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Shielded retaining wall in the form of substandard rack-mount frame

Экранированный больверк в виде нестандартной стоечной рамы

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Key words: berth; shielded retaining wall; variable foundation modulus; silage pressure; incomplete sliding wedge

Ключевые слова: причальная набережная; экранированный больверк; переменный коэффициент постели; силосное давление грунта; неполная призма обрушения

Abstract. The article gives a practical implementation of the engineering method for calculation of different retaining walls types. The main provision of the method is to provide the construction calculation model as a combination of construction elements with the respective conditions of their fixing. The calculation model of shielded retaining wall (covered type of sheet pile wharf) is proposed in the form of substandard rack-mount frame where resilient ground attachment is accepted instead of rack lower anchorage. This model uses stiffness characteristics of the ground in the form of variable foundation modulus. The article considers the definition of lateral earth pressure on the wall while staying in sliding wedge of pile bent. The article gives an engineering solution for definition of diagrams of lateral pressure on the front wall with allowance for nonlinear influence of pile bent location, piles pitch and cross section.

Аннотация. Приведена практическая реализация, предложенного ранее инженерного универсального метода расчета причальных набережных. Основное положение метода заключается в представлении расчетной схемы сооружения в виде комбинации элементов конструкции с соответствующими условиями их закрепления. Расчетная схема экранированного больверка предложена в виде нестандартной стоечной рамы, где вместо нижней жесткой заделки стоек, принимается упругая грунтовая заделка. При этом используются жесткостные характеристики грунта, в виде переменных коэффициентов постели. Рассмотрен вопрос определения бокового давления грунта на стенку при нахождении в призме обрушения свайного ряда. Получено инженерное решение определения эпюры бокового давления на лицевую стенку с учетом нелинейного влияния расположения свайного ряда, шага и сечения свай.

Introduction

The calculation of water transport hydrotechnical constructions interacting with the ground belongs to one of the mixed elastic-plastic tasks of continuous body. There are a significant number of software packages (Geomechanica, Plaxis, Msheet, LIRA, etc.). They allow by a numerical method on a PC to calculate the stress-strain state of the construction in two and three dimensional statement taking into account elastic and extreme areas that previously was impossible. The calculation of berthing constructions in software packages using the model of a continuous body allowed to obtain a more complete picture of construction work in the ground. However, the calculation of these constructions in programs which use the model of a continuous medium designed for structural materials is not strict because it gives an approximate picture of the pile foundation behaviour in discrete soil ground

Due to the complexity of consideration of many factors affecting the structures in the port hydraulic engineering various engineering solutions [1–7] are widely used including methods considering foundation modulus (hypothesis of Fuss-Winkler) and not taking into account the distribution capacity of the ground. Some of the works used abroad [8–17] should be noted. These papers consider the

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tabulated solutions to the classical Blum-Lohmeyer method of calculation [8]. Abroad in the last 10–15 years the calculation of berthing facilities is performed in the program Plaxis using the finite element method for the continuum model. This model designed for construction materials is not strict for soils. It gives an approximate picture of pile foundation behaviour in discrete soil ground.

Using software packages based on the model of continuous body reference to the tasks of port hydraulic engineering requires consideration of several factors: difference in compression and tension ground resistance, variable degree of distribution capacity of the ground adjacent to the deformed wall up to its complete disappearance, for example, because of buttress piers end displacement leading to significant transformation of initial outline of pressure diagrams on height and so on [18]. The using of vertical thin layer of ground with reduced strength characteristics (interface) and so on at the contact point between the ground and wall describes their connection in the model of continuous body corresponding to the full adhesion of two areas incorrectly. It is not accident when publishing tasks associated with anchored enclose in programs that implement a continuous body the diagram of contact lateral earth pressure on the wall is not provided, and the obtained contour curves of the horizontal stress of ground behind the wall is more or less similar to the Coulomb diagram [19–23].

The model of discrete (grainy) environment revealing the physical mechanism of intergrain interaction more corresponds to the behaviour of backfill ground behind the wall than the model of continuous body based on a phenomenological approach [24].

Shielding elements which take a part of lateral pressure are widely used in deep-water sheet pile wharf, so that allows us to use a sheet piles with lower bearing capacity [1]. In addition, both task of improving the method of covered type of sheet pile wharf calculation and task of construction units' lateral pressure determination are relevant.

The purpose of work is to refine the shielding (insulation) effect of front wall due to the pile row in shielded retaining wall. In technical literature the effect of piles' step and their distance to the front wall on the shielding effect is not considered [1, 6, etc.]. Moreover, this effect is taken into account indirectly through the stiffness characteristics of pile foundation elements while determining their bending moments [6]. The authors set the task of proposing a direct method for determining the pressure on the wall from the soil wedge cut out by a pile row. To solve this problem a scheme for the distribution of the resulting lateral pressure in the form of a strip load between the piles acting on the front wall is considered. In this case, the piles' step and the distance to the wall are taken into account.

Materials and Methods

It is commonly believed that ground vaults are formed between the piles of longitudinal row on account of ground friction on piles (Fig. 1a) [6]. For simplicity they are replaced by a scalloped surface (Fig. 1b). These vaults take up lateral pressure of the ground located behind them and transmit it to the piles.

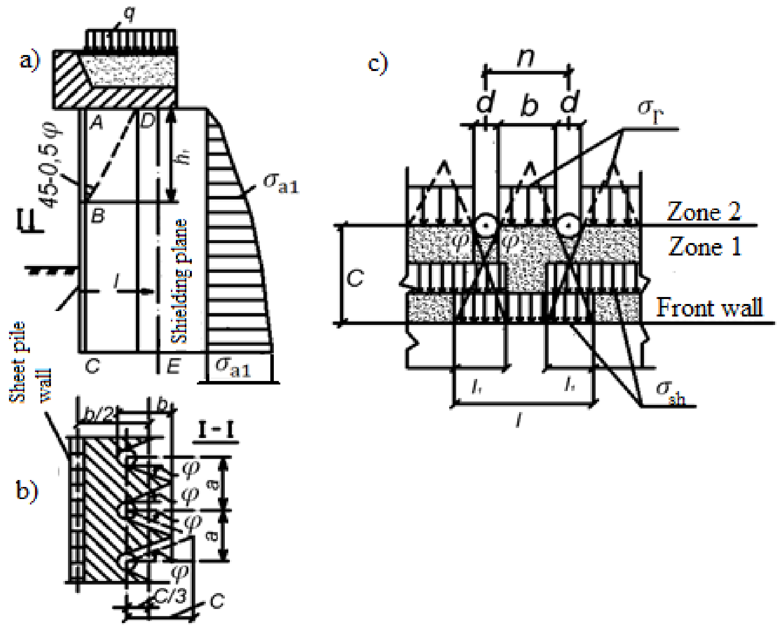

Figure 1. Composition schemes of diagrams of silage ground pressure on the front all (a) and (b) additional shielding load pressure behind pile row (c)

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Only the pressure of the ground located between the wall and the ground vaults of piles' longitudinal row fully acts on the sheet pile wall. For calculations vaulted surface is replaced by a conventional vertical shielding plane shielding DE (Fig. 1a). The position of the shielding plane is usually determined by graphical constructions (Figs. 1a and b) [6].

Shielding plane is accepted at a distance c/3 from the axis of longitudinal pile row, where c is the scallop height in this case. The diagram of backfill ground pressure on sheet pile wall is constructed considering the position of shielding plane.

Ground filling between sheet pile wall and shielding plane is in conditions similar to the state of granular material in a flat silage. In the graphical calculation silage pressure is calculated by drawing (at an angle of 45 ° – 05 φ to the vertical) from crossing point D of shielding plane with foundation grill a line DB of ground break to its intersection with the calculated wall plane (Fig. 1a.).The ground pressure at the wall segment AB rises linearly. The ordinate of pressure diagram at point B.

$$\sigma_{a1} = \gamma \cdot h_1 \cdot \lambda_a \tag{1}$$

Below point B the intensity of ground pressure remains constant σ_{a1}.

$$\sigma_{sil} = \gamma h\,\lambda(\delta),\ 0 < h < h_1; \quad \sigma_{sil} = const \ \text{ if } \ h \geq h_1 = l\,\mathrm{tg}(45° - 0.5\varphi°)\ . \tag{2}$$

The authors believe that the replacement of pile row by solid shielded wall is a particular case of pressure at sufficiently flexible front wall and a small pitch distance. In such wall deflection implements an active pressure with the manifestation of ground vaults when the step between piles is $n \leq 3d$. In technical literature the value of bending moment in the wall and pile row distributes according to their rigidity [6]. Such decision is incorrect because it does not consider the actual ground pressure on the construction elements.

Shielding effect occurs not in all types of grounds. Practice shows that shielding effect of basement in weak grounds is poorly effective. As weak grounds because of their properties closed to the viscous liquid it causes a conventional flawover (dulling) of piles by soil ground. In weak clayey grounds it is recommended not to take into account Shielding effect.

It is assumed that the intensity of lateral pressure on sheet piles consists of silage pressure σ_{sil} in zone 1 and additional pressure from the shielding pile row σ_{sh} (Fig. 1c) [25]. It is equal to the difference between external pressure in zone 2 and reverse silage pressure on shielding ground plane in zone 1. The value of this difference in turn depends on the pile step and distance C to the front wall (Fig. 1c).

$$\sigma_{sp} = \sigma_{sil} + \sigma_{sh}\ . \tag{3}$$

The authors propose to determine the pressure on sheet piles from zone 2 considering the influence of pile step and the distance C to the front wall by means of distribution coefficient K_d = 0÷1.

$$\sigma_{sh} = K_d\,\sigma_r = (n - d)/n)(2l_1/l)\sigma_r, \tag{4}$$

where $\sigma_r = (\sigma - \sigma_{sil})$ – resulting pressure in a plane passing through the shielding piles; $\sigma = (q + \gamma h)\lambda_a$ – lateral pressure on pile row (Fig. 1*c*); *n* – piles' step; $l = b + 2c \cdot tg\varphi$ – the area of distribution of resulting load between piles in section b on the sheet pile wall; $l_1 = c \cdot \mathrm{tg}\varphi$ – the area of distribution of resulting load behind piles on the sheet pile wall, (Fig. 1*c*); *d* – diameter of shielding pile.

The influence of pile row step. If $d \to n$ we have $b \to 0$ multiplier $K_p \to 0$ the pressure is completely taken up by pile row, so $\sigma_{sh} \to 0$ (Fig.1c). If $d \to 0$ silage pressure tends to zero $\sigma_{sil} \to 0$, so $\sigma_{sp} \to \sigma$;

The influence of pile row distance from wall. In case $c \to 0$ silage pressure tends to zero, $\sigma_{sh} \to 0$. If $c \to \infty$, $l \to \infty$ respectively, so $\sigma_{sh} \to 0$;

The nonlinear influence of zone 1 width and piles' step at the value of silage pressure intensity. Vertical and inclined piles slope sliding wedge of the ground behind the wall and take upon itself a part of lateral pressure of the ground (shield) decreasing the ground pressure on the front wall. However, the intensity of shielding of triangular sliding wedge`s section is uneven from its width. That is why it is nonlinear.

Consider the character of the phenomena occurring in the ground while slotting sliding wedge by pile row which accepted for simplicity continuous. If pile row is absent, sliding wedge CGE which creates lateral earth pressure CGD impacting on the wall (Fig. 2a). The appearance of pile row in sliding wedge decreases sliding wedge by the amount ABE and accepts pressure diagram ABD. This led to a decrease of diagrams of pressure on the wall by the amount ABD respectively.

Expressing the resultant of shielded pressure on the wall E_a in the form of a nonlinear function of the distance to pile row:

$$E_{\text{eq}} = E_a (x/L)^a , \tag{3}$$

where L – length of sliding wedge; a ≤1 – exponent.

If x = 0 shielded pressure on the wall E_{sh} = 0, that means that the pressure is completely taken up by pile row, but if $x \geq L$ the influence of pile row at the front wall is absent. The dependence of shielded pressure resultant from distance between pile row and the wall x in case n = 0.5 in relative magnitudes is given in Figure 2b.

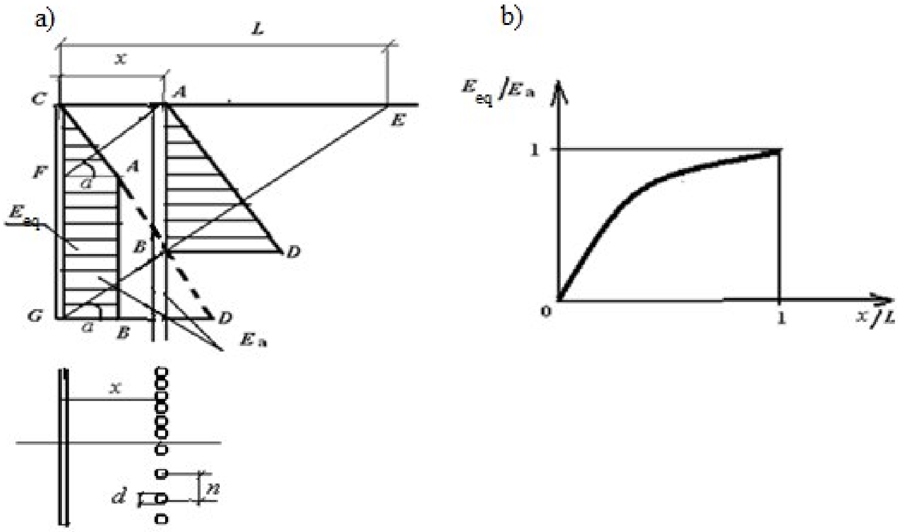

Figure 2. (a) Scheme for accounting of shielding; (b) The dependence of shielded pressure resultant from distance between pile row and the wall

Example 1. Determine the resultant of shielded pressure on the wall if x/L is equal 0.1; 0.2; 0.3.

Using the expression for E_{eq} if n =0.5 we obtain E_{eq1} = 0.31E_a; E_{eq2} = 0.45E_a; E_{eq3} = 0.54E_a, respectively. So the optimal arrangement of shielding pile row should not exceed 0.1 x/L.

Consider pile row with diameter d driven into the ground with step n. In this case the resultant of shielded pressure on the wall E_{sh} is represented as a function of distance to pile row which also depends on the pile diameter and distance between them (Fig. 2a).

$$E_{sh} = E_a (x/L)^a + [E_a - E_a (x/L)^n] \, [\, (n - d)/n]^m , \tag{6}$$

where E_a – resultant of the active pressure excluding shielding; m ≤ 1 – exponent.

Exponents a and m require experimental researches.

If d = 0 the shielding effect is absent, so $E_{sh} = E_a$, but if $d = n$ there is continuous pile row, respectively.

The first term of equation for E_{sh} is a value of the resultant of silage pressure on the sheet pile wall depending on silage width x. The second term of the equation gives the component of redistributed pressure on the wall from the pressure on pile row considering reverse silage pressure.

Preselection of sheet pile type and driving depth of covered type of sheet pile wharf is accepted as for conventional sheet pile wharf. According to Russian Construction Norms and Regulations SNiP 2.02.03-85 the driving depth of the piles is specified and the lateral pressure on sheet pile wharf elements is determined according to the Table 1.

The calculation model of covered type of sheet pile wharf is a rack-mount cantilever frame. In the frame the rear pillar with relieving platform is fastened to rear part of the front wall by hinged support (Fig. 3a) or fastened to rigid supports respectively (Fig. 3b). Additionally the frame front part has hinged support displaying the device of anchor nodes (Fig. 3).

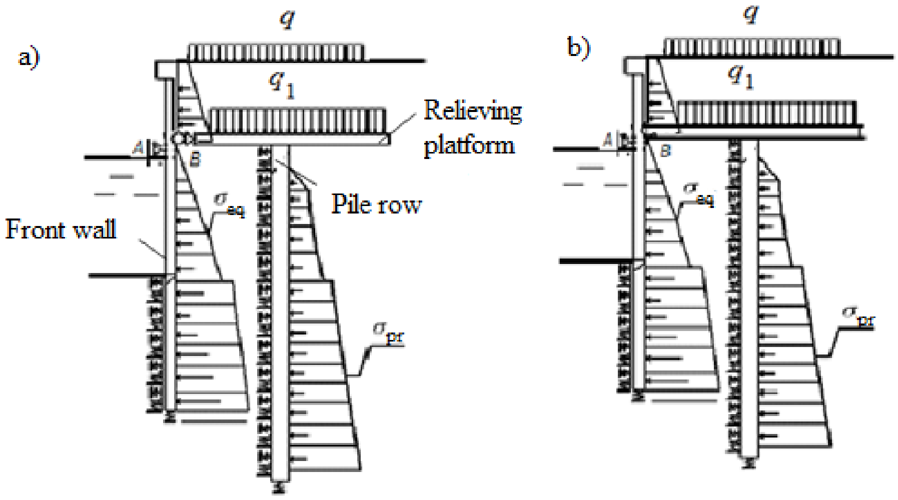

Figure 3. The calculation model of covered type of sheet pile wharf is in the form of: (a) Rack-mount frame with hinged anchor support A in its outer part and hinged support B in its inner part of front pillar to the top of frame; (b) Rack-mount frame with hinged anchor support A in its outer part and rigid supports B in its inner part of front pillar to the top of frame

Foundation modulus for sheet pile wall and piles. The front wall is partially dipped into foundation soil, pile row is totally in it. Lower ends of rack-mount frame have a hinged sliding supports. Ground behavior is described by variable foundation modulus:

$$K_f = y\,K\,K_a{\cdot}K_{pl} \text{ – for sheet pile wall;} \quad (7)$$

$$K_f = a{\cdot}K_{fr}\,K{\cdot}y/\gamma_c \text{ – for piles,} \quad (8)$$

where y – the depth of the considered point; K (kN/m^4) – coefficient of proportionality [7], K_{fr} – friability coefficient (using for piles which submerge into backfill ground); K_a – anisotropy coefficient: for granular soil is 1 and for clayey soil – 0.7 ÷ 0.8, K_{pl} = 0.6 ÷ 0.8 – coefficient considering the plastic properties of foundation in front of the wall for upper ground layer 1÷2 m; a = 0.8 – coefficient taking into account the influence of neighboring piles; γ_c = 3 – coefficient of work conditions. The iteration method implemented in the SCAD-Cross can be used to clarify K_f [26].

The main provisions of suggested calculation use the idea of engineering multi-purpose method in the form of different combinations of construction members [17]. This combination taking into account different conditions of element fixing uses local and structural strains. The calculation model of covered type of sheet pile wharf is accepted in the form of substandard rack-mount cantilever frame. The rear pillar of frame with relieving platform hingedly fastened to rear part of the front wall. In its turn the front part of frame also has hinged support. This support corresponds to anchoring of wall.

Initial data in form loads on the covered type of sheet pile wharf (Fig. 3):

A. Payload on the territory q;

B. The load from the backfill ground q_1.

C. Dead load of relieving platform;

D. The resulting horizontal load of ground on pile row and on the front wall.

To solve this task for the covered type of sheet pile wharf in the SCAD program it is necessary:

1. Introduce the elements of construction, select nodes (according to coordinates), set the connections at the ends of the front wall and piles, assign necessary rigidness to elements.

2. Introduce initial data in the program in the form of loads on the covered type of sheet pile wharf

3. Introduce in each element parts the schemes needed according to supporting conditions of the horizontal foundation modulus (ground stiffness) amount.

4. Set up a basic load combination and make a linear analysis.

5. In graphical analysis print out the results in the form of: bending moment diagram M, shearing force diagram Q, axial force diagram N, reactive soil pressure R and the scheme of construction deformation.

Example 2 given below is performed in accordance with the recommendations of the work [27].

Table 1. Lateral pressure on the shielded pile row and the front wall

Silage pressure on the pile row (The diagram from the left, (Fig. 4a))	**Resulting pressure on the pile row (See the diagram right to solid line), (Fig. 4a)**	**Resulting pressure on the front wall (Fig. 4b)**
$\sigma_{sil,1} = 0$; $\sigma_{sil,2} = 0$; $\sigma_{sil,3} = \gamma_1 h_2 \lambda_{a,1}$; $\sigma_{sil,4} = (\gamma_1 h_2 + \gamma_2 h_3)\lambda_{a,1}$; $\sigma_{sil,5} = \sigma_{sil,4}$; $\sigma'_{sil,5} = \sigma_{sil,5}\lambda_{a,2}/\lambda_{a,1} - \sigma_{ac}$; $\sigma_{sil,6} = \sigma'_{sil,5}$; Angle $a = (45°-0.5\varphi_1)$; σ_{sh} – pressure on the pile row by backfill soil ; σ_{sil} – reverse silage pressure on the pile ro from water side.	1. $\sigma_{sh,1} = 0$; 2. $\sigma_{sh,2} = 0$; 3. $\sigma_{sh,3} = \gamma_1 h_2 \lambda_{a,1} - \sigma_{sil} = 0$; 4. $\sigma_{sh,3}' = [q + \gamma_1 (h_1 + h_2) + \gamma_2 h'_3]\lambda_{a,1} - \sigma'_{sil,3}$; 5. $\sigma_{sh,5} = [q + \gamma_1 (h_1 + h_2) + \gamma_2 (h'_3 + h_4)]\lambda_{a,1} - \sigma_{sil}$; 6. $\sigma'_{sh,5} = \sigma_{sh,5}\lambda_{a,2}/\lambda_{a,1} - \sigma_{ac} - \sigma_{sil,5}$; $\sigma_{ac} = 2c \cdot \mathrm{tg}(45° - 0.5\varphi_2)$ – friction value; 7. $\sigma_{sh,6} = (\sigma'_{sh,5} + \gamma_2 h_5)\lambda_{a,2}$; $\lambda_{a,i}$ – coefficient of active earth pressure considering the force of friction on the wall;	1. $\sigma_{sh,1} = q_1 \lambda_{a,1}$; 2. $\sigma_{sh,2} = (q_1 + \gamma_1 h_1)\lambda_{a,1}$; $\sigma'_{э,2} = 0$; 3. $\sigma_3 = \sigma_{sil,3} + K_{al}\sigma_{sh,3}$; 4. $\sigma_3' = \sigma'_{sil,3} + K_{al}\sigma'_{sh,3}$; 5. $\sigma_5 = \sigma_{sil,5} + K_{al}\sigma_{sh,3}$; 6. $\sigma'_5 = \sigma_{sil,5} + K_{al}(\sigma_{sh,5} - -\sigma_{ca})$; 7. $\sigma_{,6} = \sigma_{sil,6} + K_{al}(\sigma_{sh,6} - \sigma_{ca})$; K_{al} – coefficient of distribution of additional load on the front wall from a pile row.

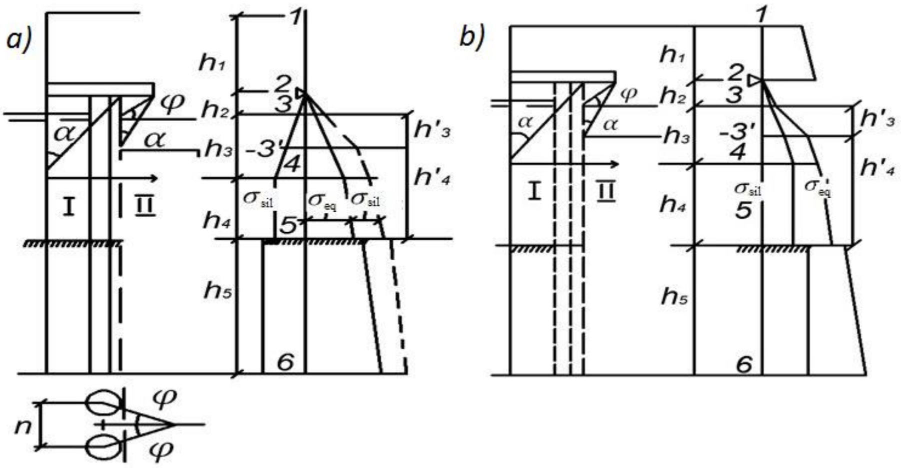

Figure 6. Scheme of loads on pile row: a) Silage pressure on the pile row (see the diagram from the left) and resulting pressure on the pile row (see the diagram right to solid line); b) Resulting pressure on the front wall

Results

Example 2. Calculate the covered type of sheet pile wharf with height of 13.5 m made of pipe pile with diameter 720 mm. The pile row consists of mantle pipes with diameter 800 mm which are immersed in increments of 2.0 m. The relieving platform 4.0 m-width is located on the pipes. The backfill ground is medium sand $\varphi = 31^0$, foundation soil - semisolid loam $I_L = 0.4$, $\varphi = 27°$, $c = 8$ кPa.

The calculation results of the covered type of sheet pile wharf in the SCAD program are given on Figure 5.

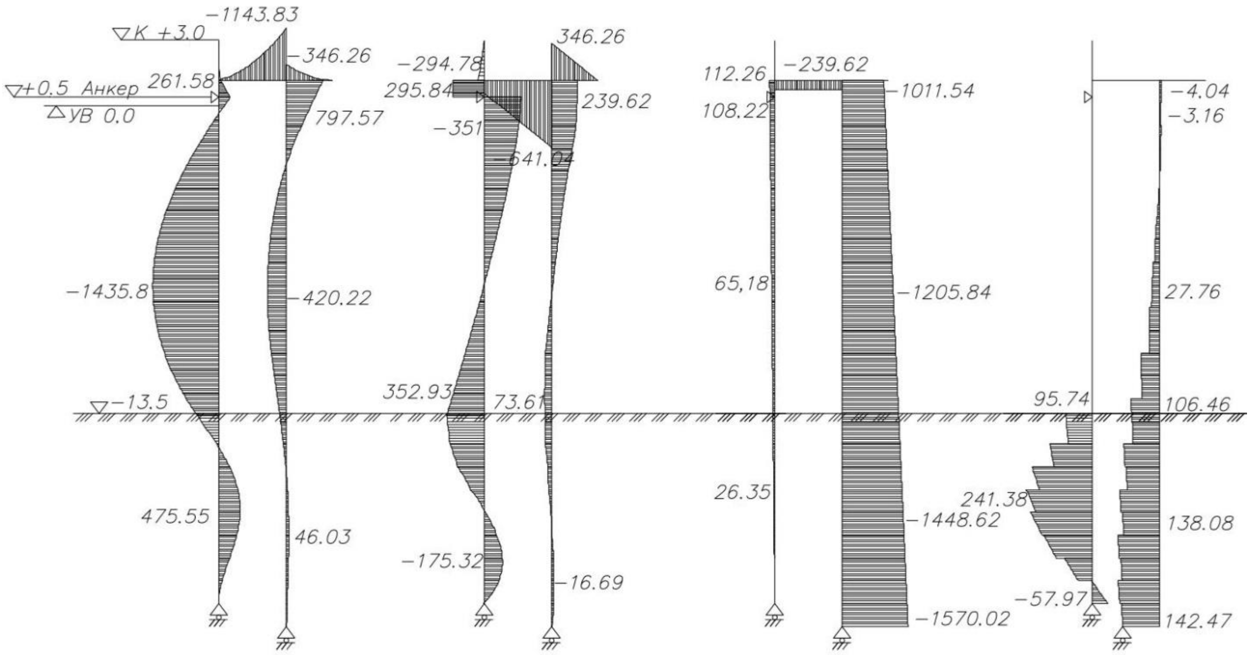

Figure 5. The diagram of internal forces in the elements of the covered type of sheet pile wharf. Conventional signs: a) Bending moment diagram; b) Shearing force diagram; c) Axial force diagram considering the weight of pipes and sand inside it; d) Diagrams of reactive soil pressure on the frame pillars.

The calculation results of the Example 2 are given in the Table 2.

Table 2. Comparative table of internal forces value in the covered type of sheet pile wharf

№	Name of the internal forces	The proposed method	The existing method [6]
1	Maximum bending moment in the pipe pile, kNm	1435.8	1250.0
2	Maximum bending moment in the pile row, kNm	470.22	750
3	Lateral forces, kN		
	The pipe pile	352.9	-
	The pile row	239.6	-
4	Axial force, kN		
	The pipe pile	112.26	-
	The pile row	1011.34	931.7
5	Maximum bending moment in the relieving platform, kNm	1143.83	1500.0
6	Maximum deflection in the pipe pile, mm	60.75	-

Discussion

The calculation of covered type of sheet pile wharf in the form of substandard rack-mount frame is an improved particular case of N.M. Gersevanov method in respect to the pile foundation grillage.

The article gives an engineering solution for definition of diagrams of lateral pressure on the front wall taking into account the redistribution of pressure on pile row.

The article gives a practical implementation of previously proposed engineering multi-purpose method for calculation of berths. The calculation model is proposed in the form of substandard rack-mount frame where resilient ground attachment is accepted instead of rack lower anchorage. This model uses stiffness characteristics of the ground in the form of variable foundation modulus.

Comparative variants of calculations of proposed and existing methods have shown a significant impact of ground deformation characteristics on berth elemental forces in proposed method in comparison with the existing method [6].

Conclusions

1. In the existing method a maximum bending moment in the pipe pile and pile row is determined by grapho-analytical method. Maximum bending moment in the pipe pile without shielding, M_{pp} = 2053.0 kNm.

2. Maximum axial force considering the increasing weight of pipe and soil in it 1570.0 kH.

The difference in bending moment diagram in the elements of covered type of sheet pile wharf in comparative Table 2 is connected with the fact that the deformation-free method satisfactorily describes the work only in flexible walls in dense soils base. In extra stiff wall according to the proposed method, the moment of anchorage appears inconspicuous.

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