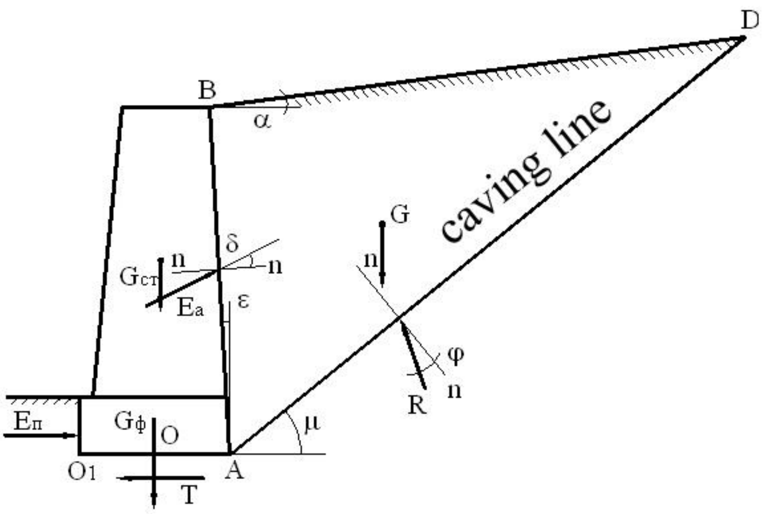


Fig. 3. Simplified design model of the slurry wall operation.

Brinch Hansen is the follower of the limit states theory proposed the method of calculation of flexible anchored walls in which he review the backfilling soil state and the limit state of the sheet-pile retaining wall including the appearance of plastic hinges in it. Special attention Hansen gave to the kinematic compatibility of soil and wall deformations.

According to the Hansen's method under the corresponding depths of the pointing chisel dipping and the dimensions of soil thrust in the sheet-pile wall there are possible only 1 or 2 plastic hinges, kinematic scheme of the sequence of their creation is given on fig. 4*, a, b, c*. On the Fig. 4, *d* there is the diagram of the soil pressure of the anchored metal wall under 2 plastic hinges presence. On these figures: F_T – friction force with the wall; *Qa* and *Qn* – correspondingly the resultant force of active and passive pressures of the soil on the wall; *M* – bending moment; *N* – longitudinal force in the wall; *s* – reaction of the soil from the wall weight.

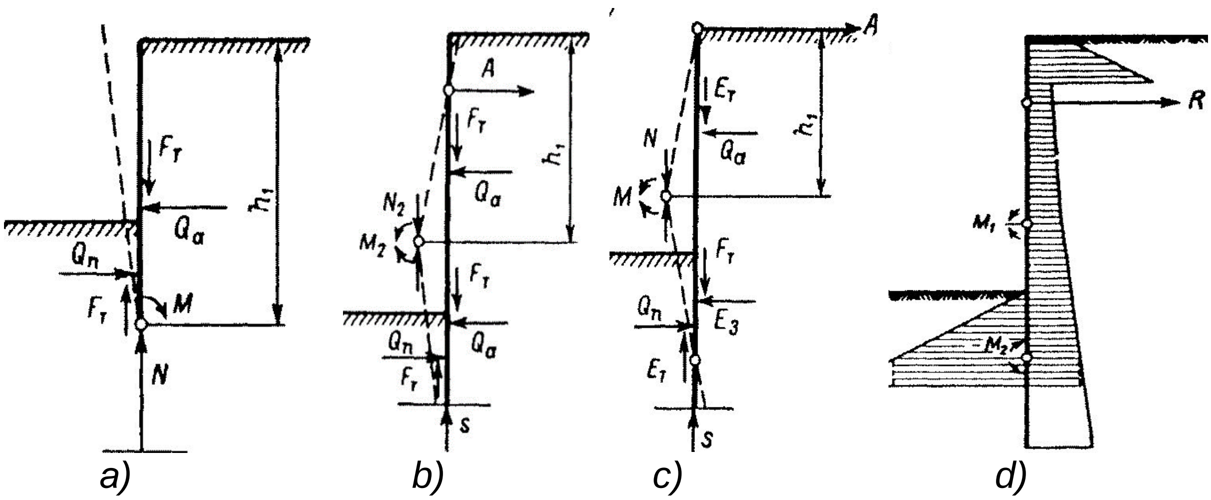


Fig. 4. Graphical interpretation of the loading process on the wall according to Hansen's theory: *a, b, c* – kinematic scheme of the sequence of the creation of plastic hinges; *d* – diagram of the loading of the soil to the wall under presence of vertical distributed loading (before creation of plastic hinge).

The order of the sheet-pile retaining wall: in the beginning we set the position of the anchoring tension bar and initially set the position of the plastic hinge on the distance h_1 from the top of the wall; later according to Hansen's tables we construct the diagram of soil pressure for the part of the wall with height h_1 that is rotating around the point of fixation of tension bar with wall. The ordinate of the diagram of active pressure near the bottom is set according to the table data for solid walls moved forward and connected by the straight line with the upper part of the diagram near the plastic hinge. Points of the diagram of passive pressure of the soil are also set according to the table data.

Calculation of retaining walls made of drilled piles is provided as for plane system. Loadings acting to the wall and the soil surface are brought to the considered row of piles for the multi-row variant or to the single pile for single-row variant.

For multi-row variant the height of the grillage h_p, m must be such that the condition will fulfill:

$$h_p \geq \frac{a}{4}, \tag{1}$$

where: a – the distance between axes of outside piles in the plane of load action, m.

The distance between piles is set in the dependence of the soil punching between piles:

$$b \leq 5{,}14\,\frac{c_I \cdot l_0 \cdot d}{E_a}, \tag{2}$$

where: b – the clearance between piles in the row, m; c_1 – design value of soil cohesion, kN/m^2; l_0 – height of the break, m; d – diameter of the pile, m; E_a – value of active soil pressure, kN/m.

Calculation of drilled piles in single-row variant in the retaining wall to the horizontal and moment loading are provided in accordance with next demands:

- value of proportionality coefficient K, kN/m^4, is taken in dependence from the soil type below the plane of break near the pile;
- conditional width of the pile B_0 m is taken not more than the distance between piles axes;
- calculation of the base stability, surrounding the pile, is taken under the coefficients value $\eta_1 \cdot \eta_2 = 0.8$;
- design values of horizontal displacement of the piles head, Δ_2, m, and angles of the piles rotation ψ, rad, should be taken in accordance with formulas:

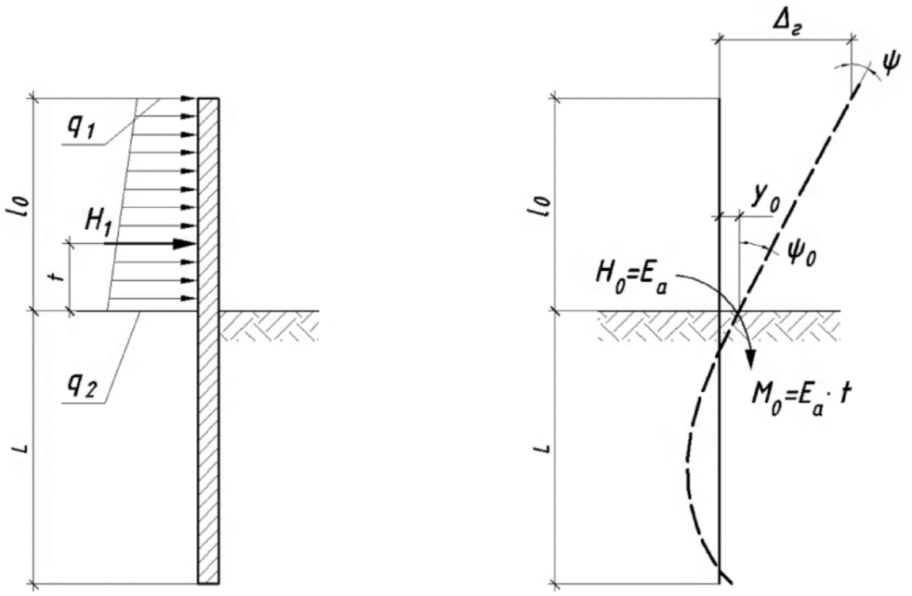
$$\Delta_2 = y_0 + \psi_0 \cdot l_0 + (11q_1 + 4q_2)\frac{l_0^4}{120 \cdot E_b \cdot I}, \tag{3}$$

$$\psi = \psi_0 + \frac{(3q_1 + q_2) \cdot l_0^3}{24 \cdot E_b \cdot I}, \tag{4}$$

where: y_0 and ψ_0 – design values correspondingly of horizontal displacement of the pile, m, and rotation angle, rad, in the level of plane of the break; l_0 – value of the pile break, m; q_1 and q_2 – distributed loading from the soil pressure, kN/m; E_c – initial elasticity module of the concrete of the pile body for compression, kN/m^2; I – inertia moment of the pile body cross-section, m^4.

Retaining walls must be calculated by two groups of limit states: first group (on stability of wall position against shift and on strength of structural elements); second group foresees the inspection of base for acceptable deformations and structural elements on acceptable meanings of cracks.

Influence of the retaining wall displacement on the lateral earth thrust. The main theory concerning the earth thrust belongs to Coulomb. But Coulomb didn’t achieve theoretical basis for determination of the earth thrust distribution along the wall. In his law he just supposed that that this distribution is quasi-hydrostatic and in accordance with it the resultant of the earth thrust is situated on the of 1/3 of the wall height from its basis. The results of experiments fulfilled by Terzaghi and Chebotarev proved Coulomb notion about the earth thrust for very stiff retaining walls with sand backfilling during their turn around the own foot.



a) *b)*

Fig. 5. Scheme of piles in single-row variant of the retaining wall.

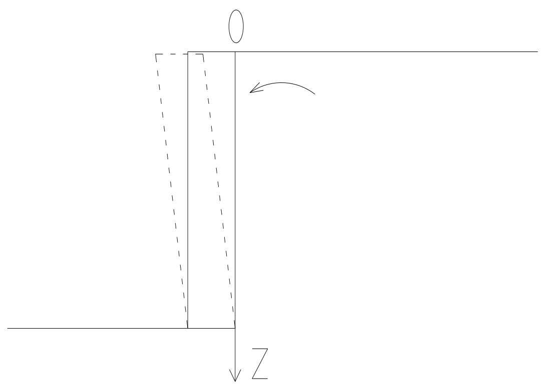
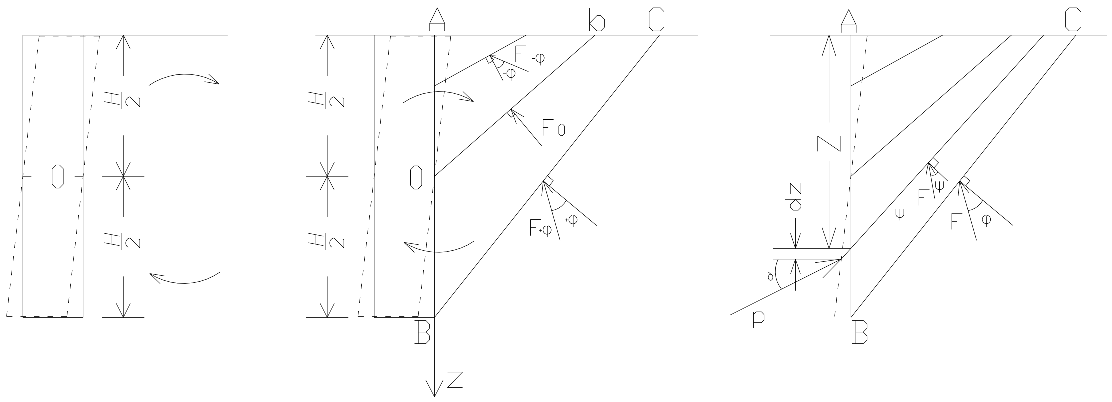


Fig. 6. Turn of retaining wall around the foot out of the backfilling.

Under other kinds of displacements such as turn around the top or the center of the wall or translation the experiments showed that the thrust distribution has the form close to the parabolic. Though the mechanism causing such differences hasn’t fully estimated. In accordance with fulfilled experiments the difference between distribution of thrust in stiff retaining structures and flexible walls is the function of displacements depending on the height position in the structure.

Let's consider comparatively little known method of determination of thrust distribution that have significant advantages. This method was published in 1963 by G. A. Dubrova and has the name of "method of thrust distribution". On the figure 7. (*a*) it is shown how the stiff wall turns around the center. On the figure 7. (*b*) it is shown the mechanic scheme of soil interaction with the wall that was presented by Dubrova. It is evident that when the upper part of the wall AO press on the soil the backfilling situated below the line Ob passively presses on the wall. By the analogy, the lower part of the wall under the turn out from the soil will perceive active press from it. The distribution of such thrust along the wall is not known. To avoid this problem Dubrova concede that in such case the limit passive state exists only near the top of the wall, but the limit active state – only near the bottom of the wall and both these states exist simultaneously.



a) *b)* *c)*

Fig. 7. Earth thrust of the stiff wall with turn around the center: *a* – displacement scheme of the wall; *b* – interaction between the soil and the displacing wall; *c* – scheme to the formula (5).

So the resultant force F along the line damage line BC, crossing through the bottom of the wall will be inclined on $+\varphi$ angle to the normal, when the resultant force F along the damage line for limit passive state crossing through the point A (on fig. 7 – *b*, this line displaced below) make with normal angle $-\varphi$. Between these limit states there is supposed to exist infinite number of destruction lines with angles of the inclination of resultant forces F concerning normal to these lines that are changed from $-\varphi$ to $+\varphi$*.* Designating the angle between force and normal to each line via ψ, Dubrova supposes that changing of this angle with sweep *z* (vertical distance from the top of the wall to the point in which the destruction line intersects with the wall) is the liniar (Fig. 7, *c*) or:

$$\psi = \frac{2\varphi z}{H} - \varphi. \quad (5)$$

Resultant force F_0 along the line of destruction 0b will be perpendicular to it. In the basis of this assumption there is observation that proved the degree of mobilization of soil strength depends directly from the allowed displacement of the wall. So in the *0* point the displacement is equal zero than $\varphi=0$ along the Ob line and the resultant force F_0 will be perpendicular to the line of destruction.

Further Dubrova accept the assumption that the correctness of the Coulomb's decision. Than the angle between the line of destruction and the horizontal under each z will be equal:

$$\theta = \frac{\pi}{4} + \frac{\psi}{2} = \frac{\pi}{4} - \frac{\varphi}{2} + \frac{\varphi z}{H}. \quad (6)$$

This definition allows connect the consideration of passive and active thrusts in the single expression. Fulfilling this substitution, we receive under each z:

$$P = \frac{\gamma}{2cos\delta}\left[\frac{z}{\left(\frac{1}{cos\psi}\right) + \sqrt{tg^2\psi + tg\psi tg\delta}}\right]^2. \quad (7)$$

To determine the distribution of the thrust along the wall we determine the derivative from the equation (7) on z and have:

$$p(z) = \frac{dP}{dz} = \frac{\gamma}{cos\delta}\left[\frac{zcos^2\psi}{(1+msin\psi)^2} - \frac{2z^2\varphi cos\psi}{H(1+msin\psi)^3}\left(sin\psi + \frac{1+m^2}{2m}\right)\right], \quad (8)$$

where: $m = [1 + \left(\frac{tg\delta}{tg\psi}\right)]^{1/2}$.

For the particular case when the surface friction can be neglected, $\delta=0$ and the equation can be expressed in the view of:

$$p(z)_0 = \gamma tg^2\left(45 - \frac{\psi}{2}\right)\left(z - \frac{2z^2\varphi}{Hcos\psi}\right). \quad (9)$$

Dubrova simplify further the equation (8) taking m as a constant. It means that the surface friction along the wall is considered as the function of the soil strength and therefore the constant value of φ but not the function of the angle ψ of the destruction plane. It shows that it is possible without large error to accept

$$\frac{1+m^2}{2m} = m. \quad (8)$$

So when $\delta=0$ the value m=1 and

$$(1+m^2)/2m=1. \quad (9)$$

than the equation will be independent from these assumptions.

Transforming the formula Dubrova gets:

$$p(z) = \frac{\gamma}{cos\delta}\left[\frac{zcos^2\psi}{(1+msin\psi)^2} - \frac{2z^2\varphi cos\psi}{H(1+msin\psi)^3}(sin\psi + m)\right], \quad (10)$$

where: $m=[1+(tg\delta/tg\varphi)]^{1/2}$.

Conclusion that leads to equations (10) and (9) is the basis of the Dubrova method. So it is possible to determine $p(z)$ for each turn of the wall if ψ is given.

Evenly distributed along the surface of the backfilling loading with intensity *q* in accordance with considered earlier assumptions can be regarded as the imaginary soil layer with height *h=q/γ*. For smooth vertical wall (*δ=0*) with horizontal backfilling under the evenly distributed loading q the loading formula will have such expression:

$$P_q = \left(\frac{\gamma H^2}{2} + Hq\right) tg^2\left(45 \pm \frac{\varphi}{2}\right), \qquad (11)$$

where sign plus is taken for passive pressure, sign minus – to active one.

For mentioned above assumptions we receive next thrust distribution (Coulomb's law, when δ=0).

$$p(z)_0 = tg^2\left(45 - \frac{\psi}{2}\right)\left(\gamma z + q - \frac{2\gamma z^2\varphi + 2q\varphi z}{H cos\psi}\right), \qquad (12)$$

where: $\psi = (2\varphi z/H) - \varphi$.

Let's calculate the equivalent soil layer for substitution of the loading from the existing building with this extra soil layer. Proportionally to the thickness calculate the average value of the angle φ° and specific weight γ:

$$\varphi^\circ = \frac{23 \cdot 10 + 18{,}5 \cdot 8 + 20 \cdot 3 + 19 \cdot 4{,}5 + 12 \cdot 3{,}0 + 9 \cdot 12}{10 + 8 + 3 + 4{,}5 + 3 + 12{,}0} =$$

$$= \frac{667{,}5}{40{,}5} = 16{,}5^\circ$$

$$\gamma = \frac{0{,}00162 \cdot 10 + 0{,}001885 \cdot 8 + 0{,}00182 \cdot 3 + 0{,}00185 \cdot 4{,}5}{40{,}5} +$$

$$+ \frac{0{,}00199 \cdot 3 + 0{,}002 \cdot 12}{40{,}5} = \frac{0{,}075035}{40{,}5} = 0{,}00185\frac{kg}{cm^3} = 1{,}85\frac{t}{m^3}.$$

Distributed load from the existing building can be calculated as the total weight divided on its total length in the plane view of the problem:

$$q = \frac{35{,}5 \cdot 10}{63{,}8} = 5{,}66\frac{t}{m^2}.$$

The additional soil layer height in such case will be:

$$H = \frac{P}{\gamma} = \frac{5{,}56}{1{,}85} = 3{,}0\ m.$$

The load distribution according to Coulomb:

$$\frac{P_k}{\gamma} = H\left[\frac{1}{(1/cos\varphi) + tg\varphi}\right]^2 = 31{,}5\left[\frac{1}{(1/cos16{,}5) + tg16{,}5}\right]^2 = 31{,}5 \cdot 0{,}55$$

$$= 17{,}569m.$$

Coulomb's decision:

$$\frac{17{,}569 \cdot 31{,}5}{2} = 276{,}71m^2.$$

Solution of nonlinear problem of limit equilibrium of soil masses under interaction with envelope structures by numerical methods of finite elements. Calculations of envelope structures of deep pits with developing of universal design models for soil mass in a contact with rigid elements of closures of pits and foundations using methods of nonlinear theory of elasticity and plasticity and their computer realization is the actual modern problem of buildings and structures design.

1. Thrust distribution along the wall according to Dubrova's method and from numerical method of finite elements.

z, m	Ψ, °	tg (45-$\frac{\psi}{2}$)	cosψ	$\frac{2z^2\varphi}{H \cdot cos\psi}$	$P(z)/\gamma$, m (without load)	$P(k)/\gamma$, m (with load)	Numerical calculation, m
3,0	-13,36	1,6	0,9729	0,17	4,528	9,06	5,03
6,0	-10,22	1,43	0,984133	0,658	7,64	11,44	9,15
9,0	-7,08	1,28	0,99237	1,47	9,64	12,81	10,42
12,0	-3,94	1,14	0,997636	2,60	10,76	13,34	10,74
15,0	-0,8	1,028	0,9999	4,05	11,258	13,43	10,07
18,0	2,34	0,92	0,999914	5,89	11,16	12,95	9,22
21,0	5,48	0,82	0,99538	7,97	10,78	12,08	8,01
24,0	8,62	0,74	0,98862	10,49	9,99	11,11	6,74
27,0	11,76	0,65	0,97890	13,40	8,92	9,69	5,46
30,0	14,90	0,57	0,9662	16,76	7,64	8,2	4,27
31,5	16,47	0,558	0,95897	18,6	7,19	7,7	3,35

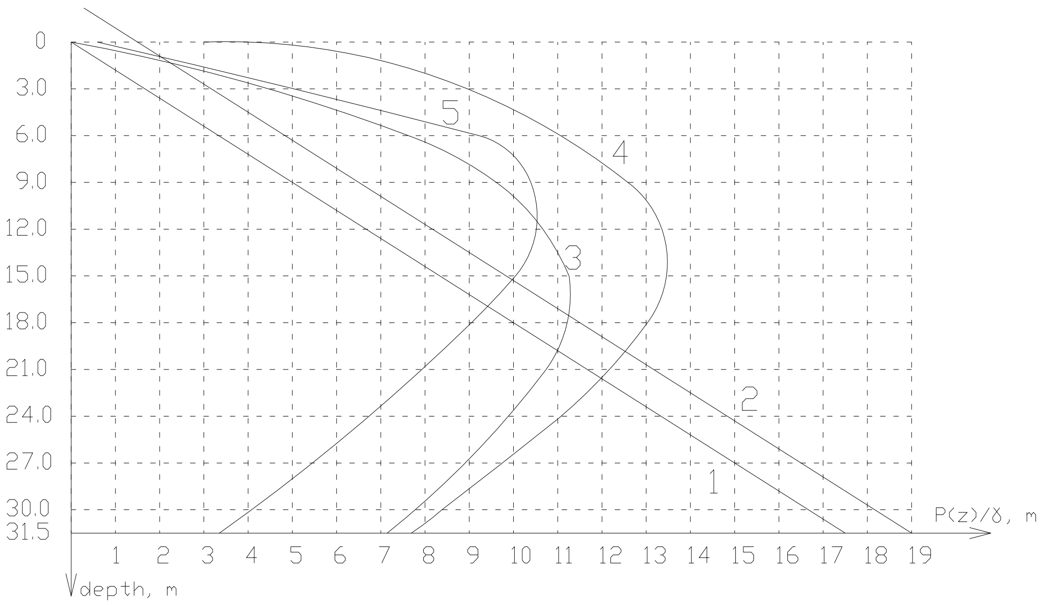


Fig. 8. Earth thrust diagrams. 1 – Coulomb's assumptions (without *q*); 2 – Coulomb's assumptions (with *q*); 3 – Dubrova's assumptions (without *q*); 4 – Dubrova's assumptions (with *q*); 5 – Finite elements method results (with *q*).

The basis of introduced method is the generalization of dependence of soil mechanics for getting rules that allow more precisely know the meaning of deflected mode of closures of pits, bases and foundations of adjacent buildings depending on the heterogeneity of soil base. This method of determination of rated characteristics of soil base differs from others because it allows take into account not only its heterogeneity but also anisotropy of physical-mechanical characteristics of soils, separate elements and their replacement [5, 6].

Comparative results are shown on Fig. 8.

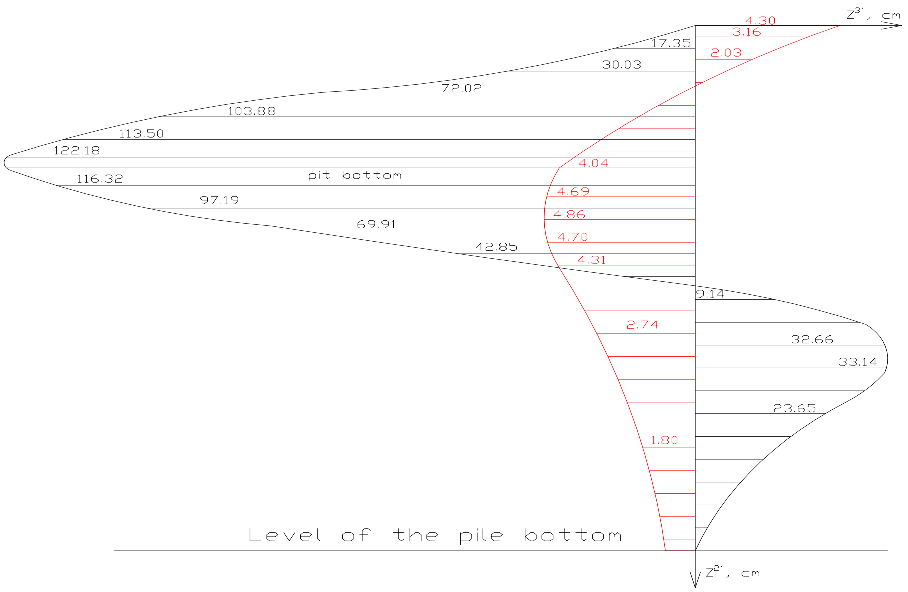


Fig. 9. Bending moments and displacements diagram in the cross-section of retaining wall according to calculation by finite elements method.

Calculation on the strength of structural element of the sheet pile wall using the numerical calculation results of the deflected mode.

Data:
Pile's diameter – 820 mm.
Concrete class C25/30.
Protection layer – a=30 mm.

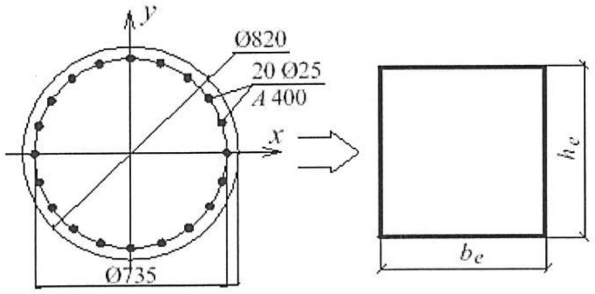


Fig. 10. Scheme of bringing the real pile cross-section to equivalent one. Accept h_e=b_e.

1) Determination of the area and thickness of equivalent reinforcement ring

$$\sum F_{\varnothing 25} = 20 \cdot 4{,}91 = 98{,}2\ cm^2$$

$$\sum F_{\varnothing 25} = \pi \cdot d_k \cdot \delta_k = 98{,}2\ cm^2 \tag{13}$$

$$\delta_k = \frac{\sum F_{\varnothing 25}}{\pi d_k} = 0{,}4253\ cm \tag{14}$$

2) Determination of parameters of equivalent rectangular cross-section of the pile is fulfilled through the system of 2 equations:

$$\begin{cases} E_e \frac{b_e h_e^3}{12} = E_g I_g; \\ E_e b_e h_e = E_g A_g. \end{cases} \tag{15}$$

Substitution of the system (15) gives: $\frac{h_e^2 E_g A_g}{12} = E_g I_g$; $h_e = \sqrt{\frac{12 E_g I_g}{E_g A_g}}$;

$$E_g I_g = E_\delta I_\delta + E_{\text{ст}} I_{\text{ст}}; \quad E_g A_g = E_\delta A_\delta + E_{\text{ст}} F_{\text{ст}};$$

$E_\delta = 3{,}31 \cdot 10^5\ kgf/cm^2$, $E_{\text{ст}} = 2{,}1 \cdot 10^6 kgf/cm^2$.

$F_{\text{ст}} = 98{,}2\ cm^2$; $A_\delta = \frac{\pi D^2}{4} - 98{,}2 = \frac{\pi 82^2}{4} - 98{,}2 = 5182{,}81\ cm^2$.

$I_{conc}^{x-x} = \frac{\pi \cdot D^4}{64} - I_{st}^k$; $I_{st}^k = 0{,}3926 d_k^3 \delta = 6{,}62991 \cdot 10^4\ cm^4$;

$I_\delta^{x-x} = \frac{\pi \cdot 82^4}{64} - 66299{,}11 = 2{,}1530465 \cdot 10^6\ cm^4$;

$E_g I_g = 3{,}3 \cdot 10^5 \cdot 2{,}1530465 \cdot 10^6 + 2{,}1 \cdot 10^6 \cdot 6{,}62991 \cdot 10^4 =$
$= 8{,}51886 \cdot 10^{11} kgcm^2$;

$E_g A_g = 5182{,}81 \cdot 3{,}31 \cdot 10^5 + 2{,}1 \cdot 10^6 \cdot 98{,}2 = 1{,}92173 \cdot 10^9 kgf$;

$$h_e = \sqrt{\frac{12 \cdot 8{,}51886 \cdot 10^{11}}{1{,}92173 \cdot 10^9}} = 72{,}94\ cm.$$

Accept h_e=96,0 cm.

$$E_e = \frac{E_g A_g}{h_e^2} = \frac{1{,}92173 \cdot 10^9}{96^2} = 2{,}09 \cdot 10^5 \frac{kgf}{cm^2}.$$

For the considered cross-section we have next geometric and physical-mechanical characteristics:

- Area of the reinforcement: F_{st}=98,2 cm^2.
- Area of the concrete: F_{conc}=5182,81 cm^2.
- Inertia moment around *x-x* of the reinforcement: I_{st}=6,62991·10^4 cm^4.
- Inertia moment around *x-x* of the concrete: I_{conc}=2,1530465·10^6 cm^4.

Moment resistance of the reinforcement:

$$W_{st} = \frac{6{,}62991 \cdot 10^4}{36{,}75} = 1804{,}05\ cm^3$$

Moment resistance of the concrete:

$$\mathrm{W_c} = \frac{2{,}1530465 \cdot 10^6}{41} = 52513{,}33\ \mathrm{cm}^3.$$

Concede that strains in the cross-section of the steel concrete structure are redistributed between steel and concrete elements proportionally to the correspondent stiffness, it means that parts of whole strains from the longitudinal forces are distributed proportionally to longitudinal stiffness, and parts of bending moments distribute proportionally to bending stiffness. Determine the value of redistribution of longitudinal forces and bending moments:

$$M_{st} = \frac{M_{max} \cdot E_{st} \cdot I_{st}}{EI} = \frac{1221{,}8 \cdot 100 \cdot 2{,}1 \cdot 10^6 \cdot 6{,}62991 \cdot 10^4}{8{,}51886 \cdot 10^{11}} = \\ = 199{,}68\ kN \cdot m.$$

$$M_{conc} = \frac{1221{,}8 \cdot 3.31 \cdot 10^5 \cdot 2{,}1530465 \cdot 10^6}{8{,}51886 \cdot 10^{11}} = 1022{,}12\ kN \cdot m.$$

Check: 199,68 + 1022,12 = 1221,8 kN·m.

$$N_{st} = \frac{N_{max} \cdot E_{st} \cdot F_{st}}{EF} = \frac{1055{,}2 \cdot 2.1 \cdot 10^6 \cdot 98{,}2}{1{,}92173 \cdot 10^9} = 113{,}23\ kN.$$

$$N_{conc} = \frac{N_{max} \cdot E_{conc} \cdot F_{conc}}{EF} = \frac{1055{,}2 \cdot 3{,}31 \cdot 10^5 \cdot 5182{,}81}{1{,}92173 \cdot 10^9} = 941{,}96\ kN.$$

$$\sigma^{\mathrm{st.}}_{\substack{\mathrm{max} \\ \mathrm{min}}} = \frac{\mathrm{N_{st}}}{\mathrm{F_{st}}} \pm \frac{\mathrm{M_{st}}}{\mathrm{W_{st}}} = \frac{113{,}23}{98{,}2} \pm \frac{199{,}68 \cdot 100}{1804{,}05} = 122{,}2\ \mathrm{MPa} \le \mathrm{R_y}\gamma_{\mathrm{c}} \\ = 240\ \mathrm{MPa}.$$

$R_y \gamma_c = 240\ MPa$ (for steel A240C).

$$\sigma^{conc.}_{\substack{max \\ min}} = \frac{N_{conc}}{F_{conc}} \pm \frac{M_{conc}}{W_{conc}} = \frac{941{,}96}{5182{,}81} + \frac{1022.12 \cdot 100}{52513{,}33} = 2{,}126\ \frac{kN}{cm^2} = 21{,}26 \\ \le (21{,}4\text{МПPa}).$$

Design resistance of the concrete of class C25/30 on the second group of limit states is 22,4 MPa, so the strength is provided.

Conclusions. Structure of the pilling wall of the pit is accepted in the “slurry wall” type that is made (according to the design) of injected piles of diameter 820 mm of concrete class C25/30 with longitudinal reinforcement rods – 24Ø25 mm A240C (instead of initially accepted A400), and under this there is provided piles lowering to the relative depth of 31,35 m at absolute values of the up – 186,5 m and down – 155,15 m on the whole perimeter of the pit in the accordance with the plan.

Fulfilled preproject researches of the interaction of space-enclosing structures of deep pits with soil half-space that include the bases and foundations of existing buildings indicate that erection of new administrative building practically doesn’t violate the conditions of the equilibrium of the ground and underground parts of existing nearby building and doesn’t cause any considerable internal efforts in their structure.

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МЕТОДИ РОЗРАХУНКУ ПІДПІРНИХ СТІН

Є. А. Бакулін, В. М. Бакуліна, Н. О. Костира

Анотація. *Розрахунки огороджувальних конструкцій глибоких котлованів з побудовою універсальних розрахункових моделей для ґрунтового масиву при контакті з жорсткими елементами огорож котлованів і фундаментів із застосуванням методів нелінійної теорії пружності і пластичності та їх комп'ютерна реалізація є актуальною проблемою сьогоденного проектування будівель та споруд.*

В основу запропонованої методики покладено узагальнення залежностей механіки ґрунтів для отримання закономірностей, що дозволяють більш обґрунтовано визначати величину напружено-деформованого стану огорож котлованів, основ та фундаментів прилеглої забудови залежно від неоднорідності ґрунтової основи. Даний підхід визначення розрахункових характеристик ґрунтової основи відрізняється тим, що дозволяє враховувати не тільки її неоднорідність, але й анізотропність фізико-механічних характеристик ґрунтів, окремих елементів і їх зміну.

У розрахунковій схемі задачі передбачається дискретне моделювання плоского ґрунтового неоднорідного (багатошарового) півпростору з наявністю порожнин (котлованів новобудов, підземних приміщень існуючої забудови) і включень (елементів огороджувальних конструкцій, захисних екранів, фундаментів прилеглих будівель і споруд).

Виконані передпроектні дослідження взаємодії огороджувальних конструкцій глибоких котлованів з ґрунтовим півпростором і ґрунтовими основами і фундаментами існуючих будівель засвідчують, що будівництво нової будівлі практично не порушує умови рівноваги надземної і підземної частини існуючих будівель і не викликає значних додаткових внутрішніх зусиль в конструкціях цих будівель.

Ключові слова: ***конструкція «стіна у ґрунті», методи розрахунку, ґрунт, стійкість, тиск ґрунту, будівля, фундаменти, переміщення, деформації, несуча спроможність ґрунту***

МЕТОДЫ РАСЧЕТА ПОДПОРНЫХ СТЕН

Е. А. Бакулин, В. М. Бакулина, Н. А. Костыра

Аннотация. *Расчёты ограждающих конструкций глубоких котлованов с построением универсальных расчетных моделей для грунтового массива при контакте с жёсткими элементами ограждений котлованов и фундаментов с использованием методов нелинейной теории упругости и пластичности их компьютерная реализация есть актуальной проблемой сегодняшнего проектирования зданий и сооружений.*

В основе предложенной методики заложено обобщение зависимостей механики грунтов для получения закономерностей, которые позволяют более обосновано определить величину напряжённо-деформированного состояния ограждений котлованов, оснований и фундаментов прилегающей застройки в зависимости от неоднородности грунтового основания. Данный подход определения расчетных характеристик грунтового основания отличается тем, что позволяет учитывать не только неоднородность основания, а и анализировать физико-механические характеристики грунтов отдельных элементов и их замену.

В расчетной схеме задачи предусматривается дискретное моделирование плоского грунтового неоднородного (многослойного) полупространства с явными пустотами (котлованов новостроек, подзем-ных помещений существующей застройки) и включений (элементов ограждающих конструкций, защитных экранов, фундаментов прилегающих зданий и сооружений).

Проведенные предпроектные исследования взаимодействия огражда-ющих конструкций глубоких котлованов с грунтовым полупространством и грунтовыми основаниями и фундаментами существующих зданий свидетельствуют о том, что строительство нового здания практически не нарушает условия

равновесия надземной и подземной части существующих зданий и не вызывает значительных дополнительных внутренних усилий в конструкциях этих зданий.

Ключевые слова: ***конструкция «стена в грунте», методы расчета, грунт, устойчивость, давление грунта, здание, фундаменты, перемещения, деформации, несущая способность грунта.***

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ЕКОЛОГІЧНІ ТА ЕКОНОМІЧНІ ПРОБЛЕМИ ЗБИРАЛЬНО-ТРАНСПОРТНИХ ПРОЦЕСІВ ДЛЯ КОРЕНЕПЛОДІВ ЦУКРОВИХ БУРЯКІВ

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Анотація. *Визначені підходи та шляхи вирішення екологічних та економічних проблем під час збирання та транспортування коренеплодів цукрових буряків. Екологія та економіка в сільськогосподарському виробництві знаходяться у тісному зв'язку, їх вплив друг на друга має довгостроковий характер. Для забезпечення окупності імпортних машин необхідно раціональне збільшення виробничих обсягів на кожний збирально-транспортний комплекс.*

Для підвищення ефективності технологій перевезення цукрових буряків з поля з урахуванням екологічності виробничих процесів необхідно вирішувати підбір раціонального збирально-транспортного комплексу (ЗТК) з метою якісного очищення коренеплодів, а також адаптувати параметри логістичного ланцюга для цукрових буряків та обгрунтувати раціональну технологію транспортування продукції із заміною спеціалізованих засобів на універсальні. Для поточної технології доцільним є транспортування продукції із використанням напівпричепів-самоскидів (НП) як оборотних засобів. Швидкі зміни погодних умов: осінні дощі та перезволожений грунт потребують відповідної адаптації технології та технічних засобів з переходом на перевалочний варіант із застосуванням потужного навантажувача-очищувача коренеплодів. Заміна спеціалізованих