https://doi.org/10.26552/com.C.2007.1.67-70

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THE STABILITY OF HYDRO TECHNICAL EMBANKMENTS UNDER VARIOUS SEEPAGE CONDITIONS

The flood control devices in Bytom Odrzanski protect the city against the Odra water flooding with a probability $p = 10 \div 20$ %. The flood protection devices (class validity II) of hydro-technical structures are designed and realized for protection improvement and designed on the flow in the Odra river with a probability $p = 1$ % . The earth embankment (2044 total in length) will be built with local non-cohesive grounds. On the 900 m length, the base of the embankment presents no cohesive grounds. It causes conditions for free filtration through the base and body of the embankment. The computer calculations of filtration showed high (locally) hydraulic heads ($I_{max} = 0{,}984$) that cause the need for ground protection against flux and erosion by using filtration geotextile. The smallest value of the stability coefficient obtained using the 'Fellenius' method was $F_s = 1.439 > 1.2$. The exact analysis of filtration conditions and stability allowed for the undertaking of solutions ensuring the safety for the structure.

1. Introduction

The safety of hydro technical buildings such as soil banks, flood embankments and earth dams is a very important and essential issue because of two aspects: The safety of the constructions themselves and the prevention of water-logging adjacent areas in cases of high river levels. One of more important elements affecting safety (stability of the earth dam) is filtration of water through permeable grounds used for building the dam. Water percolated in the conditions of the high flow gradient could cause occurrence of such phenomena as suffosion, washing out and/or hydraulic uplift which negatively influences the stability of the construction. It is very important that the filtration analysis is not limited to only determining the hydroisohypses course. It should also include a calculation of the length and direction of the velocity water flow vectors. It will allow the determination of regions where the hydraulic head could exceed admissible and critical values.

The results of water flow to flood embankment protection with negative effects of filtration are presented in this paper using the example of the left-bank of the Odra valley in the Bytom Odrzański region. The flood protection program for this locality predicts a two stage investment task [5, 6]. Stage I presents the improvement of protection of the most at risk of flooding, north – west part of the city. Stage II includes further building-up the left-hand flood embankment of the Odra river: km 416.95 + 419.20. With reference to embankment distance, it is a section of 2+044 km in length from km 0+550 to km 2+594, thus from a newly built surrounding embankment in stage I to the urban sewage treatment plant (Fig. 1). Whilst preparing the engineering project (flood protection of the Bytom Odrzański region) – stage II, there was a need to carry out the investigation of free seepage and stability of the embankment, which is the object of this work.

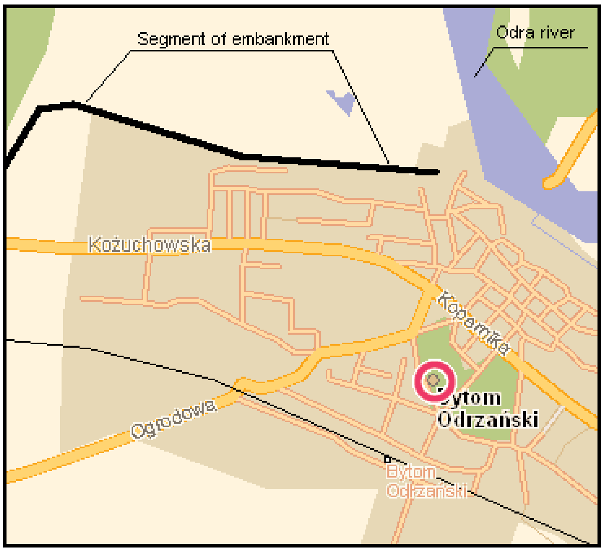

Fig. 1 Situation map

2. Geology and hydrogeology

In the foundation of the projected embankment (section km 1+700 ÷ 2+594) mineral bearing soils were discovered representing Holocene and Pleistocene formations of the river lagoon, glacier and river origin (Fig. 2). Also were discovered sands, medium sands and sandy gravels but seldom dust sands or coarse sands with gravel. These are permeable and very permeable grounds with filtration coefficient from 0.5 m/d in the case of fine sands and silty sands to 55 m/d in the case of sandy gravels. Sporadically among

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the sands there are also the river lagoon's inclusion of cohesive grounds, therefore, clays and silty clays [3].

The groundwater depths are shallow and very shallow. The free groundwater table is at a levels of about 0.5 ÷ 2.0 m below the surface of the terrain (in the period of the mean levels) depending on the region and distance from the Odra bed.

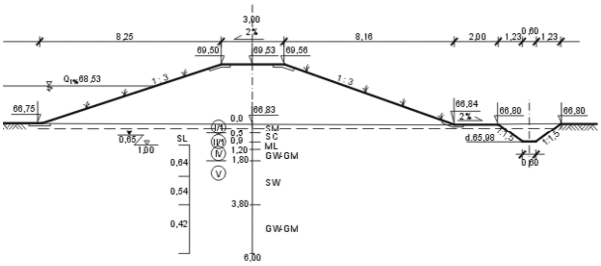

Fig. 2 Vertical cross-section of embankment

3. Characterization of the researched object

The left bank embankment of the Odra river protecting built-up areas of Bytom Odrzański was qualified to a class II validity of structure. The embankment location corresponds to "A general strategy against flood protection in the upper and middle Odra basin". The relative safe height of the dam crest below the reliable water level was taken 1.0 m [6, 7], which met technical standards for water management objectives – class II validity structure. The investigations of the filtration and stability conditions for the section of embankment from stage II of the investment were done on the stage of studying project [1,2]. The following embankment parameters were assumed:

- width of the dam crest 3.0 m
- slope 1:3
- total length 2 044 m
- height 1.8 ÷ 3.8.

The slope and crest of the dam are stabilized by grassing on the humus.

There is a local alluvial soil layer in the foundation of the embankment which causes conditions for the seepage under pressure. However, on the embankment section (length 894 m, in km 1+700 ÷ 2+594) where alluvial soil does not occur, seepage through the dam and subsoil foundation will not appear. [1].

4. Computation for free filtration in typical vertical cross-section of embankment

The vertical flat flow occurs in the case of liquid filtration through the objects where the flow in a perpendicular direction to the cross section could be recognized as inessential (dams, embankments and others barriers from porous materials). The equation which describes the potential of the liquid flow area [4, 8, 10] is presented:

$$\frac{\partial}{\partial x}\left(k_s \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial z}\left(k_s \frac{\partial h}{\partial z}\right) = 0 \tag{1}$$

And in the case of homogenous and isotropic medium, where the conductivity is $k_x = k_z$, the Laplace's equation is given:

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial z^2} = 0 \tag{2}$$

Solving the issue of stationary flow with a free surface there remains the problem of determining the depression curve location where kinematics boundary conditions must be implemented ($p = 0 \rightarrow h = z$ – potential is equal to the position head) and $q_n = 0$ – showing a shortage of flow in the normal direction to the depression curve. It demands starting the iterative process where the differences of depression curve location in two following iteration are taken as a criterion of the accurate assessment of the solution.

The computer program FILTR based on the finite elements method was used for solving the problem. The program generates the net of trianglular elements for which the function approximating the potential in the element is the linear function of two variables. The consequence of this fact is the constant value of the filtration velocity in the element. The results of the computations are presented as the velocity field in the filtration area and hydrodynamic network. The total discharge of flow is also given.

Assumptions and results of computations

The computation scheme presented in Fig. 3 was taken into the research. The computer calculations of filtration were done with the following alternative assumptions:

Variant B_1:
- $H = 2.5$ m,
- thickness of the permeable ground $M = 10$ and filtration coefficient $k = 18$m/d,
- permeable embankment body ($k = 18$m/d) with tight screen on the water side slope,
- surrounding collector trench with groundwater depth at 0.5 m below the surface of the terrain.

The filtration process takes place in the free filtering conditions through the basement soil and embankment.

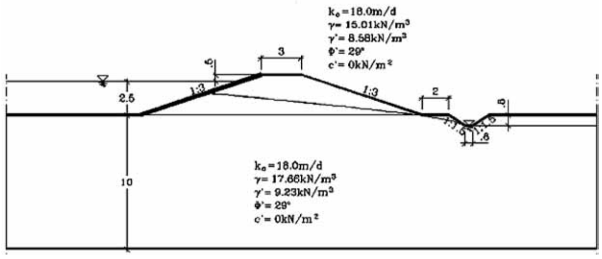

Fig. 3 Computation scheme for vertical cross-section of embankment (variant B_1 and B_2)

Variant B_2:

The assumptions are similar to the variant B1, only the depth of water level in the trench is 0.8 m below the surface of the terrain.

It was stated, basing on the computer calculations of filtration, that:

For the variant B_1:
- value of the inflow rate on the linear metre of trench amounts to $q_r = 1.017*k_o = 1.017*18.0 = 18.31\ m^3/d$,
- maximum velocity of inflow to the surrounding collector trench $V_{max} = 0.581*k_o = 0.581*18.0 = 10.46$ m/d,
- maximum value of the hydraulic head in the zone of the inflow to the trench $I_{max} = 0.581$.

For the variant B_2:
- value of the inflow rate on the linear metre of trench amounts to $q_r = 1.088*k_o = 1.088*18.0 = 19.58\ m^3/d$,
- maximum velocity of inflow to the surrounding collector trench $V_{max} = 0.942*k_o = 0.942*18.0 = 17.71$ m/d,
- maximum value of the hydraulic head in the zone of the inflow to the trench $I_{max} = 0.984$.

The example of computation results are showen on Figs. 4 and 5.

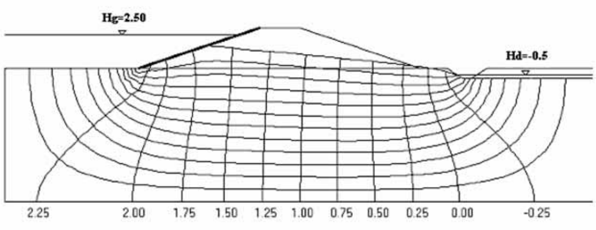

Fig. 4 Result of computations - hydrodynamic network and velocity field (variant B1)

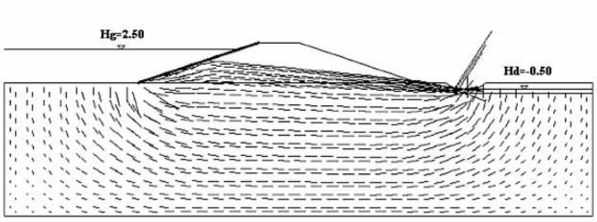

Fig. 5 Result of computations - hydrodynamic network and velocity field (variant B2)

5. Calculations of embankment stability

The stability calculations of the downstream slope of embankment were done for the representative cross-section shown on the scheme (Fig. 3). The scheme contains the assumed dimensions and geotechnical parameters of the basement soil and body of embankment. The computations were done using the 'Fellenius' method using our own computer program SZMF. The lowest value of stability coefficient was $F_s = 1.439$.

6. Result of computations for seepage (variant B1 and B2)

The result of computations are given in 4 parts of this paper. The hydrodynamic network and velocity fields are presented in Figs. 4 and 5. There was a seepage on the toe of the upstream slope of the embankment in the variant B1 (fig. 4). However in the variant 62 there was not direct seepage but the depression curve was almost on the surface of the terrain on the toe of embankment. The values of the inflow rate on 1 linear meter of the trench in variants B_1 and B_2 amount accordingly to $q_r = 18.31\ m^3/d$ and $19.58\ m^3/d$.

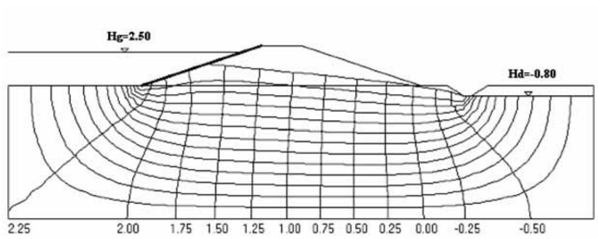

Fig. 6 Results of embankment stability computations

The maximal values of the hydraulic head in the inflow zone to the surrounding collector trench amount to: for variant B_1 – $I_{max} = 0.581$ and for variant B_2 - $I_{max} = 0.984$.

The critical slope of base of embankment amount to $I_{crit} = 0.99$ and the porosity $n = 0.40$ so the constant coefficient of destruction of the hydro technical structure is:

For variant B_1

$$\gamma_n = \frac{I_{crit}}{I_{max}} = \frac{0.99}{0.581} = 1.70 > 1.2 \quad (3)$$

For variant B_2

$$\gamma_n = \frac{I_{crit}}{I_{max}} = \frac{0.99}{0.984} = 1.70 > 1.2 \quad (4)$$

The computer calculations of filtration were done for the afore-mentioned schemes assuming that the filtration coefficients were given on the background of the geotechnical research referring to the object [3]. The permeable basement of the embankment with a growing depth characterizes the increase of water permeability. The value $k = 18.0$ m/d was assumed by computations. The filtration process through the base of the embankment's upper section with a smaller filtration coefficient, had an essential influence on the value of flow. The calculated values of the velocities and rate of inflows to the drainage devices could be overestimated proportionally to assumed the value of the filtration coefficient.

The shape and location of the depression curve confirms the appropriateness of using the tight screen on the upstream slope and also suggests the need for drainage (breakstone) in the lower section of the downstream slope. The high (locally) hydraulic heads ($I_{max} = 0.984$) on the inflow to the trench induce the need for ground protection against flux and erosion by using filtration geotextile. The assumed scheme for calculations (Fig. 3) takes into consideration the unfavorable conditions of stability, meaning a basement and body of embankment are water permeable. The obtained value of the smallest stability coefficient $F_s = 1.439$ (Fig. 6) meets the stability condition because $F_s = 1.439 > 1.2$.

7. Conclusions

1. For ensuring the safe conditions of filtration it is necessary to expect a tight screen on the water side slope on all the length of the designed embankment on firm grounds.
2. There is a need for ground protection against flux and erosion by using the filtration geotextile because of the high (locally) hydraulic heads ($I_{max} = 0.984$) on the inflow to the trench in free water table conditions.
3. To ensure the stability it is necessary to expect drainage (breakstone) on all the lengths of embankment in the lower section of the downstream slope protecting it against the water outflow on the slope.
4. The obtained calculated value of the smallest stability coefficient using the 'Fellenius' method meets the stability condition of the downstream slope of embankment.
5. Computer analysis of conditions of free filtration and stability of the embankment allowed, at the stage of working out of engineering design, the undertaking of solutions ensuring the safety for the structure.

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