Foundation pit protection by a diaphragm at NPS Krško (Yu)
Protection d'une excavation de fondation par une paroi moulée a NPS Krško (Yu)

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SYNOPSIS: A report on the conception of protecting the deep foundation pit at a delevelling of ca 17 m, giving an account of the basic design, engineering and technological features of the protective diaphragm, together with a review of geostatical analyses, using, among others, original solutions by the authors, which once more were not countermanded by the practice.

1 TASK OF THE DIAPHRAGM

The first nuclear power station/plant (NPS) in Yugoslavia has been built at Krško, a settlement on the left bank of river Sava, 50 km upstream of Zagreb (NPS Krško).

For the foundation purposes of the structure it had been necessary to excavate a foundation pit with a maximum depth of appr. 20 m (in the reactor's zone) and achieve a protection of the pit against the ingress of groundwaters (Fig. 1), because the delevelling of water had been appr. 17 m.

Complex geophysical, geological and seismological investigations had been performed at the said microlocation. The design solution of the diaphragm was worked out within a conception of a water-tight protective counter-seepage screen, which leaves the role of the lateral bearing capacity to a passive "wedge" of unexcavated soil in the foundation pit (Fig. 2).

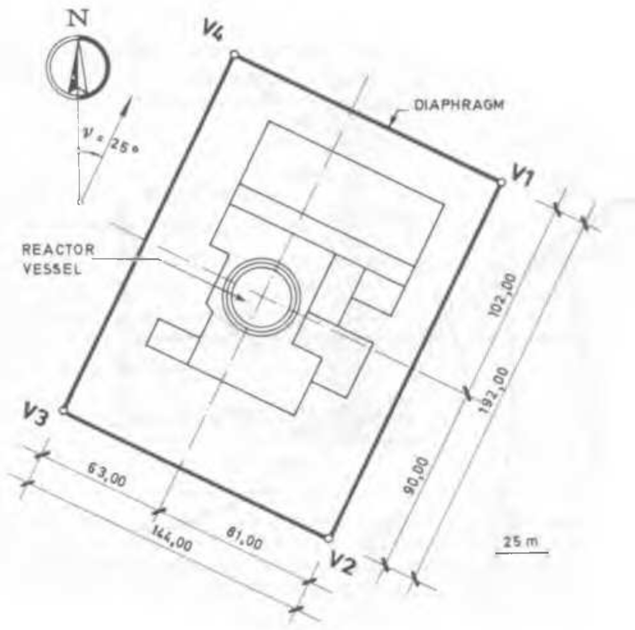

Figure 1. Layout of structure & diaphragm.

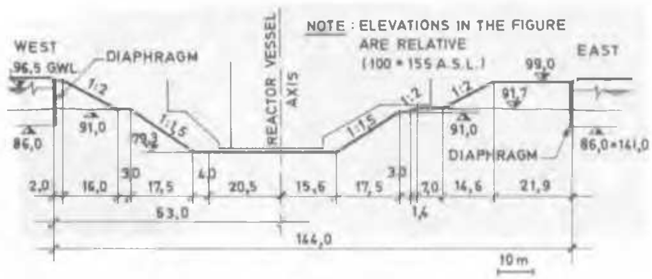

Figure 2. Section through the foundation pit, direction west-east.

In the structural engineering sense, the diaphragm thus becomes a core planary compressed on both sides where appear bending moments with values depending on developed relative displacements, as well as the stiffness of the diaphragm. Were such a diaphragm built by the classical un-reinforced concrete, its stiffness would be relatively great, resulting in greater bending moments and, consequently, in a potential possibility of cracking of the diaphragm, followed by the penetration of water into the foundation pit. In order to decrease the diaphragm's stiffness it was sought to choose such a material for the fill which will have a tendency to be a plastic diaphragm, being therefore deformed without any significant stresses. Besides, the diaphragm has had to have a determined strength to transmit the lateral pressure, while at the same time the particular requirements of constructional character, as well as the economy, had to be met. An optimum solution has been achieved choosing the so called clay-concrete diaphragm fill, being made of a mix of cement, bentonite (clay mineral) gravel and water. As the water-bearing alluvial stratum at the NPS site has been 8-10 m thick (Fig. 2), with poorly pervious strata of silty material underneath, the diaphragm has been built down to a depth of 14 m. Namely, analyses had shown that the effects of seepage deeper down, through sides, as well as through the bottom of the foundation, will not imperil the hydraulic stability of the pit, so that such a decision has had its engineering justification, accompanied with considerable savings.

So, a clay-concrete counter-seepage screen, having a role of a floating diaphragm, has been built around the pit, changing thus in a specific way the water flow in the soil around the foundation pit. In the zone of contact between the water-bearing alluvium (gravel) and poorly pervious silty series a working berm has been executed, where pumps to pump the seepage water from the bottom of the foundation pit had been installed (Fig. 3).

Figure 3. South-western corner of the foundation pit.

2 BASIC COMPUTATIONAL ANALYSES

The equipotential disposition (Fig. 4) has been obtained for the steady flow on the plane x, y, which had been supposed in the subject problem. This problem has been solved by the FEDAR [1] programme with given boundary conditions for the subject area. As it is known, this programme serves to seek the potential energy minimum within the flow area, depending on the potential function h and using the finite element method. After the construction of the flow net, began analyses of the hydraulical stability and statical analyses of the foundation pit. These analyses gave satisfactory results, but with relatively low safety factors, which could have been neverthaless adopted, considering the temporary character of the excavated pit. After that, the geostatical calculation of the diaphragm had been performed. In the structural engineering sense, the diaphragm had been treated as a beam construction with a thickness d and the width of 1 m, being calculated as a beam built in the soil. A geostatical calculation has been made for the critical section of the diaphragm. As active forces it had been taken that the diaphragm is loaded by the hydrostatic pressure and the active pressure of the soil, while these loadings are counteracted by passive resistances of the soil from the side of the foundation pit (Fig. 5). Concerning the diagram of required passive resistances, which must be formed on the inner side of the diaphragm, the starting point were basic conditions of static equilibrium, namely that this diagram has the same surface area as the diagram of total active loadings, and that the centre of gravity of this diagram is at the same level as the center of gravity of the total active loadings diagram. These two conditions are unified in the supposition of a parabolic distribution of passive resistances [2]. For the known boundary conditions an equation of the passive resistances parabola can be obtained in form of: $p_r = ay^2+by$. The mentioned work [2] also includes the solution for the case of the so called plastic stress in the soil, whereby the parabola degenerates into a boundary straight line of the possible passive resistance. The integration of the basic equation $p_r = f(y)$ also gives the equation of required passive forces

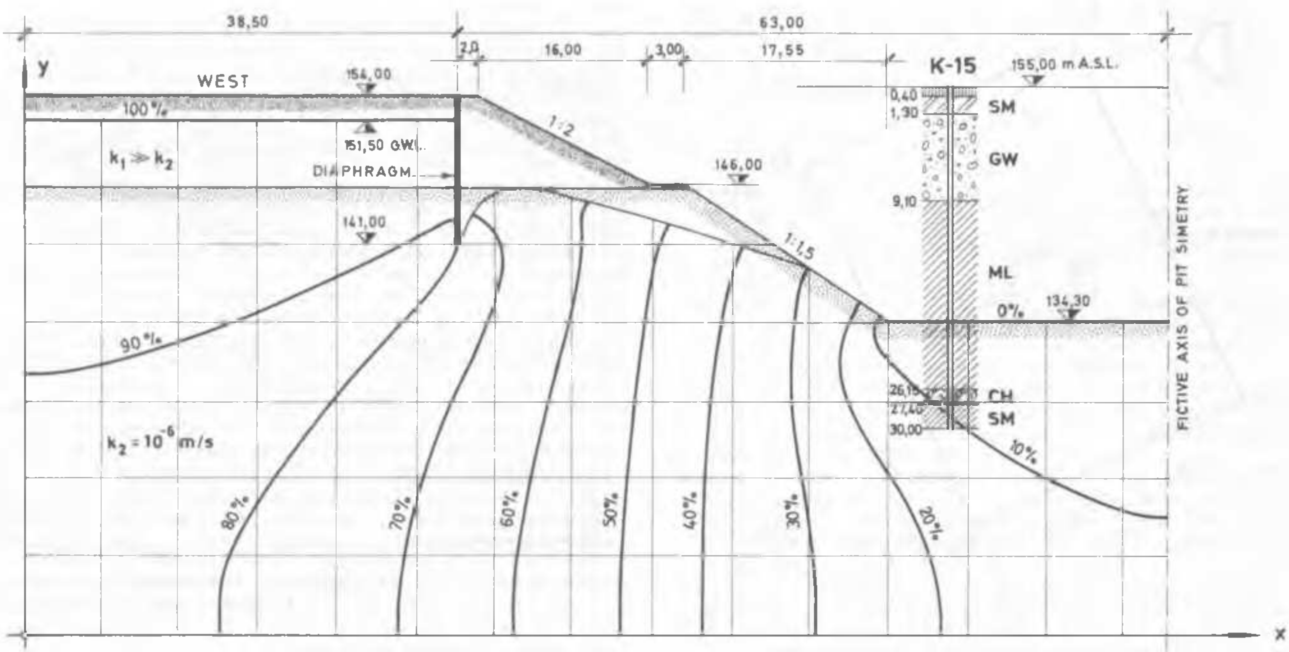

Figure 4. The equipotential disposition of the steady flow on the x, y plane

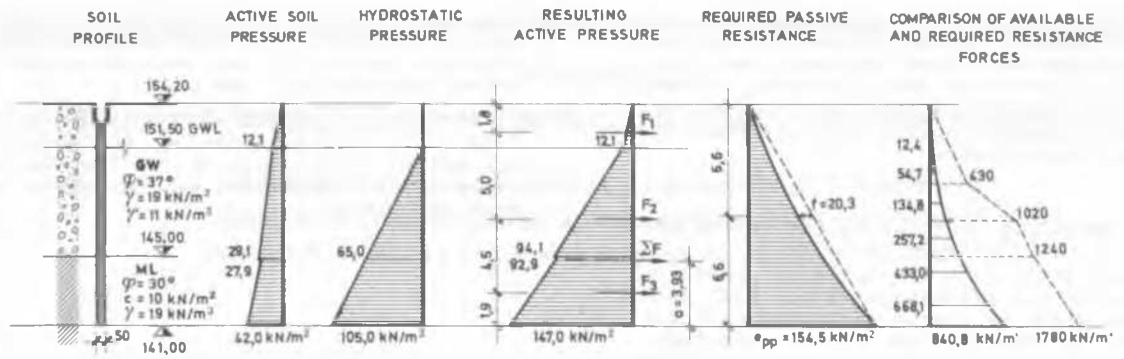

Figure 5. Diagrams of loading upon the diaphragm, and the comparison of passive resistance forces.

P_r, which must be computationally created at particular levels through the passive volume of the soil. This problem is solved numerically, using the original computer programme SPORT [3]. But, apart from the required passive resistances, it was also necessary to determine the minimum boundary resistances P_m of the soil for individual levels. A number of potential sliding surfaces, having a circular or arbitrary shape had been presupposed. The problem has been solved in the accordance with the general numerical analysis by Cesarec [4] for circular and arbitrary sliding surfaces in a stratified soil, considering as well the pore pressure and using the original computer programme POT. The conception of the method is given in Fig. 6 and the general formula, under a hypothesis that inter-slice forces are perpendicular on boundaries of slices, ($T_1 = \Delta T_i = 0$) can be written as follows:

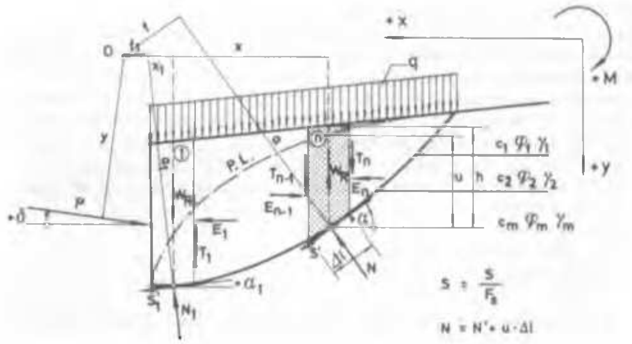

Figure 6. General case of passive resistance, according to Cesarec (1981).

$$P = \left[\Sigma W_R \cdot x + \frac{1}{F_s}\Sigma \frac{c \cdot \Delta x + (W_R - u \cdot \Delta x)\,tg\varphi'}{\bar{m}_\alpha} \cdot a - \Sigma \frac{W_R + \frac{1}{F_s}(c' - u \cdot tg\varphi')\,tg\alpha \cdot \Delta x}{\bar{m}_\alpha} \cdot f\right] \cdot \left(y + \frac{\sin\delta}{\bar{m}_{\alpha 1}} \cdot f_1 - \frac{tg\varphi'}{F_s} \cdot \frac{\sin\delta}{\bar{m}_{\alpha 1}} \cdot a_1\right)^{-1} \quad \ldots\ldots(1)$$

For the circular sliding surface the moment legs are $a = R$, $x = R \cdot \sin\alpha$ and $f = 0$, so that the formula is:

$$P = \left(\Sigma W_R \cdot \sin\alpha + \frac{1}{F_s}\Sigma \frac{c' \Delta x + (W_R - u \cdot \Delta x)\,tg\varphi'}{\bar{m}_\alpha}\right) \cdot \left(\frac{y}{R} - \frac{tg\varphi'}{F_s} \cdot \frac{\sin\delta}{\bar{m}_{\alpha 1}}\right)^{-1} \quad \ldots\ldots(2)$$

In the obave expressions;

$\bar{m}_\alpha = \cos\alpha - (tg\varphi'/F_s) \cdot \sin\alpha$, $W_R = W + q \cdot \Delta x$, F_s is the safety factor and other symbols are as in Fig. 6. For the Rankine's case of passive resistance the general expression (1) (in which case $c = 0$, $T_i = 0$, $F_s = 1$ and $\delta = 0$) is reduced to the following expression:

$$P = \frac{\Sigma W \cdot x + \Sigma \frac{W \cdot tg\varphi'}{\bar{m}_\alpha} \cdot a - \Sigma \frac{W}{\bar{m}_\alpha} \cdot f}{y} \quad \ldots\ldots(3)$$

The expression (3), as demonstrated in the work [5], can be directly reduced to Rankine, so that the Rankine's problem represents merely a special case of the outlined general equation (1).

By the quotient of forces of minimal possible passive resistances, P_m, and forces of the required passive resistances, P_r, at individual levels, safety factors against the passive failure are obtained for respective levels, $F_s = \frac{P_m}{P_r}$. The minimal factor for the critical level has been $F_{s\,min} = 2{,}12$, which proved to be

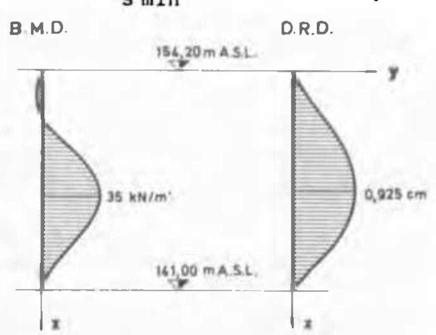

Figure 7. Diagrams of bending moments (B.M.D.) and relative displacements (D.R.D.).

satisfactory. In the further procedure, using the SRESS programme, values of inner forces within the diaphragm have been obtained, the diaphragm being loaded with the difference between the active and passive loading. Figure 7 gives the diagram of bending moments and the diagram of relative displacements.

3 CHECK-UP INVESTIGATIONS DURING THE OPERATION

In the course of structure's building, check-up measurements of the groundwater level had been performed by means of double piezometers within and without the area enclosed by the diaphragm, so as to ascertain the groundwater regime in the newly established condition. The stability of the diaphragm had also been checked by measuring the inclinometers positioned in boreholes, drilled through the middle part of the diaphragm. After the diaphragm has been finished, a resistivety logging had been performed at some of joints of the diaphragm: a method from the domain of undestructive geophysics. As well, a core has been taken out of the diaphragm's body and submitted to both, visual and laboratory tests in order to establish the quality of the fill. These check-up investigations brought satisfactory and expected results, proven by the successful foundation of the whole structure, which has now already been in operation for same 10 years.

4 CONCLUSION

The foundation pit for NPS Krško had been excavated within an area of 144 x 192 m and enclosed by a protective diaphragm being 672 m long in total. The depth of the foundation pit is biggest at the position of the reactor: appr. 20 m. The clay-concrete diaphragm is built down to the depth of 14 m (floating diaphragm), this being possible due to a favourable distribution of soil strata - water bearing gravels stretching down to the depth of 10 m, while deeper on there are poorly pervious strata of silty materials. Although analyses of the seepage into the foundation pit had been pointing to relatively high hydraulic gradients, the conception of a floating diaphragm has been nevertheless realized, as it had been a matter of a temporary state of steady flow. Just in case, sets of well-points had been kept ready at the building site, to be used in the case of a possible accident. In the end, these well-points came to no use, as all planned operations had been performed without any larger difficulties.

LITERATURE

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