23. Jodejko-Pietruczuk A., Nowakowski T., Werbińska-Wojciechowska S. Block inspection policy model with imperfect inspections for multi-unit systems // RT&A. 2013. Vol. 8, Issue 3 (30). P. 75–86.
24. Prognozirovanie rabotosposobnosti listoprokatnyh valkov / Belodedenko S. V., Goryanoy V. M., Buh I., Yatsuba A. V. // Problemy prochnosti. 2014. Issue 5. P. 89–95.
25. Datsyshyn O. P., Panasyuk V. V. Methods for the Evaluation of the Contact Durability of Elements of the Tribojoints (A Survey) // Materials Science. 2017. Vol. 52, Issue 4. P. 447–459. doi: 10.1007/s11003-017-9977-x

Розроблено метод розрахунку бокового тиску грунту на шпунтову стінку з контрфорсами різної форми – прямокутної, трапецеїдальної з розширенням донизу, трапецеїдальної з розширенням догори. Проведено математичне моделювання системи «шпунтова стінка з контрфорсами – грунтове середовище». Досліджено епюри бічного тиску грунту на шпунтові стінки з контрфорсами. Отримано кількісна оцінка розвантажуючої дії контрфорсів різної форми

Ключові слова: метод розрахунку, шпунтова стінка, контрфорси, боковий тиск грунту, розвантажувальний вплив

Разработан метод расчета бокового давления грунта на шпунтовую стенку с контрфорсами различной формы – прямоугольной, трапецеидальной с расширением книзу, трапецеидальной с расширением кверху. Проведено математическое моделирование системы «шпунтовая стенка с контрфорсами – грунтовая среда». Исследованы эпюры бокового давления грунта на шпунтовую стенку с контрфорсами. Получена количественная оценка разгружающего действия контрфорсов различной формы

Ключевые слова: метод расчета, шпунтовая стенка, контрфорсы, боковое давление грунта, разгружающее влияние

UDC 627.335.2
DOI: 10.15587/1729-4061.2018.121707

A METHOD DEVELOPED TO CALCULATE LATERAL EARTH PRESSURE ON A SHEET PILE WALL WITH COUNTERFORTS

A. Slobodyanik
PhD, Assistant
Department of sea and river ports, waterways and their technical maintenance
Odessa National Maritime University
Mechnikova str., 34, Odessa, Ukraine, 65029
E-mail: annaslobodyanik27@gmail.com

1. Introduction

The development of the ports of Ukraine requires the construction of deepwater berthing facilities for servicing modern large-tonnage vessels. The existing construction solutions for deepwater berths are labor-intensive and material-intensive if they require, for example, complex tonguing and grooving [1, 2] or using transverse rows of sheet piles [3]. Therefore, it is necessary to develop and implement innovative design solutions in hydraulic engineering. Nowadays, the most rapidly constructed structures are sheet pile walls [4], so the creation of new design solutions with sheet piles is important. One of the proposed solutions is a sheet pile wall with counterforts [5], which has received a patent for the invention itself [6] and a utility patent for the construction method [7]. Counterforts contribute to a significant reduction of lateral earth pressure on the front wall and a rational distribution of material in the construction. However, the use of a new design in practice necessitates the development of a method for calculating the lateral earth pressure, taking into account the relief effect of the counterforts. It is also necessary to conduct research on the stress-strain state of the system "a sheet pile wall with counterforts plus the soil environment". The solution of the task is an important link in the study of the new type of construction, which will allow introducing it into engineering practice.

2. Literature review and and problem statement

The main load on retaining walls is produced by lateral earth pressure. To study the joint effect of the soil backfill and the construction, numerical calculation models [8] are developed and approaches for optimal design are sought [9]. In order to reduce earth pressure on the retaining wall, various relief devices are offered, such as horizontal shelves located on the backfill side [10, 11] or a relief platform [12].

Counterforts are also one of the types of relief elements in the structure. Calculation methods for defining the relief influence of counterforts are based on theoretical, laboratory and field studies. Moreover, a method has been proposed for calculating the screening effect of counterforts, which provides for correcting the active earth pressure using empirical dependence obtained on the basis of tests [13]:

$$\sigma = (\gamma z + q)\lambda_a(1-k), \tag{1}$$

where k is the coefficient of decrease in the intensity of the active earth pressure:

$$k = 1 - \left(0.74\frac{b}{c} - m\right)\frac{z}{H}, \qquad (2)$$

where H is the height of the counterfort, m; b is the counterfort width, m; c is the distance between the counterforts, m; m is the empirical coefficient, which is determined depending on the ratio $\frac{d}{c}$, where d is the thickness of the counterfort, m; and z is the point penetration relative to the backfill surface, m.

The resulting theoretical solution [14] involves calculating the pressure on the "separating" slip plane formed in the backfill at the wall displacements and the estimation of the distribution of the horizontal component of the initial pressure between the front wall and the counterforts. The calculation is based on assuming the formation of shifts in the soil along the planes, coinciding with the faces of the counterforts. In order to determine the earth pressure, the following dependence was obtained:

$$\sigma = \sigma_a \exp\left[-\sigma_a \frac{\xi tg\phi}{\upsilon_{kp} E(c-b)}\left(2At + \exp[-\sigma_a \frac{2At\xi tg\phi}{\upsilon_{kp} E(c-b)}]t^2\right)\right], \qquad (3)$$

where σ_a is the active earth pressure on the "separating" plane, kPa; ξ is the coefficient of lateral earth pressure in the absence of transverse expansions; A and t are the geometric parameters of the horizontal section of the prism that moves along with the structure, m; E is the modulus of the soil deformation, MPa; υ_{kp} is the shear angle corresponding to the limiting value of the deflection angle $\delta=\varphi$, deg.

The method that assumes a non-linear distribution of the earth pressure on the retaining wall with vertical [15] and inclined [16] counterforts is applicable for calculating the relief effect of reinforced concrete counterforts.

The effect of counterforts on the lateral earth pressure was investigated in laboratory experiments [17]. A calculated dependence was proposed, the drawback of which is the assumption of the presence of active earth pressure along the entire perimeter of the cell. In [18], on the basis of laboratory experiments, a case was considered when the length of the counterfort covered the entire collapse prism, partially entering the fixed part of the soil backfill. Also, the same author examined the earth pressures on the retaining walls with counterforts depending on the movements [19]. However, such constructions do not always find practical application.

In full-scale conditions, the relief effect of counterforts was studied in the port of Hakata (Japan) on a corner-type embankment with counterforts [20]. It is known from the report that the pressure on the front wall and the side faces of the counterforts was distributed according to a linear law. At the same time, the concentration of pressure at the ends of the counterforts was recorded due to the inclination of the wall towards the backfill.

RD 31.31.27–81 "Guidelines for the design of marine berthing facilities" [21] recommends a degree of 20÷30 % of the relief influence of counterforts. This approach is not entirely accurate due to the fact that it was developed for one type of configuration of counterforts used in the construction of mooring structures such as a corner wall.

The existing methods for calculating the relief effect of counterforts, which were developed earlier, are very approximate when calculating a new design, since they were developed for reinforced concrete corner walls. Nevertheless, when calculating lateral earth pressure on an innovative sheet pile wall with counterforts, it seems possible to extend the scope of V. S. Zelensky's method. Such an approach seems to be fully justified if the diagram of lateral earth pressure is considered in parts in terms of height, with determining the degree of influence of the counterforts on the front wall for each part.

3. The aim and objectives of the study

The aim of this work is to develop a method for calculating the relief effect of counterforts and to research the stress-strain state of the system "a sheet pile wall plus the soil environment". The results are likely to help introduce a new constructive solution into the practice of designing and constructing hydraulic structures and to facilitate improving the quality of future projects and the reliability of the constructed facilities in the process of their operation.

To achieve the aim, the tasks are the following:

– to improve the method for calculating the lateral earth pressure, taking into account the relief effect of the counterforts in the sheet pile wall for various forms of counterforts;

– to carry out numerical modeling of the system "the soil environment plus a sheet pile wall with counterforts";

– to assess the stress-strain state of the system "the soil environment plus a sheet pile wall with counterforts".

4. Materials and methods for studying the distribution of lateral earth pressure on a sheet pile wall with counterforts

The scope of V. S. Zelensky's method was extended for the case of calculating earth pressure on sheet pile walls with different configuration of counterforts (with downward expansion and with upward expansion). This did not take into account the phenomenon of "hanging" soil over the counterforts of variable height. In the rear part of the wall, the soil was considered as inhomogeneous.

Earlier, in [22], only the section with a counterfort was considered. In the present work, in the design scheme, two characteristic sections are identified throughout the entire height of the face wall. The first section is the construction with a counterfort, located within the prism of collapse, and the second section is without a counterfort. The boundaries of the upper section are $0 \le z \le d$, and the boundaries of the lower section are $d \le z \le H$, where d is the height of the counterfort, and H is the height of the wall. In this case, $b_{1,2}$ are the widths of the counterfort at the top and the bottom, respectively; θ is the angle between the plane of collapse and the horizontal plane, which is equal to $45+0.5\ \varphi$; α is the angle of inclination of the rear face of the counterfort to the horizontal plane.

4. 1. The distribution of lateral earth pressure on a sheet pile wall with a counterfort with downward expansion

In solving this problem, the equilibrium condition is considered for the elementary volume 12341'2'3'4' with the width S, separated within the upper section (Fig. 1).

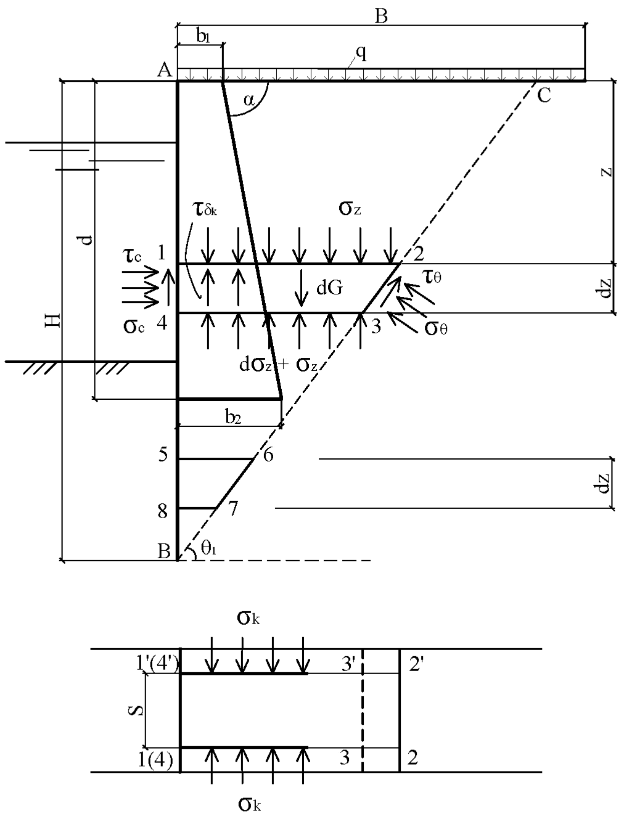


Fig. 1. A diagram for calculating the earth pressure on the wall with a downward expanded counterfort

By designing the normal and tangential forces acting along the faces of the elementary volume, together with the volume force dG on the horizontal plane, the following equation is obtained:

$$\sigma_c \cdot dz \cdot S + \tau_\theta \cdot \cos\theta \cdot S \cdot \frac{dz}{\sin\theta} - \sigma_\theta \cdot S \cdot \sin\theta \cdot \frac{dz}{\sin\theta} = 0. \quad (4)$$

Taking into account the equality $\tau_\theta = \sigma_\theta \mathrm{tg}\phi$, expression (4) is transformed into the following equation:

$$\sigma_\theta = \sigma_c \frac{\mathrm{tg}\theta}{\mathrm{tg}\theta - \mathrm{tg}\phi}, \quad (5)$$

where φ is the angle of internal friction of the soil, deg; S is the step of the counterforts, m.

By designing the vertical and tangential forces acting along the faces of the elementary volume, together with the volume force dG on the vertical plane, the following expression is developed:

$$-\tau_c \cdot dz \cdot S - \tau_\theta \cdot \frac{dz}{\sin\theta} \cdot S \cdot \sin\theta -$$
$$-\sigma_\theta \cdot \frac{dz}{\sin\theta} \cdot S \cdot \cos\theta + dG \cdot S \cdot \frac{H-z}{\mathrm{tg}\theta} -$$
$$-d\sigma_z \cdot \frac{H-z}{\mathrm{tg}\theta} \cdot S + \sigma_z \cdot \frac{dz}{\mathrm{tg}\theta} \cdot S -$$
$$-\tau_{\delta_k} \cdot dz \cdot 2 \cdot (z \cdot \mathrm{ctg}\alpha + b) = 0. \quad (6)$$

Taking into account that

$$\tau_c = \sigma_c \mathrm{tg}\delta_c, \quad dG = \gamma \frac{H-z}{\mathrm{tg}\theta} dz, \quad \tau_{\delta_k} = \sigma_z \xi \mathrm{tg}\delta_k,$$

equation (6) takes the following form:

$$d\sigma_z (H-z) = dz\big(\gamma(H-z) - \psi_1 \sigma_c - \sigma_z(\psi_3 z + \psi_2)\big), \quad (7)$$

where

$$\psi_1 = \mathrm{tg}\delta_c \mathrm{tg}\theta + \frac{\mathrm{tg}\theta(g\phi \mathrm{tg}\theta + 1)}{\mathrm{tg}\theta - \mathrm{tg}\phi}; \quad \psi_2 = \frac{2b\xi \mathrm{tg}\delta_k \mathrm{tg}\theta}{S} - 1;$$

$$\psi_3 = \frac{2\xi \mathrm{tg}\delta_k \mathrm{tg}\theta}{S \mathrm{tg}\alpha};$$

δ_c is the angle of friction of the soil against the wall, deg; δ_k is the angle of friction of the soil against the counterfort, deg; γ is the specific gravity of the soil, kN/m^3; and ξ is the coefficient of lateral rest pressure equal to $\xi = 1 - \sin\phi$.

The equation of the moments of forces relative to face 22' is as follows:

$$-\tau_c \cdot dz \cdot S \cdot \frac{H-z}{\mathrm{tg}\theta} + dG \cdot S \cdot \frac{H-z}{2\mathrm{tg}\theta} -$$
$$-d\sigma_z \cdot \frac{H-z}{\mathrm{tg}\theta} \cdot S \cdot \frac{H-z}{2\mathrm{tg}\theta} -$$
$$-\tau_{\delta_k} \cdot dz \cdot 2 \cdot (z \cdot \mathrm{ctg}\alpha + b) \cdot \left(\frac{H-z}{\mathrm{tg}\theta} - \frac{z \cdot \mathrm{ctg}\alpha + b}{2}\right) = 0. \quad (8)$$

Relevant transformations produce the following equation:

$$d\sigma_z (H-z) =$$
$$= dz\left(\gamma(H-z) - \psi_8 \sigma_c - \sigma_z\left(2\psi_3 z + 2\psi_4 - \frac{\psi_5 z^2 + \psi_6 z + \psi_7}{H-z}\right)\right), \quad (9)$$

where

$$\psi_4 = \frac{2b\xi \mathrm{tg}\delta_k \mathrm{tg}\theta}{S}; \quad \psi_5 = \frac{2\xi \mathrm{tg}\delta_k \mathrm{tg}^2\theta}{S \mathrm{tg}^2\alpha};$$

$$\psi_6 = \frac{4b\xi \mathrm{tg}\delta_k \mathrm{tg}^2\theta}{S \mathrm{tg}\alpha}; \quad \psi_7 = \frac{2b^2 \xi \mathrm{tg}\delta_k \mathrm{tg}^2\theta}{S}$$

and

$$\psi_8 = 2\mathrm{tg}\delta_c \mathrm{tg}\theta.$$

As a result of equating the right-hand sides of equations (7) and (9), a relationship is established between vertical stresses and horizontal pressure, characterized by the coefficient of lateral earth pressure:

$$\sigma_c = \sigma_z \lambda_1, \quad (10)$$

where

$$\lambda_1 = \frac{(\psi_2 - \psi_3 z - 2\psi_4)(H-z) + \psi_5 z^2 + \psi_6 z + \psi_7}{(\psi_8 - \psi_1)(H-z)}. \quad (11)$$

Taking into account expression (10), equation (7) takes the form

$$\sigma_z' + \sigma_z \frac{\eta_1 z^2 + \eta_2 z + \eta_3}{(H-z)^2} - \gamma = 0, \quad (12)$$

where

$$\eta_1=\frac{2\psi_1\psi_3-\psi_3\psi_8+\psi_1\psi_5}{\psi_8-\psi_1};$$

$$\eta_2=\frac{2\psi_1\psi_4-\psi_2\psi_8+\psi_1\psi_6+H\left(\psi_3\psi_8-2\psi_1\psi_3\right)}{\psi_8-\psi_1}$$

and

$$\eta_3=\frac{\psi_1\psi_7+H\left(\psi_2\psi_8-2\psi_1\psi_4\right)}{\psi_8-\psi_1}.$$

When solving the resulting equation, the integration constant is based on the initial conditions z=0, σ_z=q, where q is the load on the surface of the fill soil:

$$C_1=qH^u\exp\left(\frac{w}{H}\right), \tag{13}$$

where $u=2H\eta_1+\eta_2$, $w=\eta_1H^2+\eta_2H+\eta_3$.

Taking into account the value of the integration constant C_1, the solution of equation (12) takes the form

$$\sigma_z=(H-z)^{-u}\exp\left(-\eta_1z-\frac{w}{H-z}\right)\times$$
$$\times\left(C_1+\int_0^z\frac{\gamma}{(H-z)^{-u}\exp\left(-\eta_1z-\dfrac{w}{H-z}\right)}\mathrm{d}z\right). \tag{14}$$

Then the expression for determining the horizontal earth pressure in the first section of the wall, taking into account (11), can be represented in the following form:

$$\sigma_c^I=(H-z)^{-u}\exp\left(-\eta_1z-\frac{w}{H-z}\right)\times$$
$$\times\frac{\left(\psi_2-\psi_3z-2\psi_4\right)(H-z)+\psi_5z^2+\psi_6z+\psi_7}{\left(\psi_8-\psi_1\right)(H-z)}\times$$
$$\times\left(C_1+\int_0^z\frac{\gamma}{(H-z)^{-u}\exp\left(-\eta_1z-\dfrac{w}{H-z}\right)}\mathrm{d}z\right). \tag{15}$$

Next, the equilibrium condition of the elementary volume 56785’6’7’8’, identified within the second section ($d\leq z\leq H$), is considered. By designing the vertical and tangential forces acting along the faces of the elementary volume on the horizontal plane in the second section, expression (5) is also obtained.

Designing the vertical and tangential forces on the vertical plane produces the equation

$$-\tau_c\cdot dz\cdot S-\tau_\theta\cdot\frac{dz}{\sin\theta}\cdot S\cdot\sin\theta-$$
$$-\sigma_\theta\cdot\frac{dz}{\sin\theta}\cdot S\cdot\cos\theta+dG\cdot S\cdot\frac{H-z}{\mathrm{tg}\theta}-$$
$$-d\sigma_z\cdot\frac{H-z}{\mathrm{tg}\theta}\cdot S+\sigma_z\cdot\frac{dz}{\mathrm{tg}\theta}\cdot S=0. \tag{16}$$

Performing similar transformations transforms equation 16 into the following:

$$d\sigma_z(H-z)=dz\left(\gamma(H-z)-\psi_9\sigma_c+\sigma_z\right), \tag{17}$$

where

$$\psi_9=\frac{\mathrm{tg}\theta\left(\mathrm{tg}\delta_c\mathrm{tg}\theta-\mathrm{tg}\delta_c\mathrm{tg}\phi\right)+\mathrm{tg}\theta\mathrm{tg}\phi+1}{\mathrm{tg}\theta-\mathrm{tg}\phi}.$$

Forming the equation of the moments of forces relative to face 66’ produces the following equation:

$$-\tau_c\cdot dz\cdot S\cdot\frac{H-z}{tg\theta}+dG\cdot S\cdot\frac{H-z}{tg\theta}\cdot\frac{H-z}{2tg\theta}-$$
$$-d\sigma_z\cdot\frac{H-z}{tg\theta}\cdot S\cdot\frac{H-z}{2tg\theta}=0. \tag{18}$$

The transformation of this equation produces the following:

$$d\sigma_z(H-z)=dz\left(\gamma(H-z)-\psi_8\sigma_c\right). \tag{19}$$

As a result of equating the right-hand sides of equations (17) and (19), a relationship is established between the vertical stresses and the horizontal pressure (10), where

$$\lambda_1=\frac{1}{\psi_9-\psi_8}. \tag{20}$$

Taking into account expression (10), equation (17) takes the form

$$\sigma_z^{'}+\sigma_z\frac{\eta_4}{H-z}-\gamma=0, \tag{21}$$

where

$$\eta_4=\frac{\psi_9}{\psi_9-\psi_8}-1.$$

The integration constant is determined from the condition of equality of vertical stresses on the boundary of the sections σ_z=q, z_1=d, where q is the vertical pressure at depth d:

$$C_2=\frac{\sigma_z(d)}{(H-d)^{\eta_4}}-\int_0^d\frac{\gamma}{(H-d)^{\eta_4}}\mathrm{d}z. \tag{22}$$

Then the general solution of equation (21) takes the form

$$\sigma_z=(H-z)^{\eta_4}\left(C_2+\int_d^H\frac{\gamma}{(H-z)^{\eta_4}}\mathrm{d}z\right). \tag{23}$$

Taking into account that the expression for the lateral earth pressure coefficient within the second section has the form of (20), the intensity of the horizontal earth pressure in the second section of the wall can be written as follows:

$$\sigma_c^{II}=(H-z)^{\eta_4}\frac{1}{\psi_9-\psi_8}\left(C_2+\int_d^H\frac{\gamma}{(H-z)^{\eta_4}}\mathrm{d}z\right). \tag{24}$$

The obtained solutions have been numerically implemented on the basis of the developed algorithms with the use of the computer-aided calculation program MathCad.

4. 2. The distribution of lateral earth pressure on a sheet pile wall with a counterfort with upward expansion

The equilibrium condition for the elementary volume 12341'2'3'4' is considered for the case when the wall width decreases downwards (Fig. 2). The boundaries of the upper section are $0 \le z \le d$, and the boundaries of the lower section are $d \le z \le H$.

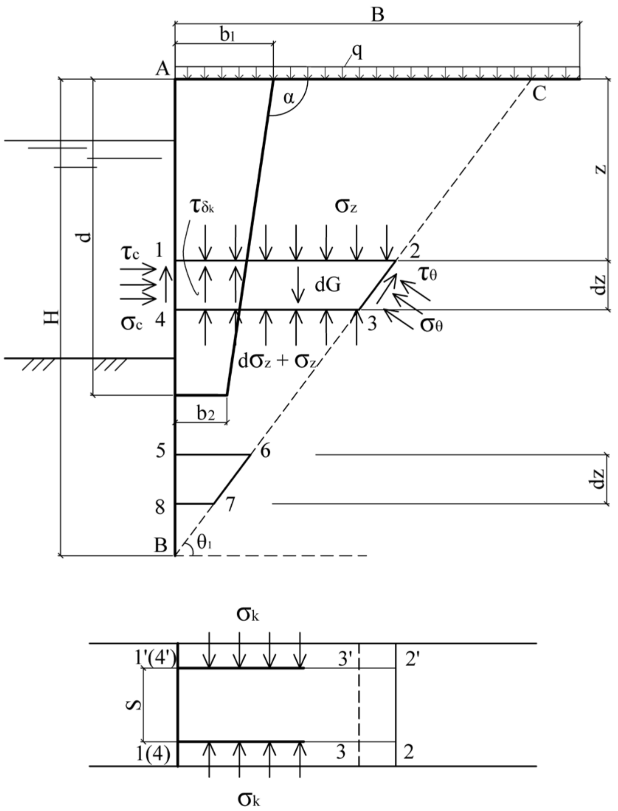


Fig. 2. A diagram for calculating the earth pressure on the wall with an upward expanded counterfort

By designing the normal and tangential forces acting along the faces of the elementary volume, together with the volume force dG, on the horizontal plane, equations (4) and (5) are obtained.

The designing of the normal and tangential forces acting along the faces of the elementary volume, together with the volume force dG, on the vertical plane produces the equation

$$\begin{aligned}&-\tau_c \cdot dz \cdot S - \tau_\theta \cdot \frac{dz}{\sin\theta} \cdot S \cdot \sin\theta - \\ &-\sigma_\theta \cdot \frac{dz}{\sin\theta} \cdot S \cdot \cos\theta + dG \cdot S - d\sigma_z \cdot \frac{H-z}{tg\theta} \cdot S + \\ &+\sigma_z \cdot \frac{dz}{tg\theta} \cdot S - \tau_{\delta_k} \cdot dz \cdot 2 \cdot (b - z \cdot ctg\alpha) = 0. \qquad (25)\end{aligned}$$

By performing similar transformations, the following equation is obtained:

$$\begin{aligned}&d\sigma_z (H-z) = \\ &= dz\big(\gamma(H-z) - \psi_1 \sigma_c - \sigma_z(\psi_2 - \psi_3 z)\big). \qquad (26)\end{aligned}$$

Relative to face 22', the equation of moments acquires the form

$$\begin{aligned}&-\tau_c \cdot dz \cdot S \cdot \frac{H-z}{tg\theta} + dG \cdot S \cdot \frac{H-z}{2tg\theta} - \\ &-d\sigma_z \cdot \frac{H-z}{tg\theta} \cdot S \cdot \frac{H-z}{2tg\theta} - \\ &-\tau_{\delta_k} \cdot dz \cdot 2 \cdot (b - z \cdot ctg\alpha) \cdot \left(\frac{H-z}{tg\theta} - \frac{b - z \cdot ctg\alpha}{2}\right) = 0. \qquad (27)\end{aligned}$$

Taking into account the transformations, equation (27) becomes the following:

$$\begin{aligned}&d\sigma_z (H-z) = \\ &= dz\left(\gamma(H-z) - \psi_8 \sigma_c - \sigma_z\left(2\psi_4 - 2\psi_3 - \frac{\psi_5 z^2 - \psi_6 z + \psi_7}{H-z}\right)\right). \quad (28)\end{aligned}$$

As a result of equating the right-hand sides of expressions (26) and (28), equation (10) is obtained, where

$$\lambda_1 = \frac{(\psi_2 + \psi_3 z - 2\psi_4)(H-z) + \psi_5 z^2 - \psi_6 z + \psi_7}{(\psi_8 - \psi_1)(H-z)}, \qquad (29)$$

the coefficients ψ_1, $\psi_2 \ldots \psi_8$ are given above.

Taking into account expression (10), equation (26) takes the form

$$\sigma_z^{'} + \sigma_z \frac{\eta_1 z^2 - \eta_2 z + \eta_3}{(H-z)^2} - \gamma = 0, \qquad (30)$$

where

$$\eta_1 = \frac{2\psi_1\psi_3 - \psi_3\psi_8 + \psi_1\psi_5}{\psi_8 - \psi_1};$$

$$\eta_2 = \frac{2\psi_1\psi_4 - \psi_2\psi_8 + \psi_1\psi_6 + H(\psi_3\psi_8 - 2\psi_1\psi_3)}{\psi_8 - \psi_1}$$

and

$$\eta_3 = \frac{\psi_1\psi_7 + H(\psi_2\psi_8 - 2\psi_1\psi_4)}{\psi_8 - \psi_1}.$$

When solving the resulting equation, the integration constant is determined starting from the initial conditions that $z=0$ and $\sigma_z = q$:

$$C_1 = qH^u \exp\left(\frac{w}{H}\right), \qquad (31)$$

where

$$u = 2H\eta_1 - \eta_2, \quad w = \eta_1 H^2 - \eta_2 H + \eta_3.$$

The form of the general solution (14) does not change. Expression (16) changes with regard to (30):

$$\begin{aligned}&\sigma_c^I = (H-z)^{-u} \exp\left(-\eta_1 z - \frac{w}{H-z}\right) \times \\ &\times \frac{(\psi_2 + \psi_3 z - 2\psi_4)(H-z) + \psi_5 z^2 - \psi_6 z + \psi_7}{(\psi_8 - \psi_1)(H-z)} \times \\ &\times \left(C_1 + \int_0^z \frac{\gamma}{(H-z)^{-u} \exp\left(-\eta_1 z - \frac{w}{H-z}\right)} \mathrm{d}z\right). \qquad (32)\end{aligned}$$

The structure and nature of the calculated dependences for the lower wall section remain unchanged for the configuration of counterforts under consideration.

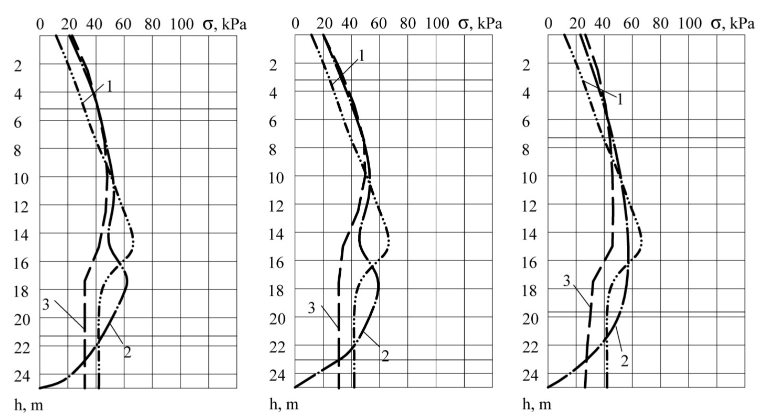
The obtained solutions have been numerically implemented on the basis of the developed algorithms with the use of the computer-aided calculation program MathCad. Besides, an algorithm has been developed for solving special cases of a wall without a counterfort and with a rectangular shape counterfort.

5. Results of studying the stress-strain state of the system "a sheet pile wall with counterforts plus the soil environment"

To assess the stress-strain state of the system "a sheet pile wall with counterforts plus the soil environment", mathematical modeling is performed on the basis of the developed calculation method.

The constant initial data are the following: the wall height H=25 m; the step of the ribs S=3.0 m; the height of the stiffeners d=17.0 m; specific earth gravity γ=11 kN/m^3; the angle of internal friction of the soil φ=30°; and the uniformly distributed load on the surface q=40 kPa. The dependence of the lateral pressure on the shape of the counterfort has been studied, and also the friction of the soil against the wall has been taken into account. Three forms of counterforts have been considered: rectangular (width b_1=b_2=3.0 m), downward expansion (width at the top (bottom) $b_1(b_2)$=1.8(4.2) m), and upward expansion (width at the top (bottom) $b_1(b_2)$=4.2(1.8) m).

On the diagrams (Fig. 3), the graphical dependences reflect the distribution of earth pressure along the height of the sheet pile wall with counterforts.



a *b* *c*

Fig. 3. Diagrams of earth pressure on the sheet pile wall with counterforts: *a* – rectangular; *b* – with downward expansion; and *c* – with upward expansion. 1 – at δ_c=δ_k=0; 2 – at δ_c=δ_k=0.5; and 3 – at δ_c=0, δ_k=φ

Table 1 is used to quantify the relief effect of the counterforts. It gives the values of the resultant forces of active earth pressure, depending on the angle of friction of the soil against the wall, obtained in calculating for the sheet pile wall with counterforts and without them.

The calculations show that the relief effect is 25.8 % (for δ_c=0 and δ_k=0). The dependence of the lateral earth pressure on the angle value of the soil friction against the wall has also been investigated (taking into account the wall roughness). The results of the tests show that counterforts reduce the lateral earth pressure on the front wall due to the forces of the soil friction along the lateral surface of the counterforts. The earth pressure on the front wall without counterforts and with them without taking into account the roughness (δ_c=0 and δ_k=0) is the same. Taking into account the roughness of the front wall and the counterforts reduces the side pressure of the soil to 2.5 % (δ_c=0.5φ and δ_k=0.5φ). Allowance for friction forces only along the lateral faces of the counterforts (δ_c=0 and δ_k=φ) reduces the pressure to 15.6 % (with a downward expanded counterfort).

Table 1

Values of the resultant forces of lateral earth pressure

No.	Counterfort shape	The resultant force of earth pressure, depending on the angle of the soil friction against the wall, kN			
		δ_c=0 δ_k=0	δ_c=0.5φ δ_k=0.5φ	δ_c=0.7φ δ_k=0.7φ	δ_c=0 δ_k=φ
1	without counterforts	1,479	1,281	1,211	1,109
2	with rectangular shape counterforts	1,097	1,092	1,196.5	961
3	counterforts with downward expansion	1,097	1,071	1,173	926
4	counterforts with upward expansion	1,097	1,096	1,200	969

6. Discussion of the results of studying the stress-strain state of the system "a sheet pile wall with counterforts plus the soil environment"

The developed method for calculating the lateral earth pressure on the front wall allows using three types of counterforts and regulating their parameters (height, width, and step). Moreover, the developed method makes it possible to study the dependence of earth pressure on friction on the face wall and on the lateral surfaces of the counterforts. The results of the calculations show that taking into account the roughness of the wall makes it possible, with sufficient accuracy for engineering practice, to estimate the values of the resultant lateral earth pressure.

To verify the reliability of the calculated dependences, a comparison has been made with the experimental tests described earlier in [23]. A quantitative comparison of the results is given in Table 2.

The analysis of the comparison between the calculations and the tests shows a discrepancy in the quantitative ratio up to 13.3 %, which is acceptable in engineering practice.

In the future, it is necessary to research the effect of changing the counterfort parameters (step, width, and height) on the lateral earth pressure to obtain the greatest relief effect at optimal dimensions, and also to study their effect on the stiffness of the structure as a whole.

Table 2

Comparison of the experimental and calculated data

Indicators	The resultant force of earth pressure on the wall (Ea, kN), depending on the counterfort shape		
	Rectangular	Trapezoidal with downward expansion	Trapezoidal with upward expansion
According to the calculated dependences	1,928	1,872	1,779
According to the experiment	1,974	1,960	2,052

The introduction of the developed calculation method into engineering practice will allow designing hydraulic engineering structures such as a sheet pile wall with various shapes of counterforts. This will enable the construction of new deepwater hydraulic structures with an increased bearing capacity.

These studies are an additional development and a continuation of the research on the patented construction of a retaining sheet pile wall with counterforts.

7. Conclusion

1. The proposed method of calculation for determining the earth pressure on the front wall, taking into account the relief effect of the counterforts, can be used in the design of new types of deepwater piers – sheet pile walls with counterforts. Moreover, the counterfort shape can be rectangular, trapezoidal with downward expansion, and trapezoidal with upward expansion.

2. The conducted tests show that the use of counterforts in a sheet pile wall with the considered parameters reduces the pressure of the filling soil to 26 % due to the friction forces along the lateral surfaces of the counterforts.

3. By changing the step and the width and by choosing the shape of the counterforts, it is possible to change the amount of the relief effect on the front wall. Moreover, if the step is reduced or the width of the counterfort is increased, it is possible to achieve a greater relief effect or an effect required by the project. In this case, the step and the width of the counterfort are assigned to be divisible by the width of the sheet pile.

References

1. European choice – deep water terminals from combined dowel / Doubrovsky M., Poizner M., Petrosyan V., Kalugnya V. // Ports of Ukraine. 2007. Issue 06 (68). P. 42–44.
2. The deep water berth is being built // Ports of Ukraine. 2010. Issue 4 (96). P. 3.
3. Fumiaki T., Noriyuki S. Self-erecting type landslide protection wall construction: Pat. No. 05009930 A Japan. Int. Cl E02D 5/02, E02D 17/04. declareted: 04.07.1991; published: 19.01.1993.
4. Doubrovsky M. P., Meshcheryakov G. N. Sheet pile testing & design improvement // Proceedings of IS-Kanazawa 2012 "Testing and Design for Deep Foundation". Kanazawa University, Japan, 2012. P. 589–596.
5. Slobodyanik A. V., Bagrationy R. R., Slobodyanik A. V. Research of the Work of Thin Retaining Wall with Stiffeners // Eastern European Scientific Journal (Gesellschaftswissenschaften). 2015. Issue 3. P. 146–151.
6. Dubrovsky M. P., Slobodyanik G. V. Retaining wall: Pat. No. 84888 UA. MPK (2006) E02D 29/2, E02B 3/06 / inventor and owner: Odessa National Marine University. No. a200605883; declareted: 29.05.2006; published: 10.12.2008, Bul. No. 23. 4 p.
7. Slobodyanik G. V. A method of erection of hydraulic structures such as a hinged wall: Pat. No. 115379 UA. MPK (2017.01) E02B 3/06, E02D 5/00, E02D 29/02 / inventor and owner G. V. Slobodyanik. No. 201611675; declareted: 18.11.2016; published: 10.04.2017, Bul. No. 7. 4 p.
8. Shiau J., Smith C. Developing Numerical Models for the Design of Cantilever Sheet Pile Wall // Research, Development and Practice in Structural Engineering and Construction. 2012. doi: 10.3850/978-981-08-7920-4_gfe-12-0308
9. Bekdaş G., Temür R. Metaheuristic approaches for optimum design of cantilever reinforced concrete retaining walls // Challeng Journal of Structural Mechanics. 2017. Vol. 3, Issue 1. P. 23–30. doi: 10.20528/cjsmec.2016.11.031
10. Shehata H. F. Retaining walls with relief shelves // Innovative Infrastructure Solutions. 2016. Vol. 1, Issue 1. doi: 10.1007/s41062-016-0007-x
11. Farouk H. Finite element analysis for the retaining wall with relief shelves // XV Danube – European Conference on geotechnical engineering (DECGE 2014). Vienna, 2014. No. 52.
12. Hu Y., Liu G., Zhao Y. Calculation Method of Deformation and Inner Force of a Sheet Pile Wall with Relieving Platform // ICTE 2013. 2013. doi: 10.1061/9780784413159.025
13. Evstigneev V. N. Experimental study of soil pressure on the wall with counterfoils // Port Hydrotechnical Construction. 1967. Issue 15 (21). P. 23–28.
14. Shihiev F. M., Fel'dman Ya. N. Some cases of ground pressure on fences with a nonplanar problem // Port Hydraulic Engineering. 1967. Issue 15 (21). P. 88–91.
15. Zelensky V. S. Determination of pressure of friable environment on retaining walls with the ribbed verge // Ground, foundations and mechanics of soil. 1969. Issue 6. P. 6–8.
16. Zelensky V. S. Calculation of the spreading pressure of the soil on the walls of the quay embankments, taking into account the unloading effect of buttresses // Marine hydraulic engineering. 1978. Issue 6. P. 13–20.
17. Kovacshazy F. Model studies on retaining walls // Acta Technica Academial Scientiarum Hungarical. 1968. Vol. 62, Issue 1-2. P. 28–35.
18. Sokolov A. D. Ground pressure on retaining structures with buttresses from the ground backfilling depending on the movements // Questions of construction mechanics, safety of structures and hydraulics. 2005. Issue 3. P. 124–146.

19. Sokolov A. D. Lightweight retaining walls with ribbed pressure face // Transport construction. 2006. Issue 8. P. 20–24.
20. Measurement of the spreading of the backfill and pressure on the bottom of the embankment in the port of Hakata (Japan) // Pros. Harbour Engng. 1964. Issue 40. P. 4756–4775.
21. RD 31.31.27-81. Guidelines for the design of marine berthing facilities. Moscow: Mortekhinformreklama, 1984. 399 p.
22. Slobodyanik A. V. The calculated substantiation of methods of technical operation of water transport facilities of sheet walls of increased rigidit // News of Odessa National Marine University. 2009. Issue 26. P. 118–131.
23. Slobodyanik A. V., Honeliya N. N. Results of studies of the lateral pressure of the backfill soil on a thin retaining wall // Budivelni konstruktsiyi. 2016. Issue 83. P. 397–402.

UDC 539.3
DOI: 10.15587/1729-4061.2018.120547

THIN-WALLED STRUCTURES: ANALYSIS OF THE STRESSED-STRAINED STATE AND PARAMETER VALIDATION

M. Tkachuk
Doctor of Technical Science, Professor, Head of Department*
E-mail: tma@tmm-sapr.org
M. Bondarenko
Junior Research assistant*
A. Grabovskiy
PhD, Senior Researcher*
A. Vasiliev
PhD*
R. Sheychenko
Chief designer of the tank-car project**
R. Graborov
Head of group of technical settlement**
V. Posohov
Senior Lecturer
Department of Repair and operation of cars and military vehicles
National academy of the National guards of Ukraine
Zakhysnykiv Ukrainy sq., 3, Kharkiv, Ukraine, 61001
E. Lunyov
Manager
JSK "ARGUS-Personnel"
Sichovyh striltsiv str., 3, Kyiv, Ukraine, 04053
A. Nabokov
First deputy director
Dnipropetrovsk Pedagogical College
Polia ave., 83, Dnipro, Ukraine, 49000
*Department of Theory and Computer-Aided Design of Mechanisms and Machines
National Technical University "Kharkiv Polytechnic Institute"
Kyrpychova str., 2, Kharkiv, Ukraine, 61002
**JSC "Science Engineering Center UK "RailTransHolding"
Volhohradska str., 24, Mariupol, Ukraine, 87502

Розроблено підхід до обґрунтування технічних рішень для тонкостінних машинобудівних конструкцій. Задача розглядається у просторі узагальнених параметрів, які об'єднують проектні й технологічні чинники та умови експлуатації. У сформованому параметричному просторі будується апроксимована поверхня відгуку. На додаток вводяться критеріальні та обмежувальні залежності. Після цього проводиться пошук оптимуму функції якості досліджуваної конструкції

Ключові слова: тонкостінна машинобудівна конструкція, напружено-деформований стан, поверхня відгуку, інноваційний виріб

Разработан подход к обоснованию технических решений для тонкостенных машиностроительных конструкций. Задача рассматривается в пространстве обобщенных параметров, которые объединяют проектные и технологические факторы и условия эксплуатации. В сформированном параметрическом пространстве строится аппроксимированная поверхность отклика. В дополнение вводятся критериальные и ограничительные зависимости. После этого проводится поиск оптимума функции качества исследуемой конструкции

Ключевые слова: тонкостенная машиностроительная конструкция, напряженно-деформированное состояние, поверхность отклика, инновационный продукт

1. Introduction

Modern industry, transport and service sector are facing these days ever growing demand for innovative engineering solutions. Therefore the machine-building companies are inevitably shifting their focus towards development, pre-production and manufacturing of such products. A substantial part of such activities is related to thin-walled machine