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Computation of mooring quay in the form of pile grillage

Расчет причальной набережной в виде свайного ростверка

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Ключевые слова: гидротехническое строительство; конструкция; высокий свайный ростверк; метод расчета; коэффициенты постели

Abstract. The calculation of a pile grillage quay, as a frame with rigid anchored legs, using in technical literature and normative documents, is called a method of Gersevanov. The transformation of a manual calculation to a computer calculation almost preserves an existing design diagram. The article presents the practical implementation of a proposed early an engineering universal calculation method for quays. In the proposed design diagram of a frame with an elastic anchorage of the bases of frame legs is used instead of the rigid anchorage. It uses stiffness characteristics of soil as the variable coefficients of the subgrade reaction. An engineering solution of determining a diagram of a lateral earth pressure in a silo of variable width and the engineering calculation method of the pressure redistribution on the back sheet pile wall from external ground pressure on the inclined pile row are showed in the article. Taking into account the deformation characteristics of soil the comparative calculations showed their significant impact on the internal forces in the elements of the quay in comparison with the N.M. Gersevanov's method.

Аннотация. Расчет свайной набережной ростверкого типа в виде стоечной рамы с жесткими заделками стоек, используемый в технической литературе и нормативных документах, представляет метод Н.М. Герсеванова. Перевод ручного расчета на компьютер практически сохранил существующую расчетную схему. Приведена практическая реализация, предложенного раннее инженерного универсального метода расчета причальных набережных. В предлагаемой расчетной схеме стоечной рамы вместо нижней заделки стоек принимается упругая заделка. При этом используются жесткостные характеристики грунта, в виде переменных коэффициентов постели. Получено инженерное решение определения эпюры бокового давления в силосе переменной ширины, показан инженерный прием вычисления перераспределенного давления на заднюю шпунтовую стенку от внешнего давления грунта на наклонный свайный ряд. Сопоставительные расчеты с учетом деформационных характеристик грунта показали их значительное влияние на усилия в элементах набережной по сравнению с методом Н.М. Герсеванова.

Introduction

The research object is a construction of the high pile grillage quay. This construction was used extensively in the last century in USSR and now used abroad. Increasing demand of quay constructions in the form of the high pile grillage is associated with increase of depths near quays. In addition, the use of reconstruction variants of urban quays in the form of the pile construction with inclined piles is more economical compared to a low pile grillage. The transverse section of the quay in the form of the high pile grillage with the back sheet pile wall is shown in Figure 1, a.

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A significant amount of works by Russians and foreign scientists describe the development of rational methods of the static calculation of the pile grillage quays.

In the first time, N.M. Gersevanov gave the general solution of the calculation problem of the pile quays with the rigid grillage, treating them as a frame with an absolutely rigid beam on elastic supports – piles with anchored bases (1913). Equating of the unknown displacement of the grillage lower edge (vertical, horizontal, and its rotation) N.M. Gersevanov received three canonical equations of the deformations method. Their solution allowed passing from a displacement of grillage to internal forces in the piles. These equations are usual conditions of the equation of the rigid body equilibrium.

Except the analytical method of calculation developed by N.M. Gersevanov, Russian scientists B.M. Lozovsky, F. Dimentbergom, V.S. Hristoforova proposed the solution of the same problem by graphical methods. The calculation methods developed by Vyunshem and Nekkentvedom are special cases of solutions of N.M. Gersevanov. F. Dimentberg developed the method of calculating of the pile quay with regard to a stage of destruction.

G.S. Spiro developed calculation of spatial pile constructions with the rigid grillage as the development of the calculation method of N.M. Gersevanov. Note, that the solutions proposed above are associated with a transfer of the origin of coordinates in the elastic center, which reduced the number of the canonical equations of the deformations method.

In recent years works associated with the calculation of the pile grillages, usually use the classical design diagram [1–3]. This definition of piles anchorage in the ground cannot be correct because it uses Blum-Lomeyera's method, which does not take into account the relationship between the pressure and the displacement of a pile in the ground. For this reason, the passive pressure on the foundation to the vertical wall is untrue.

Note the works of foreign scientists on the calculation of the pile grillage quays [4–16]. In the last 10–15 years abroad, the calculation of the berthing facilities is performed in the program Plaxis, using the finite elements method for the continuum model. This model developed for construction materials is not exact for soil. It gives the approximate picture of the behavior of the pile foundation in the discrete soil medium.

Review modern status of the problem shows that the solution N.M. Gersevanov is used in technical literature and normative documents for the calculation of the pile grillage construction as frame with rigid anchored bases of piles. At that, an appointment of the depth of the anchorage of piles is arbitrarily and determined by the method of elastic line does not match an actual structures work. Assessing an originality of the calculation method of grillage-type structures, which was proposed N.M. Gersevanov, note that this method misrepresents a design diagram of structure because it is the deformation-free method for an interaction of pile foundation in ground. This introduces a certain imprecision in a determination of internal forces in elements of a construction.

In existing technical literature [17–19] and normative documents [20, 21] each type of a port hydraulic engineering structure has its own calculation method. The transformation of a manual calculation to a computer calculation reduced substantially complexity of computing works, but did not use completely great automation capabilities, because most of calculation methods are based on provisions and assumptions, which were laid in the era of lack of computers.

The calculation of mooring constructions in software systems which using the model of continuous medium model allowed to obtain more complete picture of working constructions in soil. However, the calculation of these constructions in programs, which use the model of continuous medium, is designed for structural materials and is not strict. It gives the approximate picture of a behavior of the pile foundation in discrete soil medium. It is required the additional verification of the calculation results, due to a wide values range of the initial data and in cases of adoption of rigid anchored bases of piles in ground [22–24].

In 2002, the universal engineering method of calculation was offered to simplify the understanding of a mechanism of interaction elements of a structure with soil and use a computer maximally. This method can be applied to any construction of a quay [25] and complemented in the work [26].

In the settlement relation, every type of the quay structure is submitted in the form of the specific combination of beams interconnected by condition of a joint deformation of individual elements and by common structure, each of which has different forms of interaction between themselves. This approach, on the one hand, unifies calculation methods and, on the other,– respectively allows refusing from some simplifications adopted in existing calculations, for example, in the frame type of quays, the pile grillage type: rigid anchored bases of piles in ground and a hinge support in the grillage, a limit lateral pressure and etc.

The purpose of the proposed work is the practical implementation of the engineering universal method for the calculation of the quay grillage constructions. At the same time the design diagram and impact of assumptions, used in the departmental standards, on internal forces in constructions are measured in this work.

Theoretical background

Calculation model

Structure dimensions of the non-rigid grillage are defined in initial data [19]. The determination of internal forces in piles is necessary for determination burial depths of the piles. The characteristic feature of the non-rigid girder is the distribution of vertical and horizontal loads on supports through a girder. It is considered as a simply supported beam. The pile foundation is not represented as an integral unit but as a system of independent supports united only by the non-rigid girder. The vertical pressure on every pile or the inclined pile is defined as reaction of the simply supported beam. The horizontal loads (H) are equal with the resultant active pressure on the vertical, passing through the back. The value of the horizontal loads is usually absorbed partially by single inclined piles and mainly – paired inclined piles.

Obtained vertical and horizontal forces from the condition of the non-rigid grillage determine the cross section of elements of the high pile grillage previously. Subsidence of the piles is determined previously by values of internal forces in compression elements and the value of the coefficient of subgrade reaction on the pile tip.

In the calculation in the program SCAD the high pile grillage is represented as the statically indeterminate frame of spacer type with spatial elements partially submerged in foundation soil, which is described by a model with two coefficients of subgrade reaction (horizontal along the length of piles and vertical at the bottom end of pile) (Fig. 1, b).

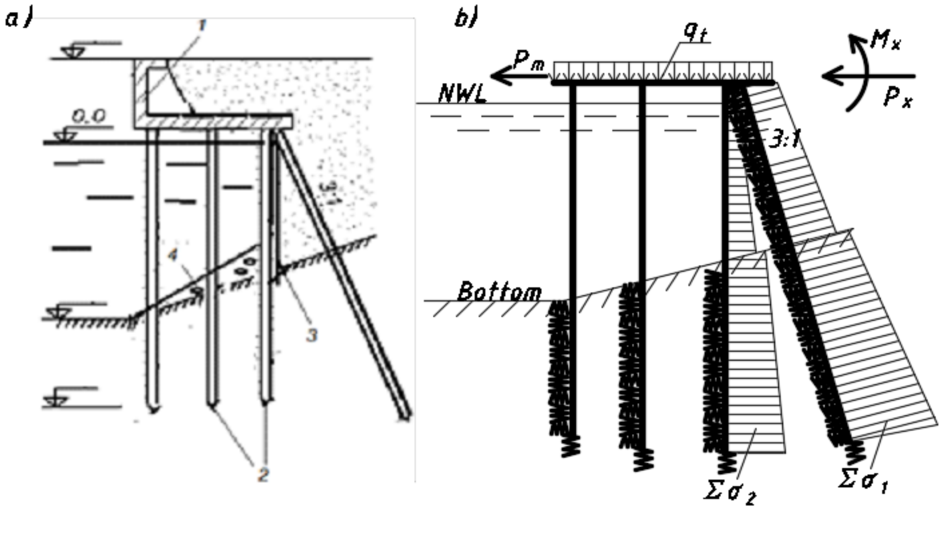

Figure 1. High pile grillage with back sheet pile wall: a –transverse section: 1 – grillage;2– piles;3 – protection of soil from dornit; 4 – riprap; b – design diagram. The legends: q_t – total vertical load;P_m – mooring load on the one running meter (1 rm); P_x, M_x – resultant active pressure on the grillage (superstructure) and moment from its; $\Sigma\sigma_1$, $\Sigma\sigma_2$ – diagrams of resulting pressure on the sheet pile wall and inclined pile. The darkened part of the piles' elements of the construction in ground (in the form of springs in two directions) represents the use of the generalized model of Fuss-Winkler

A geometric image of the design diagram is performed by entering nodes of the frame elements of the design diagram, and then – image of elements themselves (entering elements). Each element of the design diagram is introduced individually. It is recommended to assign the origin of coordinates at the first node.

The design diagram of frame is set by plots' coordinates of reference nodes. The spatial framework is chosen in the calculation of the framework. It is recommended to divide front wall sections, which interact with ground through the variable height coefficient of the subgrade resistance, into elements.

Front wall sections interacting with ground through the variable height coefficient of the subgrade resistance it is recommended to split into elements of a width of 1.0 m and of 2.0 m for remaining sections.

The numerical description of stiffness characteristics is used in this work. The longitudinal and flexural stiffness of frame element EI contains the deformation module of steel $E = 2.1 \times 10^8$ kN/m^2 and the deformation module of reinforced concrete $E = (2.7 – 3.67) \times 10^7$ kN/m^2, an area A, m^2 and a moment of inertia I, m^4 on one meter-wide. The stiffness of the pile row is divided into a number of pile steps.

Deformation characteristics of soil

The Fussa-Vinkler's model became widespread in the practice of port design in the calculation of thin quay walls. This model uses the deformation characteristic of soil in the form of the coefficients of subgrade resistance. In addition, this model has been criticized for neglect of the soil distributive capacity. However, the revision of the continuum model, which is used in program complexes with data of natural researches, showed that the influence of distributive capacity of soil, which has discrete structure, is overestimated.

The solution of the differential equation of the bending beams leaning on the continuous (winklerovsky) elastic foundation, according to the method of local deformations, does not represent special difficulties, but contains four integration constants, which need to be defined from initial conditions.

Direct experiences show that the coefficient of subgrade resistance for natural soil is not constant, but depends on the value of specific pressure upon ground and the area of transfer of loading that it is necessary to consider at calculations.

Generalized methods of the definition of soil deformations consider general elastic and local inelastic soil deformations. Of these methods, we note P. Pasternak's and V. Vlasov's methods of the two-parameter elastic foundation according to which the soil foundation is characterized by the coefficients of subgrade resistance of compression and local elastic shift. Let us note I. Cherkasov's and G.K. Klein's method of structural-revitalizing deformations, which considers general revitalizing deformations (elastic and adsorptive) and residual (structural). In the latter method, revitalizing deformations are accepted as linearly deformable and characterized by the coefficient similar to the coefficient of elastic half-space.

The calculation theory for structures lying on the deformable foundation with one and two coefficients of subgrade resistance was developed in works, which was made by: A. Dinnik, P. Pasternak, M. Gersevanov, N. Zhukovsky, A. Krylov, A. Umansky, G. Dutov, V. Kiselyov, S. Golushkevich, B. Korolyov, N. Snitko, V. Vlasov, N. Leontyev and many others. Among foreign scientists engaged in this task: H.M. Westergaard, H. Bufler, H. Lieb, G. Meier [27], Y.K. Cheung, O.C. Zienkiewicz [28], C.S. Desai, J.T. Christian[29], A.M. Ioannides, etc.

Due to changes of the coefficient of subgrade resistance on depth (for piles) and in plan (for slabs) used the so-called coefficient of soil stiffness. In the case of the calculation of different types of mooring facilities is proposed to use function of the stiffness coefficient of foundation that connects the stiffness coefficient of soil from elastic and limit states for the entire load cycle [30]. In practical use this function is piecewise linear values in each section of the structural element of five intervals of the entire load. The coefficient of subgrade resistance of foundation for the sheet pile wall considering plastic properties of soil is:

$$К_s = y \cdot К \cdot К_a \cdot К_{pl}, \qquad (1)$$

where y – depth of a point under consideration;К–a proportionality coefficient(ED 31.31.55-934); $К_a$ – an anisotropy coefficient for non-cohesive soils is 1, for clay – 0.7–0.8; $К_{pl} = 0.6 – 0.8$ – a coefficient account of the plastic properties of foundation in front of a wall are accepted for burial depths 1–2 m (lower value for loose and weak soils).

For the pile elements are used calculated values of the coefficient of subgrade resistance on the lateral surface of the pile and on a tip of the pile are accepted to JV 24.13330, Pile foundations, Moscow 2011.

Side loads from ground on gantry elements piles

Main lateral loads on the pile grillage with the back sheet pile wall and the gantry sheetpile bulkhead operate on the vertical sheet pile row and the inclined anchored piles with slope of 3:1

(Fig. 2, a). Note that the gantry sheetpile bulkhead is obtained by discarding two front piles in the pile grillage (Fig. 2, a).

Pressure of the ground zone II on the inclined pile row represents a sum of lateral pressure on the inclined plane (σ_{aa}) and the load pressure of soil hanging on piles (σ_h) (Fig. 2, a).

The active pressure on the inclined plane is equal to:

$$\sigma_{aa} = (q + \sum \gamma_i h_i) \cdot \lambda_{aa}, \tag{2}$$

where λ_{aa} – a coefficient of a lateral pressure on an outer row, when $\alpha = 18^0, 25^{II}$.

The active pressure from soil hangs on pile equal (Fig. 2, b):

$$\sigma_h = (q + \sum \gamma_i \cdot h_i) \cdot m \cdot \sin^2 \alpha, \tag{3}$$

where m = 2·d/n·ctgφ (d and n – a diameter and the pitch of piles); $\alpha = 18^0, 25^{II}$ – the angle of inclination to the vertical of the pile.

The active resultant pressure on the pile row is difference of the pressures on its outward from zone II and the zone with internal I (Fig. 2, b):

$$\sigma_r = \sigma_{aa} + \sigma_h - \sigma_i = (q + \sum \gamma_i \cdot h_i) \cdot \lambda_{aa} + \sigma_h - \sigma_i, \tag{4}$$

where $\sigma_i = (\sum \gamma_i \cdot h_i)\lambda_{\text{-}aa}$ – the internal pressure on the inclined rear piles row to the angle α.

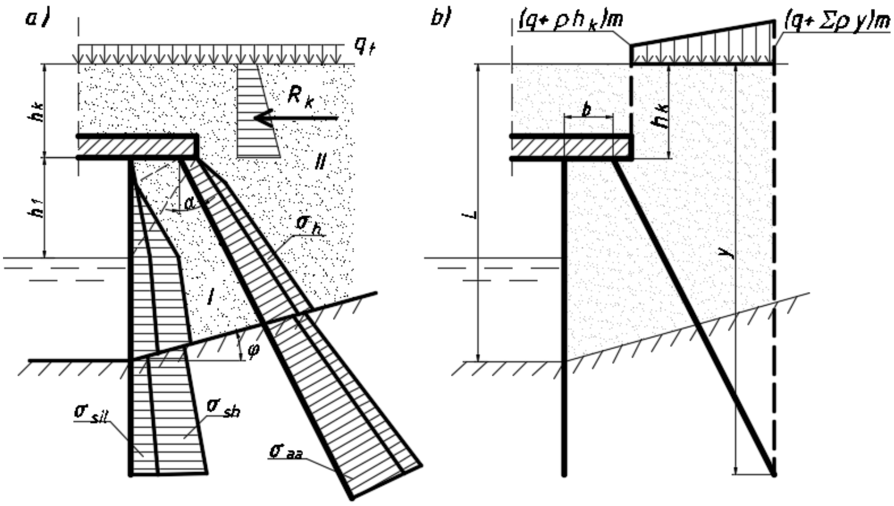

Figure 2. a – diagrams of the lateral pressure on the gantry elements of the piles. Conventions: R_k – resultant of active pressure on the grillage; σ_{sil} – pressure on the sheet pile wall in the conventional silo; σ_{sh} – additional pressure on the sheet piles from the shielding pile row; σ_{aa} – active pressure on the inclined plane of pile row; σ_h – the pressure from the load of soil hanging on piles; b – the load on the inclined anchor piles

The soil pressure in the zone I upon the sheet pile wall of the pile grillage or the gantry sheetpile bulkhead consists of the soil pressure in this zone (conventional silage, σ_{sh}) and the resulting additional pressure of the pile row (σ_r) considering distributing capacity of the piles.

The lateral pressure on the sheet pile wall represents a sum from a conditional silage pressure σ_{sil} in the zone I and the additional pressure from a shielding pile row (σ_{sh}). The last is equal to a difference of the external pressure σ_{aa} from the zone II and a return silage pressure upon the shielding inclined plane of soil σ^1_{sil} in the zone I with considering of the distribution coefficient K_d (Fig. 3, a):

$$\sigma_{shp} = \sigma_{sil} + \sigma_{sh} \tag{5}$$

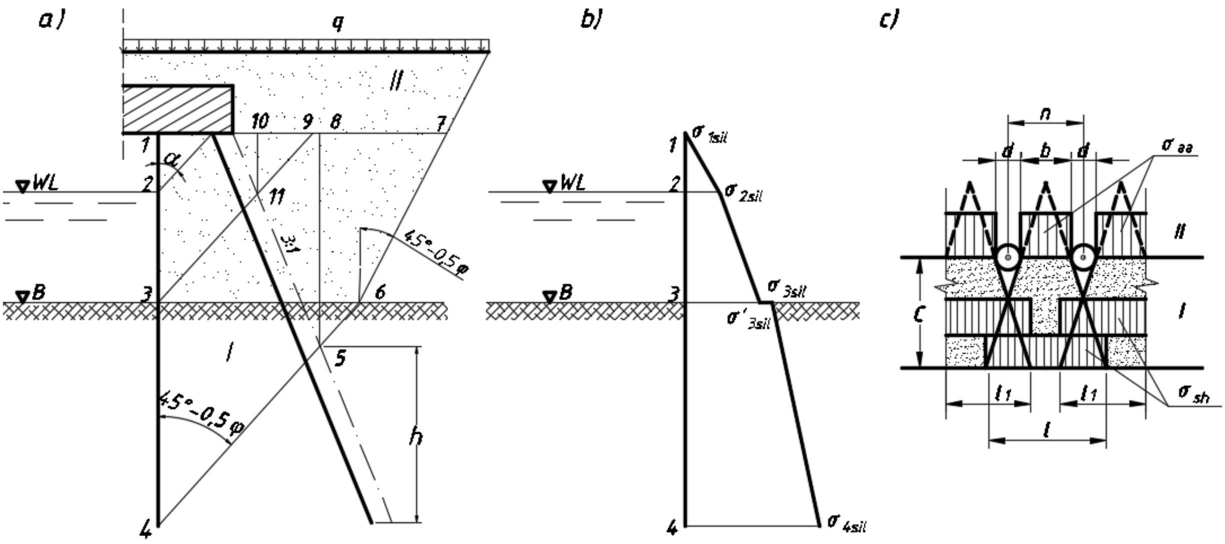

Figure 3. Design diagrams for determination of lateral pressure in zone I:
a – simplified diagram; b –specified diagram defining intensity from a condition of the maximum plane of a collapse; c – determination of pressure of a pile row upon the sheet pile wall

The lateral pressure in the silo of variable width. The lateral pressure in the point 2, determined the straight line at the angle 45° – 0.5φ to a vertical from the beginning of the shielding plane of the piles, is equal (Fig. 3, a):

$$\sigma_{2sil} = \gamma\, h_{1\text{-}2}\cdot\lambda_{a\text{-}1}, \tag{6}$$

where $h_{1\text{-}2}$ – a height between points 1 and 2 (Fig. 3, a).

The lateral pressure in the silo of variable width in the points 3 and 4 is defined by the following engineering reception with a margin of safety (Fig. 2, b). Presume that the active pressure in the point 3 is defined by a pressure of the incomplete wedge (prism) of failure 1, 3, 11, 10, and the pressure in the point 4 respectively the prism 1, 4, 5, 8. Taking into account this circumstance the lateral pressure 3 and 4 is equal (Fig. 3, a):

$$\sigma_{3sil} = \gamma_2\cdot h_{2\text{-}3}\cdot\lambda_{a1}; \tag{7}$$

$$\sigma^1_{3sil} = \gamma_2 h_{2\text{-}3}\cdot\lambda_{a2} \tag{8}$$

$$\sigma_{4sil} = \gamma_2\cdot h_{4\text{-}5}\cdot\lambda_{a2}, \tag{9}$$

where $h_{2\text{-}3}$ – a height between points 2 and 3 (Fig. 3, a); $h_{4\text{-}5}$ – a height between points 4 and 5 (Fig. 3, a).

Soil adhesion is not considered in the margin of safety. In case it appears that $\sigma_{3sil} < \sigma_{2sil}$, the value σ_{3sil} is determined by the straight line passing through the values σ_{3sil} and σ_{4sil}.

In determination, the additional pressure upon the sheet pile wall σ_{sh} from the shielding pile row in departmental norms [31] was used similar N.A. Smorodinsky's ideas (1937) about a distribution of the pressure upon the sheet pile wall and the pile is proportional to their rigidity. This offer was progressive at the beginning of origin of calculations of the pile constructions, but does not consider influence of distance of the sheet pile wall and also the cross section of piles and the step of the pile row along a line of a cordon. These factors have more essential impact on the sheet pile wall and piles than a ratio of their rigidity. The simplified decision for the definition σ_{sh} by the distribution coefficient K_d is given [32] below. The pressure upon the sheet pile wall from the zone II is calculated depending on the step of piles through K_d:

$$\sigma_{sh} = K_d\cdot(\sigma_{aa} - \sigma^1_{sil}); \tag{10}$$

$$K_d = (b/\mathrm{n})\cdot(2\cdot l_1/l), \tag{11}$$

where n – the step of the piles; $l_i = c_i\cdot\mathrm{tg}\,\varphi$ – a variable zone of the partial distribution of resultant load behind the sheet pile wall (c_i – a variable distance from the pile to the sheet pile wall); $l = \mathrm{b} + 2\cdot c_i\cdot\mathrm{tg}\varphi$ – a variable zone of distribution of the resultant load between the site b piles of the sheet pile wall; d – the diameter of the shielding pile (Fig. 3).

Results

Initial data of calculation grillage: the load on surface of the backfill q = 40 kN/m^2; the free height of the wall H = 12.5 m, R/C sheeting pile 50 x 50 cm, the cross section of the anchoring piles 40 x 40 cm, the step of piles n = 1.5 m, the incline of piles 3:1 (α= 18°25'), h_p = 2.5 m.

Design characteristics of foundation soil: banded clay γ_w= 10 kN/m^3, φ = 20°, c = 30 kPa; backfill soil: γ = 18 kN/m^3, γ_w = 10 kN/m^3, φ = 30°. The elastic modulus of the construction material E_{sh} = E_s= 2.1·10^{4}MPa. Deformation characteristics of backfill soil and the foundation in the form of the horizontal and vertical coefficients of subgrade resistance for the sheet pile wall and the piles were taken using normative documents.The calculation results are shown in Table 1 and Figures4–6.

Table 1. The results of calculation the quay using two methods

Name	The calculation by the classical method [31]	The calculation of the proposed method in the program SCAD[25]
Moment in the top anchorage of sheet piles, [kN·m]	-67.48	-374.35
Moment in the top anchorage of the piles, [kN·m]		
The first series of piles	-56.96	-223.48
The second series of piles	-36.36	275.48
The third series of piles	125.76	-118.1
Moment in the span of sheet piles, [kN·m]	-76.90	363.34
Moment in the span of piles, [kN·m]		
The first series of piles	-5.745	86.05
The second series of piles	6.03	76.40
The third series of piles	198.60	132.45
Moment in the bottom anchorage of sheet piles, [kN·m]	148.17	0.00
Moment in the bottom anchorage of piles, [kN·m]		
The first series of piles	45.47	-
The second series of piles	48.42	-
The third series of piles	-526.81	-
Longitudinal force in the vertical pile, [kN]		
The first series of piles	-177.64	-334.34
The second series of piles	-173.39	-279.65
Longitudinal force in the inclined pile, [kN]		
The third series of piles	327.18	6.5
Maximum vertical displacement of the structure, [mm]	-2.4526	11.6
Maximum horizontal displacement of the structure, [mm]	-8.13	-65.40

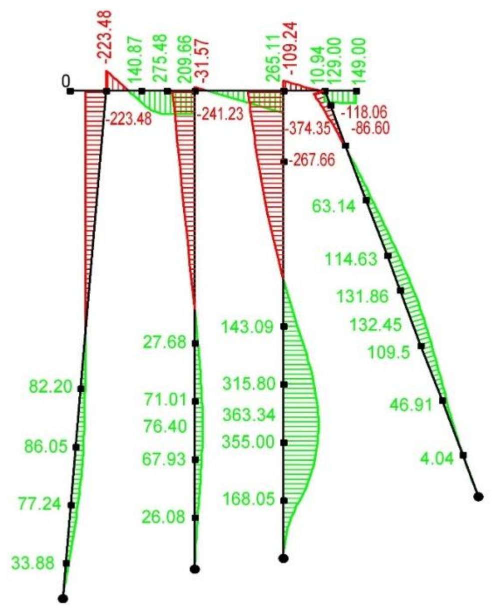

Figure 4. Diagram of bending moment in the pile grillage, kN·m [25]

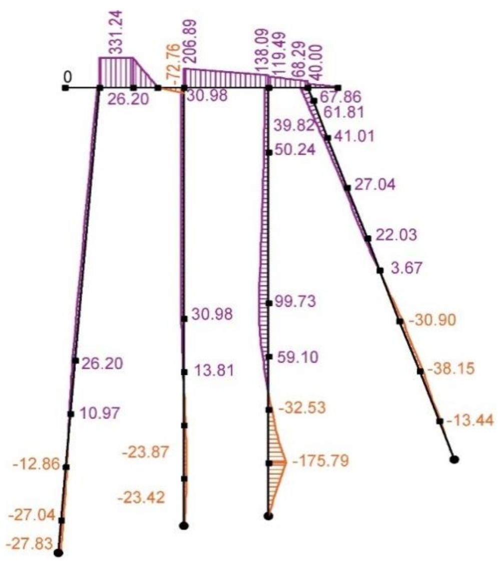

Figure 5. Diagram of shear forces in the pile grillage, kN [25]

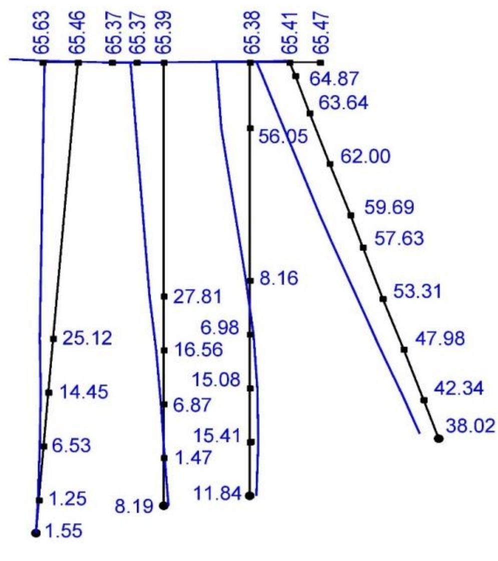

Figure 6. Diagram of displacements of the pile grillage, mm [25]

Discussion

The comparison of the calculation results by two methods revealed their lack of comparability quantities of internal forces in elements of the structure. It arose due to discrepancies of the design diagrams in the considered methods. Therefore, in the calculation, based on [31], the internal forces in the rigid anchored frame legs arise from the ground active pressure on them and from the entire structure rotation about the bottom fixed end. In the calculation, based on [25], the rigid anchorage of the frame legs is not used, but the reactive ground pressure on the inclined pile in the space between piles begins to prevent turn the whole structure. At the same time, the bending moment diagram in the pile can change a direction depending on the stiffness properties of ground.

The universal method with the variation of the horizontal and vertical coefficients of subgrade resistance (without them in filling) upward allows to receive more or less commensurable values of internal forces with departmental regulations. However, in this case, the real stiffness characteristics of foundation soil are distorted.

Conclusions

1. In the existing technical literature and normative documents each type of the port hydraulic structure has own calculation method. The calculation of the pile grillage quay as the frame with absolutely rigid beam on elastic supports-piles with the rigid bottom anchorage is the special solution (deformation-free in ground) of N. Gersevanov’s classical method. The transformation of the manual calculation to the computer calculation reduced substantially complexity computing works, but part of calculation methods is based on provisions and assumptions, which were laid without computer.

2. Shows the practical implementation of the proposed early engineering universal method of the mooring grillage quay design. The calculation used stiffness characteristics of the framework beam, piles and soil. There is no soil in the classical method.

3. It is defined the engineering solution of the determination diagram of the lateral pressure of soil in the silo of variable width.

4. It is showed the engineering reception of the calculation of the redistribution pressure on the back sheet pile wall from the ground external pressure on the inclined pile row.

5. The comparative calculations of the mooring quay with different values of the stiffness characteristics of soil, which interacts with the elements of piles, showed their significant impact on the classical design diagram of construction. In the case of the flexible frame legs and the solid foundation soil (excluding the stiffness of backfill soil), internal forces are comparable with the method of Gersevanov, which can be considered as the special case of the proposed method. In other cases, in the proposed method the internal forces in the elements of quay change significantly upwards or downwards in comparison with the classical calculation method.

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