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Deformation sheet piles of coastal protection structures, taking into account the effect of their self-weight

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Abstract:

The object of research is the design of structures used in hydraulic structures in contact with the soil environment are primarily associated with the mechanical properties of soils. Without this, it is impossible to form the conditions of the contact problem, to raise the question of stresses and deformations arising in structures, and also to determine the regularities of the distribution of reactive pressures. **Method.** The development of effective engineering methods for calculating dock structures in hydraulic structures, based on real design schemes, most fully reflecting the joint work "structure - foundations" can be solved by the Fuss-Winkler hypothesis. The advantage of this method lies in the fact that by introducing special functions called "functional breakers", proposed by Professor N.M.Gersevanov, made it possible to bring the solution to practical results. **Results**. The use of this method makes it possible to determine the deformations and forces of the walls of coastal protection structures, dock, chute structures of spillway structures, which are widely used in hydraulic engineering structures, taking into account the effect of their self-weight on lateral loads.

1 Introduction

When assessing the stress-strain state (SSS) of sheet pile walls of coastal protection structures, various limiting design cases are considered. According to these provisions, in each specific case of the calculation, the combination of acting loads and actions that is most disadvantageous for the "structure-base" system is established [1-6].

The object of research is a flexible sheet pile wall, which are widely used in berthing, coastal, dock and other structures. These structures are subjected to various loads and influences during operation. The calculation (SSS) of these structures is carried out using the Fuss-Winkler model for soils with a variable coefficient of soil stiffness [7-9].

To determine the stress-strain state (SSS) of structures on the basis of various computational-mechanical models were considered in the research of the authors [10-24].

The subject of the research is the calculation (SSS) of a flexible structure in contact with the soil medium, taking into account the influence of the structure's self-weight. The influence of the distribution load from the wall's self-weight is determined using the functional breakers proposed by Professor N. M. Gersevanov [25-27].

This calculation technique allows for a specific case to obtain the calculated expressions for the shear and angle of rotation of a flexible sheet pile wall.

This calculation method allows to determine the internal forces, on the basis of which it is possible to determine the rational need for reinforcement and the amount of concrete.

2 Materials and Methods

Below we consider the case when the water rise level is at the maximum level, and in the backfilling of the walls, the groundwater level is the lowest possible. The following main loads and effects are taken into account: the hydrostatic pressure of water, the wave load transmitted to the upper outer part of the wall, as well as the self-weight of the wall, the intensity of which is assumed constant along its height.

When calculating (SSS) flexible structures, Professor N.M. Gerasevanov, using the introduction of special functions called "functional breakers", found an opportunity to bring the solution to practical results.

In the design scheme, the sheet pile wall is represented in the form of a flexible console with a constant bending stiffness of the lower end, which is rigidly sealed. The backfill surface is horizontal and coincides with the water level in front of the tongue [7-9].

The wave load is taken into account by applying a concentrated force Q_0 and concentrated moment M_0 to the upper end of the console.

The intensity of the resulting plot of hydrostatic water pressure from the top of the console to the water level in the backfill changes linearly, and below this level it remains constant (Fig. 1).

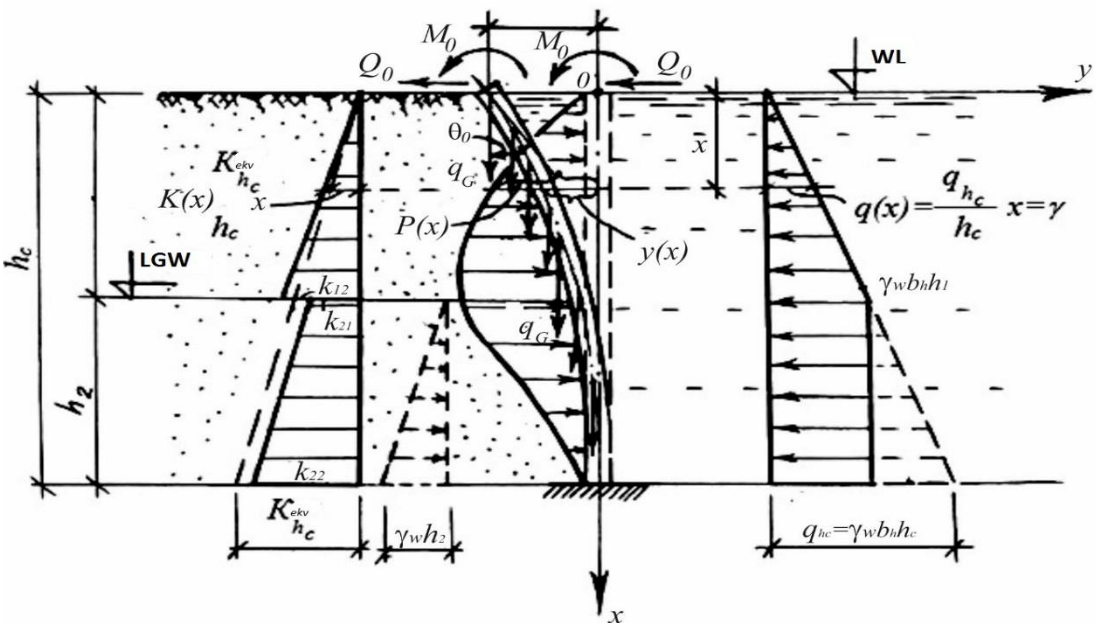

Fig. 1 – Scheme for calculating a flexible sheet piling taking into account its self-weight. (Figure by the author)

Under the action of the resulting hydrostatic water pressure *q(x)* and concentrated wave loads from Q_0 and M_0, the console moves towards the backfill. [15-19].

To determine the reactive resistance of the backfill soil to horizontal movements of the console, the Winkler-Fuss model is used, characterized by the coefficient of stiffness linearly varying in depth

$$P_s(x) = -K(x)Y(x) \tag{1}$$

where $P_s(x)$ is backfill soil reactance; *Y(x)* is movement (deflection) of the console at an arbitrary depth *x*; *K(x)* is the coefficient of stiffness of the backfill soil, taken on the basis of the Winkler model can be represented as:

$$K(x) = \frac{K_{hc}}{h_c} x \tag{2}$$

Here, K_{h_c} is the value of the stiffness coefficient of the backfill soil at the level of the lower end; h_c is console height (Fig.1.).

Taking into account (1) and (2), we have:

$$P_s(x)=\frac{K_{hc}}{h_c}xY(x) \tag{3}$$

3 Results and Discussion

As can be seen from the figure, the flexible wall is influenced by the bending moment and the shear force, which vary along the length of the wall.

Let us dwell on the derivation of the differential equation of the problem.

Bending moment and shear force in an arbitrary section of the cantilever:

$$\begin{cases} M(x)=M_0+Q_0x+M_q(x)+M_G(x)+M_s(x) \\ Q(x)=Q_0+Q_q(x)+Q_G(x)+Q_s(x) \end{cases} \tag{4}$$

Where $M_q(x)$ and $Q_q(x)$ is respectively, the bending moment and the shear force from the transverse load *q(x)* discontinuously varying along the height of the console;

$M_G(x)$ and $Q_G(x)$ are respectively, the bending moment and shear force from the self-weight of the console, linearly varying in height;

$M_s(x)$ and $Q_s(x)$ are respectively, the bending moment and the shearing force from the reactive resistance of the backfill soil to the movement of the console.

According to the second line of (4), for the intensity of the continuous distributed load, we have:

$$P(x)=P_q(x)+P_G(x)+P_s(x) \tag{5}$$

Here $P_q(x), P_G(x), P_{гр}(x)$ respectively, the values of the intensity of the distributed load from the resulting hydrostatic pressure of the water; the self-weight of the wall and the reactive resistance of the backfill soil.

According to (Fig. 1), to describe the discontinuous law of variation of $P_q(x)$ along the height of the console, we will use one-sided extended breakers by N.M. Gersevanov [22].

$$P_q(x)=\frac{P_{hc}}{h_c}x-\varGamma_{h1}\frac{P_{hc}}{h_c}x+\varGamma_{h1}\frac{P_{hc}}{h_c}h_1=\gamma_w x-\gamma_w\varGamma_{h1}(x-h_1) \tag{6}$$

where γ_w is volumetric weight of water.

According to the properties of one-way long breakers:

$$\begin{aligned} &\text{at } x<h_1; \quad \Gamma_{h_1}=0; \quad P_q(x)=\gamma_w x; \\ &\text{at } x=h_1; \quad \Gamma_{h_1}=\frac{1}{2}; \quad P_q(x)=\gamma_w h_1=const; \\ &\text{at } x>h_1; \quad \Gamma_{h_1}=1; \quad P_q(x)=\gamma_w x=const. \end{aligned} \tag{7}$$

The intensity of the lateral load from the longitudinal force *G(x)* linearly varying along the height of the wall [23-25].

$$P_G(x)=\left[G(x)Y(x)\right]''=\left[\frac{G_c}{h_c}xY(x)\right]''\frac{G_c}{h_c}\left[2Y'(x)+xY''(x)\right], \tag{7}$$

where $G_c=\delta_c h_c\gamma_\delta$ is wall weight, width h_c =1.0m.

Taking into account expressions (3), (5), (6) and (7), the differential equation of the cantilever bending can be represented as:

$$Y^{IV}(x)=a_0x+2a_cY'(x)+a_cxY''(x)-\beta xY(x)-\varGamma_{h1}a_0\,(x-h_1) \tag{8}$$

Where,

$$a_0=\frac{\gamma_w}{EJ},\left[M^{-3}\right]; \quad a_c=\frac{G_c}{h_cEJ},\left[M^{-3}\right]; \quad \beta=\frac{K_{hc}}{h_cEJ},\left[M^{-5}\right]; \tag{9}$$

Equation (8) is considered under the following boundary conditions:

$$\begin{cases} Y(0)=Y_0;\ Y'(0)=\theta;\ Y''(0)=\dfrac{M_0}{EJ}=\bar{M}_0; Y'''(0)=\dfrac{Q_0}{EJ}=\bar{Q}_0 \\ Y(h_c)=0;\ Y'(h_c)=0;\ Y''(h_c)=\dfrac{M_{hc}}{EJ}=\bar{M}_{hc}; Y'''(h_c)=\dfrac{Q_{hc}}{EJ}=\bar{Q}_{hc} \end{cases} \quad (10)$$

Since the first part of contains variable coefficients, its integration is possible only with the use of approximate methods.

A very effective approximate method for integrating ordinary differential equations of contact problems was proposed in the works [26-27].

This method is based on the construction of successive approximations according to Picard. However, the so-called boundary function obtained from the fourfold integration of the bending equation under given zero boundary conditions is taken as the zero approximation of the desired function, at *X = 0.*

After performing a fourfold integration of Eq. (8) and determining arbitrary constants of integration from (10), we reduce the problem under consideration to the following differential equation:

$$\begin{aligned} Y_n(x) &= Y_0+\theta_0(x)+\bar{M}_0\frac{x^2}{2!}+\bar{Q}_0\frac{x^3}{3!}+a_0\frac{x^5}{5!}-\Gamma_{h1}\frac{a_0(x-h_1)^5}{5!}+ \\ &+2a_c\iiiint Y_i'(x)\,dx^4+a_c\iiiint xY_i''(x)\,dx^4-\beta\iiiint xY_i(x)\,dx^4 \end{aligned} \quad (11)$$

The boundary function according to (11) has the form:

$$\bar{Y}(x)=Y_0+\theta_0 x+\bar{M}_0\frac{x^2}{2!}+\bar{Q}_0\frac{x^3}{3!}+a_0\frac{x^5}{5!}-\Gamma_{h1}\frac{a_0(x-h_1)^5}{5!} \quad (12)$$

As can be seen from (12), the boundary function $\bar{Y}(x)$ contains terms with four initial parameters $(Y_0,\theta_0,M_0$ и $Q_0)$ and a weight term with coefficient a_0, which fully characterizes the boundary conditions of the problems.

Taking into account (12), equation (11) takes the form:

$$Y_n(x)=\bar{Y}(x)+2a_c\iiiint Y_n'(x)\,dx^4+a_c\iiiint xY_n''(x)\,dx^4-\beta\iiiint xY_n(x)\,dx^4 \quad (13)$$

According to (13), successive approximations are constructed as follows:

$$\begin{cases} Y_0(x)=\bar{Y}(x); \\ Y_1(x)=\bar{Y}(x)+2a_c\iiiint Y_0'(x)\,dx^4+a_c\iiiint xY_0''(x)\,dx^4-\beta\iiiint xY_0(x)\,dx^4 \\ Y_2(x)=\bar{Y}(x)+2a_c\iiiint Y_1'(x)\,dx^4+a_c\iiiint xY_1''(x)\,dx^4-\beta\iiiint xY_1(x)\,dx^4 \\ \dots\dots\dots \\ Y_n(x)=\bar{Y}(x)+2a_c\iiiint Y_{n-1}'(x)\,dx^4+a_c\iiiint xY_{n-1}''(x)\,dx^4-\beta\iiiint xY_{n-1}(x)\,dx^4 \end{cases} \quad (14)$$

It is not difficult to see that with an unlimited increase, the number of members of this sequence is $lim\left[Y_n(x)-Y_{n-1}(x)\right]\to 0,$ and then the limit function $Y_n(x)$ will represent the general solution of the problem under consideration.

Having constructed the limiting function $Y_n(x)$ and grouping the functions by the initial parameters $(Y_0,\theta_0,M_0$ and $Q_0)$ and the load coefficient a_0, we represent the general solution of equation in the form:

$$Y_n(x)=Y_0F_1(x)+\theta_0F_2(x)+\bar{M}_0F_3(x)+\bar{Q}_0F_4(x)+a_0F_5(x) \quad (15)$$

Where $(Y_0,\theta_0,M_0$ and $Q_0)$ is respectively, the values of deflection, angle of rotation, bending moment and shearing force in the initial section of the wall (at X = 0);

the functions $F_1(x),F_2(x),F_3(x)$, $F_4(x)$ and $F_4(x)$ are linearly independent particular solutions of the corresponding homogeneous equation, i.e. equations (8) in the absence of a weight term; $F_5(x)$ is a load function, which is a particular solution of the general in homogeneous equation (8).

The main functions of the solution (8) are determined by the following expressions:

$$F_1(x)=1+\sum_{n=1}^{\infty}(-1)^n\beta^n\frac{x^{5n}}{(5n)!}\left[1\cdot 6\cdot 11\cdot\ldots\cdot(5n-4)\right]+a_c\sum_{n=1}^{\infty}(-1)^n\beta^n\frac{x^{5n+3}}{(5n+3)!}t_{1.n}\times$$

$$\times(6;20;2736;\ldots)+a_c^2\sum_{n=1}^{\infty}(-1)^n\beta^n\frac{x^{5n+3}}{(5n+3)!}t_{2.n}(54;2328;\ldots)+\ldots \quad (16)$$

$$+a_c\sum_{n=1}^{\infty}(-1)^n\beta^n\frac{x^{5n+3}}{(5n+3)!}t_{1.n}(6;20;2736;\ldots)+a_c^2\sum_{n=1}^{\infty}(-1)^n\beta^n\frac{x^{5n+3}}{(5n+3)!}t_{2.n}(54;2328;\ldots)$$

$$F_2(x)=x+\sum_{n=1}^{\infty}(-1)^n\beta^n\frac{x^{5n+1}}{(5n+1)!}\left[2\cdot 7\cdot 12\cdot\ldots\cdot(5n-3)\right]+$$

$$\sum_{n=1}^{\infty}a_c^n\frac{x^{3n+1}}{(3n+1)!}(2\cdot 5\cdot 8;\ldots(3n-1)+2a_c\sum_{n=1}^{\infty}(-1)^n\beta^n\frac{x^{5n+4}}{(5n+4)!}(12\cdot 17\cdot 22\cdot \quad (17)$$

$$\cdot\ldots(5n+7)+\ldots+a_c^2\sum_{n=1}^{\infty}(-1)^n\beta^n\frac{x^{5n+7}}{(5n+7)!}t_{3.n}(320;10280;\ldots)+\ldots$$

$$F_3(x)=\frac{x^2}{2!}+\sum_{n=1}^{\infty}(-1)^n\beta^n\frac{x^{5n+2}}{(5n+2)!}\left[3\cdot 8\cdot 13\cdot\ldots\cdot(5n-2)\right]+$$

$$+\sum_{n=1}^{\infty}a_c^n\frac{x^{3n+2}}{(3n+2)!}(3\cdot 6\cdot 9;\ldots 3n)+2a_c\sum_{n=1}^{\infty}(-1)^n\beta^n\frac{x^{5n+5}}{(5n+5)!}t_{4.n}\cdot \quad (18)$$

$$\cdot(28\cdot 389\cdot 9032\cdot\ldots)+\ldots a_c^2\sum_{n=1}^{\infty}(-1)^n\beta^n\frac{x^{5n+8}}{(5n+8)!}t_{5.n}(624;20134;\ldots)+\ldots$$

$$F_4(x)=\frac{x^3}{3!}+\sum_{n=1}^{\infty}(-1)^n\beta^n\frac{x^{5n+3}}{(5n+3)!}\left[4\cdot 9\cdot 14\cdot\ldots\cdot(5n-1)\right]+\sum_{n=1}^{\infty}a_c^n\frac{x^{3n+3}}{(3n+3)!}\times$$

$$\times(4\cdot 7\cdot 10;\ldots(3n+1)+a_c\sum_{n=1}^{\infty}(-1)^n\beta^n\frac{x^{5n+6}}{(5n+6)!}t_{6.n}(64;1272;31190\ldots)+\ldots \quad (19)$$

$$+a_c^2\sum_{n=1}^{\infty}(-1)^n\beta^n\frac{x^{5n+9}}{(5n+9)!}t_{7.n}(1048;37344;\ldots)+\ldots$$

$$F_5(x)=\frac{x^5}{3!}+\sum_{n=1}^{\infty}(-1)^n\beta^n\frac{x^{5n+5}}{(5n+5)!}\left[6\cdot 11\cdot 16\cdot\ldots\cdot(5n+1)\right]+$$
$$+\sum_{n=1}^{\infty}a_c^n\frac{x^{3n+5}}{(3n+5)!}(6\cdot 9\cdot 12;\ldots.(3n+3)+$$
$$+a_c\sum_{n=1}^{\infty}(-1)^n\beta^n\frac{x^{5n+8}}{(5n+8)!}t_{8.n}(120;2736;74160\ldots)+\ldots$$
$$+a_c^2\sum_{n=1}^{\infty}(-1)^n\beta^n\frac{x^{5n+11}}{(5n+11)!}t_{9.n}(1789;87852;\ldots.)-\ldots \tag{20}$$
$$-\Gamma_{h1}\left\{\frac{(x-h_1)^5}{5!}+\sum_{n=1}^{\infty}(-1)^n\beta^n\frac{x^{5n+5}}{(5n+5)!}\left[6\cdot 11\cdot 16\cdot\ldots\cdot(5n+1)\right]+\right.$$
$$+\sum_{n=1}^{\infty}a_c^n\frac{(x-h_{1)}^{3n+5}}{(3n+5)!}(6\cdot 9\cdot 12;\ldots.(3n+3)+a_c\sum_{n=1}^{\infty}(-1)^n\beta^n\frac{(x-h_1)^{5n+8}}{(5n+8)!}t_{8.n}+$$
$$+a_c^2\sum_{n=1}^{\infty}(-1)^n\beta^n\frac{x^{5n+11}}{(5n+11)!}t_{9.n}(1789;87852;)+\ldots$$

As can be seen from (16) - (20), each of the main functions of the solution (15) is represented as a sum of infinite rapidly converging series. When performing calculations with sufficient practical accuracy, one can restrict oneself to only the first two or three rows of each function, while limiting oneself only to the first two or three terms of the expansion of infinite series. The fast convergence of the series is ensured both by the presence of increasing factorial values in their denominators and by very small values of the parameters a_0, a_c, β.

According to the general solution, to determine the values of the angle of rotation, bending moments and shearing forces in arbitrary sections of the wall (console), we have the following calculation formulas:

$$\begin{cases}\theta(x)=Y_0F_1'(x)+\theta_0F_2'(x)+\bar{M}_0F_3'(x)+\bar{Q}_0F_4'(x)+a_0F_5'(x)\\ \bar{M}(x)=Y_0F_1''(x)+\theta_0F_2''(x)+\bar{M}_0F_3''(x)+\bar{Q}_0F_4''(x)+a_0F_5''(x)\\ \bar{Q}(x)=Y_0F_1'''(x)+\theta_0F_2'''(x)+\bar{M}_0F_3'''(x)+\bar{Q}_0F_4'''(x)+a_0F_5'''(x)\end{cases} \tag{21}$$

In the formulas (15) and (21), the unknown parameters are the displacement of the wall at the level of the filling surface Y_0 of the angle of rotation of the initial section θ_0. Based on the condition of fixing the lower end of the console to determine them, we have

$$\begin{cases}Y_0=Y_0F_1(h_c)+\theta_0F_2(h_c)+\bar{M}_0F_3(h_c)+\bar{Q}_0F_4(h_c)+a_0F_5(h_c)\\ \theta_0=Y_0F_1'(h_c)+\theta_0F_2'(h_c)+\bar{M}_0F_3'(h_c)+\bar{Q}_0F_4'(h_c)+a_0F_5'(h_c)\end{cases} \tag{22}$$

Solving this system with respect to the unknown initial parameters Y_0 and θ_0 we find:

$$\begin{cases}Y_0=\dfrac{AF_2'(h_c)-BF_2(h_c)}{F_1'(h_c)F_2(h_c)-F_1(h_c)F_2'(h_c)}\\ \theta_0=\dfrac{AF_1'(h_c)-BF_1(h_c)}{F_1'(h_c)F_2(h_c)-F_1(h_c)F_2'(h_c)}\end{cases} \tag{23}$$

Where

$$A=\bar{M}_0F_3(h_c)+\bar{Q}_0F_4(h_c)+a_0F_5(h_c);\;\; B=\bar{M}_0F'(h_c)+\bar{Q}_0F_4'(h_c)+a_0F_5'(h_c). \tag{24}$$

4 Conclusions

The main purpose of the research in this article is to develop a methodology for the deformation calculation of flexible structures in contact with the soil environment. The stress-strain state of flexible walls, taking into account the self-weight of the structure, was determined by the Fuss-Winkler method for soils with a variable stiffness coefficient.

The deformation calculation of a flexible soil wall, taking into account the influence of its self-weight, is determined using the function of instant breakers proposed by prof. N.I.Gersevanov. As a result, the basic formulas for the angle of rotation and displacement of the wall relative to the axis of the structure were obtained.

5 Acknowledgements

The result obtained allows for each case of changing the stiffness coefficient of the foundation soil and the loading scheme to obtain the equation for the curved axis of the beam. This allows defining bending and displacement, as well as internal forces in flexible structures used in hydraulic engineering. The use of this calculation method allows the rational use of building materials and ensures the reliability and durability of structures.

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