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Design of underground structure closed in plan erected with application of the Diaphragm Wall Technique

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ABSTRACT: Method of designing underground structures being erected with the application of the "Diaphragm Wall Technique" based upon mathematical modelling and the analytical solution of the corresponding elasticity theory plane contact problems is given.

1 PRINCIPAL POSITIONS OF THE DESIGN METHOD

The method of designing diaphragm walls being erected by the application of diaphragm wall technology is based upon the modern representation concerning interaction of the structure with the surrounding soil mass. The structural wall and the soil mass are considered as a common deformable system undergoing external loading. The ground mass is simulated by the linearly deformable medium, the contact stresses upon the wall from the soil (the pressure) are not given, but are determinated in the process of computation. the core.

The mathematical models and methods for structural wall computation are based upon the analytical solution of the corresponding elasticity theory contact problems. The methods are applicable for the diaphragm wall of an arbitrary form closed in plan as well as for several walls of circular cross-section in plan close to each other.

Computation methods take into account the technology of underground openings construction with the application of the diaphragm wall technique namely:
- forming and concreting under clay mud a trench closed in plane;
- extracting ground core and unloading internal surface of a structural wall.

If several shafts close each other are constructed the sequence of structural walls erection and mutual influence of structures are taken into account.

The mathematical models and design methods for diaphragm walls are based on the following principal positions:
- structural walls and surrounding soil mass are considered to be elements of a common deformable system (wall - soil mass system),
- soil mass is simulated by a linearly deformable medium because the displacements of the soil near the diaphragm wall are small,
- diaphragm wall loading is caused by excavating a soil core out of the inside.

The latter position is realized in the design method through considering the full stress state of the structural wall as a sum of initial one and that produced by removing the stresses from the internal surface of the wall because of extracting the core.

2 DESIGN OF CIRCULAR CROSS-SECTION STRUCTURES OF SHAFTS

Taking into account the structure erection technology one can distinguish two stages of erection for elaborating the design scheme:
1) forming and concreting under mud the trench closed in plan,
2) extracting soil core and unloading internal surface of a diaphragm wall.

The initial stresses upon the external and internal wall surfaces undergo formation at the first stage, those stresses being formed due to the hydrostatic pressure of the mud and equal to $\gamma_m H$, where γ_m is the mud density, H is the mud column height.

At the second stage the structure is put into operation because of the internal pressure upon the wall being removed. Thus, the resulting stresses in the diaphragm wall and upon the contact of the wall with the soil mass can be regarded as the sum of the initial stresses, equal to $\gamma_m H$ and complementing ones which are being formed in the "wall-soil mass" system, if the removed normal stresses equal to the initial ones in absolute value and opposite in sign ($-\gamma_m H$) is applied to the internal wall surface.

The design method of the circular cross-section of monolithic or assembly monolithic diaphragm wall is carried out by the general design method of a circular cross-section multi-layer lining (Bulychev,1989, 1994). In this case one can take into account the presence of different layers such as stiffener ribs, elastic or rigid reinforcement in structure and receive a complete picture of stress distribution in structure elements.

Design scheme of a multi-layer diaphragm wall for removed stress state is shown in Figure 1.

Soil pressure on a diaphragm wall is the wall with the soil mass can be regarded as a sum of the initial stresses, calculated by the formula

$$p = \gamma_m H \left(1 - \prod_{i=1}^{n-1} K^*_{0(i)}\right) \qquad (1)$$

where K is the transfer coefficient of the uniform internal radial stresses through the i-th layer. That coefficient is calculated by the formula

$$K^*_{0(i)} = \frac{d'_{2(i)}}{d'_{1(i)} + \beta_0 (d_{2(i+1)} - d_{1(i+1)} K^*_{0(i+1)})} \qquad (2)$$

where G_i ,G_0 are the wall material and the soil mass shear moduli respectively;
$r_{0(i)}$,$r_{1(i)}$ are the internal and external radii of the i-th layer of the wall cross-section.
$d'_{1(i)}$, $d'_{2(i)}$ are coefficients, chracterising i-th layer of the wall (Bulychev, 1989),

$$\beta_0 = G_i\ (c_i^2 - 1)/G_{i-1}(c_{i-1}^2 - 1), \quad c_i = r_{1(i)}/r_{0(i)}.$$

Normal tangential stresses on the internal and external outline of the i-th layer of a diaphragm wall cross-section are being determined after the contact stresses are calculated.

Let us examine for example of calculating of the monolithic diaphragm wall such as has been erected at the hydroelectric plant in Pensano (Italy). The work was done by TREVI. The four shafts wich house the turbines were executed with reinforced concrete diaphragm walls. The shafts actually consist of 2 cilyndrical coaxial shafts (Figure 2).

We do not know the real chracteristics of soil and material of the wall and attach for instance the following input data:
- for the upper shaft: G_1 / G_0 = 800, r_1 = 14.56 m, r_0 = 13.76 m, H = 35 m, = 0.013 MN/m;
- for the lower shaft: r = 12.76 m,

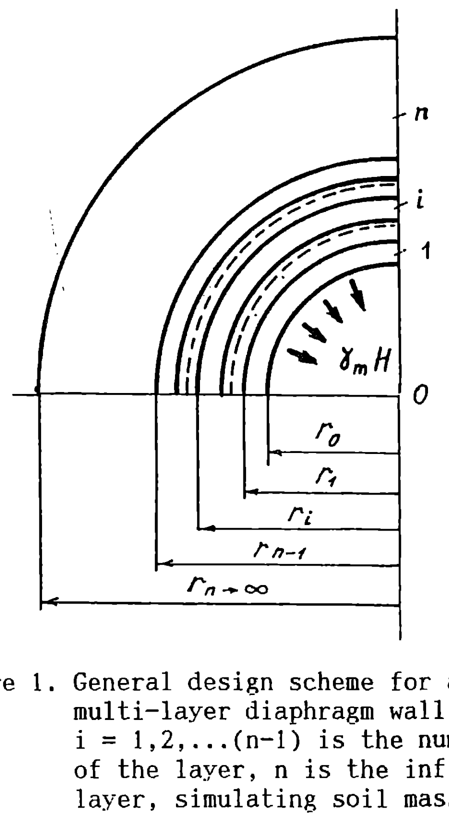

Figure 1. General design scheme for a multi-layer diaphragm wall: i = 1,2,...(n-1) is the number of the layer, n is the infinity layer, simulating soil mass.

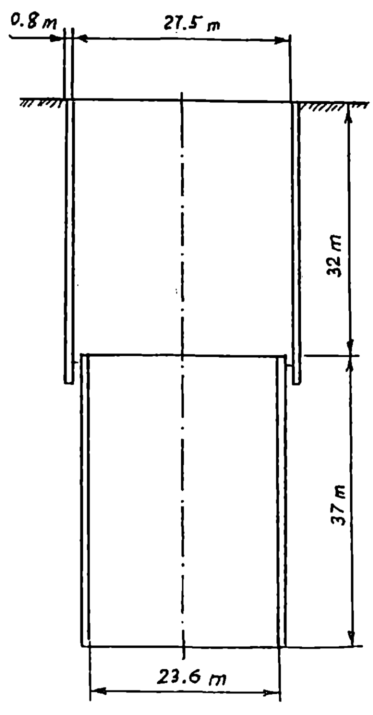

Figure 2. Scheme of the shaft.

r_0 = 11.08 m, H = 31 m.

Design scheme for the shaft diaphragm wall is shown at the Figure 3. The formula (2) aquires the following view

$$K^*_{0(1)} = \frac{\varkappa_0 + 1}{d'_{1(1)} + 2\, G_1 (c_1^2 - 1)/G_0} \qquad (3)$$

Where $\varkappa_0 = 3 - 4\,\nu_0$

ν_0 is the Poisson Ratio of soil mass.

The results of the calculation are the following

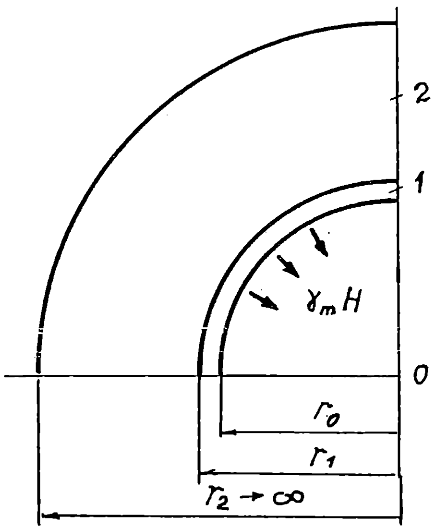

Figure 3. Design scheme of the shaft diaphragm wall.

Stresses	Shafts	
	Upper	Lower
p_0, MPa	0.41	0.46
σ_θ^{in}, MPa	7.6	7.5

3 DESIGN OF DIAPHRAGM WALL OF AN ARBITRARY FORM IN PLANE

Method of design is based on the consideration of the design scheme including the ring of an arbitrary shape simulating the structure supporting the opening in a linearly deformable medium simulating the soil mass. Taking into account the structure erection technology we assume, that the initial stresses in the wall-soil system are equal to the γ_mH solution's pressure where H is the mud column height. The extracting soil core results in unloading internal surface of a structural wall, i.e. in appearing additional normal stresses $\sigma_\rho = -\gamma_m H$ on the internal outline of the ring cross-section simulating the diaphragm wall. Therefore for the calculation of that structure the analytic solution of the corresponding elastisity theory plane contact problem (Fotieva, 1974) may be applied.

The results of the computer calculation for the rectangular and oval forms of diaphragm wall (see Figure 4) are shown in Figures 5 and 6. The following input data are accepted in the calculations: the concrete deformation modulus

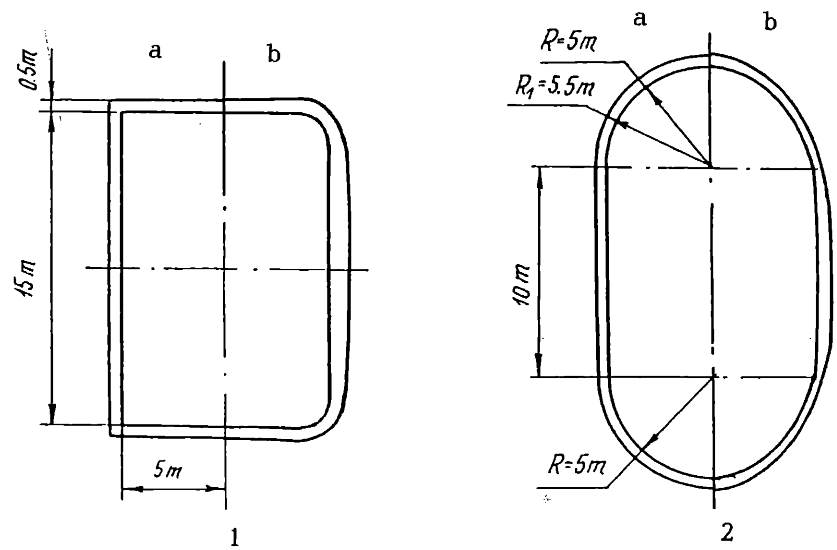

Figure 4. Rectangular (1) and oval (2) shapes in plane diaphragm walls for designing: 1 - project forms, 2 - shapes after conformal mapping.

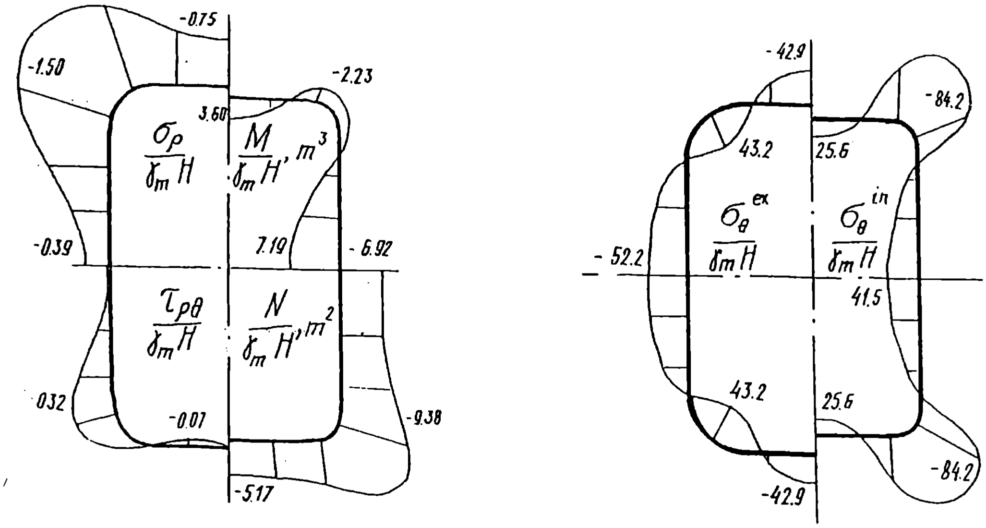

Figure 5. Sresses and internal forces in diaphragm wall of rectangular shape in plane: σ_ρ/γ_m H is the dimensionless normal contact stresses, $\tau_{\rho\theta}/\gamma_m$ H is the same contact shear stresses on the contact between wall and soil;
M, N are the bending moment and internal forces;
$\sigma_\theta^{in}/\gamma_m$ H and $\sigma_\theta^{ex}/\gamma_m$ H are dimentionless normal tangential stresses on the internal and external outlines correspondingly.

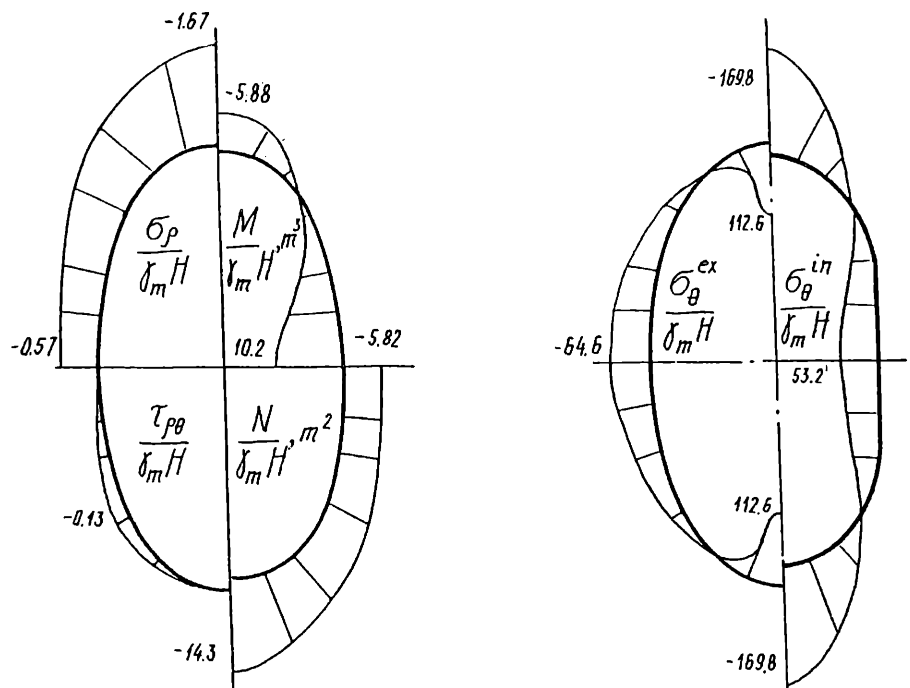

Figure 6. Sresses and internal forces in diaphragm wall of oval shape in plane: σ_ρ/γ_m H is the dimensionless normal contact stresses, $\tau_{\rho\theta}/\gamma_m$H is the same contact shear stresses on the contact between wall and soil;
M, N are the bending moment and internal forces;
$\sigma_\theta^{in}/\gamma_m$ H and $\sigma_\theta^{ex}/\gamma_m$H are dimentionless normal tangential stresses on the internal and external outlines correspondingly.

E = 25000 MPa, soil massif deformation modulus E_0 = 700 MPa, the Poisson Ratio of concrete ν_1 = 0.2, of the soil ν_0 = 0.38.

The $\sigma_\rho / \gamma_m H$ normal and the $\tau_{\rho\theta} / \gamma_m H$ shear contact stresses, the M/ H bending moments and the $N/\gamma_m H$ longitudinal forces are given in Figures 5,a and 6,a; the $\sigma_\rho / \gamma_m H$ normal tangential stresses on the external and internal outlines of the structure cross-sections are given in Figures 5,b and 6,b.

CONCLUSION

Method of designing underground structures erected with the application of the diaphragm wall technique, being the methods, of underground structure mechanics take into account the interaction of the structure with the soil mass and make an a priori assignment of loads (stresses upon the contact of the structure with the soil mass) unnecessary.

The calculations are being made in the process of a general design of the whole system of "mass soil-structure", taking into consideration in full extent the loading capability of the soil mass itself. It allows in certain cases the more economic design solutions to be obtained.

Let us mark that no difficulty is observed when the reological characteristics of the soil mass are taken into account with can be made on the base of the linear hereditary creep theory using the method of variable moduly.

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