CZASOPISMO TECHNICZNE
WYDAWNICTWO POLITECHNIKI KRAKOWSKIEJ
z. 3-Ś/2008
ISSN 0011-4561
ISSN 1897-6336

ANDRZEJ TRUTY, JAN SZARLIŃSKI, KRZYSZTOF PODLEŚ,
PAWEŁ ZIOBROŃ*

PARAMETRIC EFFECTIVENESS ASSESMENT OF JET GROUTING CUTOFF WALLS IN REHABILITATED FLOOD-PROTECTIVE DIKES

PARAMETRYCZNA OCENA SKUTECZNOŚCI PRZESŁON INIEKCYJNYCH W REMONTOWANYCH WAŁACH PRZECIWPOWODZIOWYCH

A b s t r a c t

In the paper a problem of effectiveness of jet grouting cutoff walls with respect to the reduction of volume/amount of water seeping through the body and the ground of the reconstructed/renovated flood protective dikes has been considered and analyzed. A parametric study for one cross section of the Vistula river dike in the Kaniów village has been made.

Keywords: dikes, seepage, cutoff walls, numerical simulations

S t r e s z c z e n i e

W artykule rozważono problem efektywności przesłon typu iniekcyjnego w kontekście redukcji objętości przepływu filtracyjnego przez korpus i podłoże remontowanych wałów przeciwpowodziowych. Wykonano studium parametryczne dla jednego przykładowego przekroju na Wiśle w Kaniowie.

Słowa kluczowe: wały przeciwpowodziowe, przesłony szczelne, symulacje numeryczne

* Dr hab. inż. prof. PK Andrzej Truty, Prof. dr hab. inż Jan Szarliński, dr inż. Krzysztof Podleś, mgr inż. Paweł Ziobroń, Zakład Podstaw Konstrukcji Inżynierskich, Instytut Geotechniki, Wydział Inżynierii Środowiska, Politechnika Krakowska.

1. Introduction

Rehabilitation and renovation of existing flood-protective dikes is a very important problem facing civil engineers, since a large number of them do not ensure sufficient safety in the cases of passing the flood waves, for which they are, or at least should be, designed. Thus, this problem of improving existing conditions of the dikes not satisfying safety requirements, especially those concerning seepage and stability ones, by increasing their reliability in this respect by application of Jet Grouting, executed/constructed inside the dikes, has been undertaken within the framework of the PRODICON project. The main task of the Politechnika Krakowska (PK) research group engaged in the Project is to compute, determine and analyze the behaviour and safety of dikes in various rather extreme situations, for which they should be foreseen and designed, to assess effectiveness of the proposed PRODICON measures. In the first stage of the PK activity, since data for the actual and real bench-dike have not been available yet, a parametric study/analysis has been carried out by the PK group for a typical dike, with given (fixed) dimensions like those of the dike in Kaniow (0+100 km) of Vistula river, but with various lengths of the cutoff wall and levels of water on the river side, and such a study/analysis is presented in the paper, along with results of the computations and mechanical analysis of the dike, as well as some conclusions, which can be drawn from them. The following text constitutes an excerpt from a broader elaboration, namely from a periodical report issued within the PRODICON project in the second year of its running (which is to be delivered soon), thus everybody interested in some more detailed data concerning the pertinent problem is referred to this foreseen report [7].

2. Rehabilitation technologies in the PRODICON project

Fig. 1. Installation of injection pipes
Rys. 1. Instalacja rur iniekcyjnych

The main goal of the PRODICON project was to enhance the existing product called

MTG-I, MTG-II used in the injection technologies applied to rehabilitation of dikes and making impermeable barriers. The most important aspects studied in the project were related to the modification of the MTG product with bentonite and fly ashes additives increasing the injection range for the assumed pressure, inject workability and finally tightness of the composite soil-inject. The dikes rehabilitation PRODICON technology consists of the two steps, first installation of the perforated injection pipes (see Fig. 1) and then injection, from the bottom to the top, under pressure of 120...180 [bar] of the MTG product. The injection pipes are located along two axes shifted by the expected range of the injection (see Fig. 2) to avoid potential imperfections. The final effect of the injection is shown in Fig. 3. Due to the limited space of the article the interested reader should refer to the PRODICON web site www.prodicon.eu for more details.

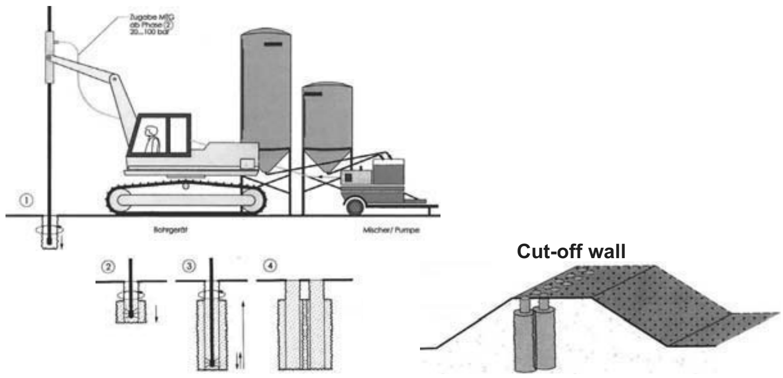

Fig. 2. Injection technology
Rys. 2. Technologia iniekcji

Fig. 3. Injection columns after excavation
Rys. 3. Kolumny iniekcyjne po wykonaniu odkrywki

3. Theoretical model of uncoupled/coupled total stress analysis of dikes

The detailed description of the theory of the two-phase partially saturated medium, following work by Aubry [2] and Lewis and Schrefler [5], and related implementation issues are given in [6]. Hence only a short summary of it is given here. The strong form of the problem consists of the following differential equations and the corresponding boundary and initial conditions

- the overall equilibrium equation written in terms of the total stress

$$\sigma_{ij,j}^{\mathrm{tot}} + b_i = 0 \tag{1}$$

$$b_i = \left(\gamma_{\mathrm{dry}} + n\,S\,\gamma^{\mathrm{F}}\right)\bar{b}_i \tag{2}$$

- the extended effective stress concept after Bishop

$$\sigma_{ij}^{\mathrm{tot}} = \sigma_{ij} + \delta_{ij}\; S\; p \tag{3}$$

- the fluid flow continuity equation including the effect of fluid compressibility and partial saturation

$$S\,\dot{\varepsilon}_{kk} + v_{k,k}^{\mathrm{F}} = \left(n\frac{S}{K^F} + n\frac{\partial S}{\partial p}\right)\;\dot{p} \tag{4}$$

- the linearized strain-displacement relations

$$\varepsilon_{ij} = \frac{1}{2}\left(u_{i,j} + u_{j,i}\right) \tag{5}$$

- a nonlinear elasto-plastic constitutive relation for solid phase

$$\dot{\sigma}_{ij} = D_{ijkl}^{\mathrm{e}}\left(\dot{\varepsilon}_{kl} - \dot{\varepsilon}_{kl}^{\mathrm{p}}\right) \tag{6}$$

- the extended Darcy's law for fluid velocity

$$v_i^{\mathrm{F}} = -k_r(S)\,k_{ij}\left(-\frac{p}{\gamma^{\mathrm{F}}} + z\right)_{,j} \tag{7}$$

- constitutive equations for saturation ratio S (valid only for suction), after van Genuchten [3], and relative permeability coefficient $k_r(S)$ after Irmay [4]

$$S(p) = S_r + (1 - S_r)\left[1 + \left(\alpha \frac{p}{\gamma^{\mathrm{F}}}\right)^n\right]^m \tag{8}$$

$$k_r(S) = \left(\frac{S - S_r}{1 - S_r}\right)^3 \tag{9}$$

where the residual saturation ratio is denoted by S_r, and α is a material parameter responsible for decreasing the saturation ratio with increasing pressure suction while parameters n and m are fixed to the values $n = 2$ and $m = -0.5$,

- boundary conditions to be satisfied at any time $t \in [0, T]$

$$\sigma_{ij}^{\mathrm{tot}} n_j = \overline{t_i} \text{ on } \Gamma_{\mathrm{t}} \tag{10}$$

$$v_i^{\mathrm{F}} n_i = \overline{q} \text{ on } \Gamma_{\mathrm{q}} \tag{11}$$

$$u_i = \overline{u}_i \text{ on } \Gamma_{\mathrm{u}} \tag{12}$$

$$p = \overline{p} \text{ on } \Gamma_{\mathrm{p}} \tag{13}$$

Γ_{t}, Γ_{q}, Γ_{u}, Γ_{p} are parts of the boundary where the total stresses, fluid fluxes, displacements and pore pressures are prescribed,

- initial conditions

$$u_i(t = t_o) = u_{io} \tag{14}$$

$$p(t = t_o) = p_o \tag{15}$$

3.1. van Genuchten model

The simplified van Genuchten model used for seepage analysis for partially saturated media depends on the two material parameters α and residual saturation ratio S_r. In the above formulation no hysteretic effects, due to cycles of wetting and drying, are taken into account, therefore parameter α is estimated to get best fit for both wetting and drying experimental curves $S(p)$. It must be emphasized here that in the practice we do not have access to such a data, hence some values of these parameters, for certain classes of soils, have to be extrapolated from the existing experiments. The comprehensive experimental data was recently published in the paper by Yang et. al [8]. To illustrate how the simplified van Genuchten model fits the experiments the analytical and experimental results for medium sand (Fig. 4), with the following grain distribution characteristics $D_{60} = 1.25$ mm, $D_{30} = 0.62$ mm, $D_{10} = 0.29$ mm, porosity $n = 0.35$, then for fine sand (Fig. 5) with $D_{60} = 0.35$ mm, $D_{30} = 0.23$ mm, $D_{10} = 0.17$ mm, porosity $n = 0.411$, clayey sand I (Fig. 6) with $D_{60} = 0.66$ mm, $D_{30} = 0.051$ mm, $D_{10} = 0.003$ mm, porosity $n = 0.348$ and clayey sand II (Fig. 7) with $D_{60} = 0.56$ mm, $D_{30} = 0.021$ mm, $D_{10} = 0.0005$ mm, porosity $n = 0.432$ are presented. For medium and fine sand the analytical and experimental results match very well while for the other cases it is slightly worse. A better matching, as it is shown in the paper by Yang [8], can be obtained for different, than default, values of m and n parameters.

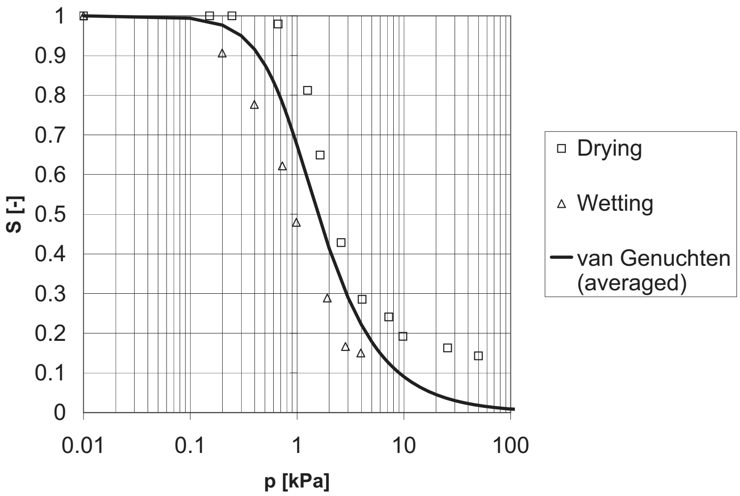

Fig. 4. van Genuchten approximation of $S(p)$ curve for medium sand ($\alpha = 11\ \mathrm{m}^{-1}$, $S_r = 0$)
Rys. 4. Aproksymacja van Genuchtena krzywej $S(p)$ dla piasku średniego ($\alpha = 11\ \mathrm{m}^{-1}$, $S_r = 0$)

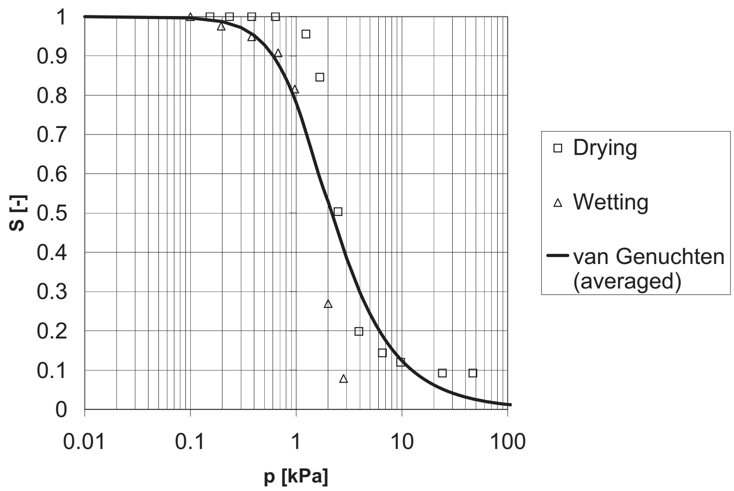

Fig. 5. van Genuchten approximation of $S(p)$ curve for fine sand ($\alpha = 8\ \mathrm{m}^{-1}$, $S_r = 0$)
Rys. 5. Aproksymacja van Genuchtena krzywej $S(p)$ dla piasku drobnego ($\alpha = 8\ \mathrm{m}^{-1}$, $S_r = 0$)

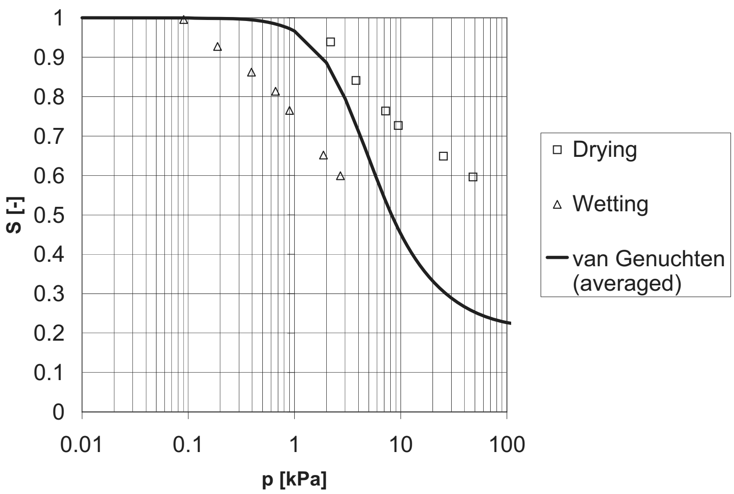

Fig. 6. van Genuchten approximation of $S(p)$ curve for clayey sand ($\alpha = 3$ m^{-1}, $S_r = 0.2$)
Rys. 6. Aproksymacja van Genuchtena krzywej $S(p)$ dla piasku gliniastego ($\alpha = 3$ m^{-1}, $S_r = 0.2$)

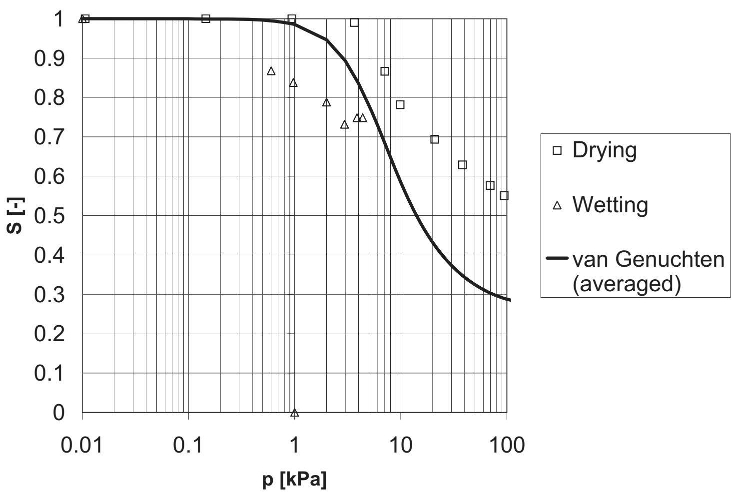

Fig. 7. van Genuchten approximation of $S(p)$ curve for clayey sand ($\alpha = 2$ m^{-1}, $S_r = 0.25$)
Rys. 7. Aproksymacja van Genuchtena krzywej $S(p)$ dla piasku gliniastego ($\alpha = 2$ m^{-1}, $S_r = 0.25$)

4. Application to Vistula river dike at Kaniów

The cross-section of the rehabilitated dike of the Vistula river, located in the village of Kaniow (0+100 km), has been taken as an example for parametric studies carried out with aid of the FEM program Z_Soil.PC v2009. This dike due to relatively high permeability of the embankment was supposed to be rehabilitated by using injection methods and hence the question was what the optimal suspension level of the cutoff wall should be. The cross section of the dike is presented in Fig. 8. The embankment is built of rock waste material (zone M1 in Fig. 8) (this material after certain time of deposition, change of the water content and sudden change of the confining stress takes form of a granular medium with a relatively low seepage coefficient). The other four important subsoil layers are: medium sand (zone M2), gravelly sands (zone M3), peat (M5) and tertiary clays (zone M4). The layer tertiary clays can be treated as impermeable during flood wave transition. Injection cutoff walls, like MTG for instance, have rather low permeability far below 10^{-8} m/s and their thickness is usually close to 40cm. This yields certain difficulties in the finite element modeling as size of the embankment is rather large compared to the cutoff wall thickness. To eliminate the need of discretization of the cutoff wall along the thickness and to avoid badly shaped finite elements in that zone the domain of the cutoff wall can be represented in the discrete model by means of an artificial (zero thickness) contact interface with an equivalent transversal seepage coefficient $k^* = \frac{k^d}{t}$ with thickness of the cutoff wall denoted by t and seepage coefficient of the cutoff wall matrix denoted by k^d. Assuming no material neither geometrical imperfections one may treat the cutoff wall, in the analysis of flood wave transition, as fully impermeable. All older and current versions of Z_Soil program offer this kind of the modeling tools to the user.

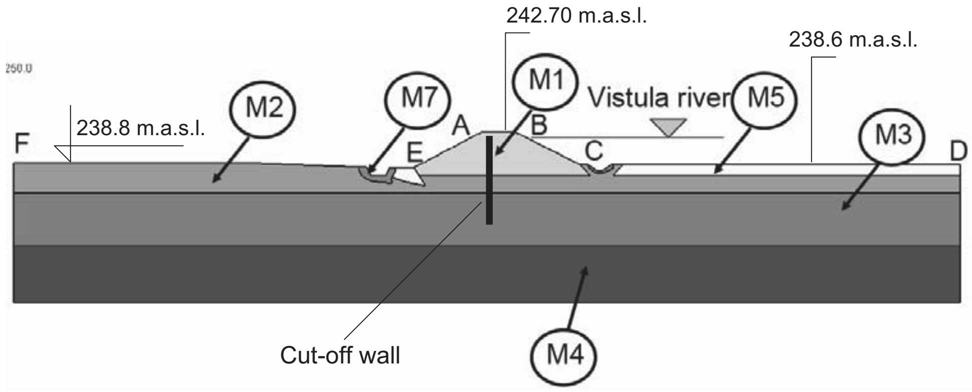

Fig. 8. Cross section of the dike
Rys. 8. Przekrój poprzeczny wału przeciwpowodziowego

The typical flood hydrograph for the Vistula river at the considered cross section is shown in Fig. 9. This hydrograph becomes an input for setting varying in space and time, pressure boundary conditions, at the boundary B-C-D (Fig. 8). To simplify this setting the pressure potential $h(t)$ definition ($h(t) = -\frac{p}{\gamma^F} + y$) is used. A special treatment is needed along free boundaries (A-E-F) in order to allow for potential outflow through the downstream face of the embankment (boundary A-E) and subsoil surface (boundary E-F) (see Fig. 8). As the pressure field in the domain is not known apriori, therefore explicit setting of zero flux or zero pressure boundary conditions along these boundaries is not possible. In the Z_Soil program special seepage elements [9] are added at the foregoing domain boundaries to automatically switch from zero flux to zero pressure once the compressive pore water pressure is detected at the domain boundary. To start the analysis of flood wave transition we need also to specify the initial conditions for the pore pressures and for the in situ effective stresses. This can easily be made by running the initial state analysis which yields the initial effective stress state as well as the steady state solution for the pressure field at time $t = 0$.

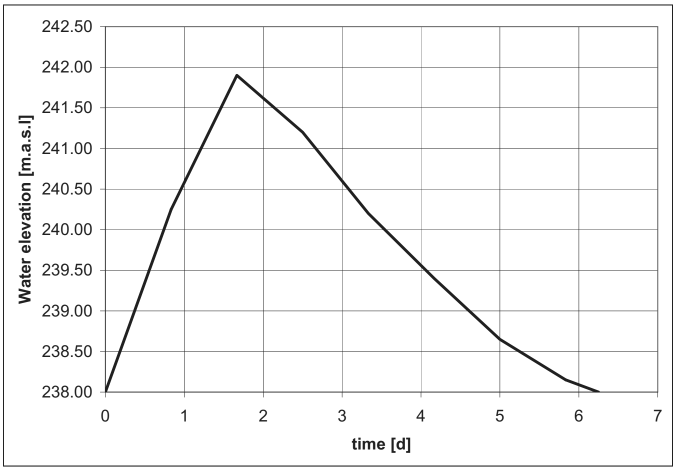

Fig. 9. Assumed flood hydrograph
Rys. 9. Założony hydrogram powodzi

The material properties of all the material layers are specified in the Table 1. The stiffness moduli were taken after Polish code PN-03020 [1] but it is obvious that values of these parameters can be far from the measured ones. As the stiffness moduli do not affect values of the stability factors and problem is driven mainly by the transient pore pressure field, caused by varying in time pressure boundary conditions, therefore even rough estimation of the stiffness should be good enough from the practical point of view. For all materials the residual saturation ratio was assumed as $S_r = 0$. This is very important to avoid additional cohesive effect due to partial saturation and to get safety factor predictions being on the safe side. It has to be emphasized here that the strength parameters must be taken as effective while the ones given in the Polish code [1] do not usually satisfy this restriction. This problem concerns mainly cohesive soils for which effective friction angle is usually larger than the one given in the Code while cohesion is definitely smaller. The relatively high value of seepage coefficient in the embankment has been estimated by back analysis to confirm presence of leakage effects on the downstream face of the dike during flood transition.

Table 1

Material properties

Mat.	E	ν	γ_{dry}	e_o	k	c	ϕ	α
	[kN/m^2]	[-]	[kN/m^3]	[-]	[m/d]	[kN/m^2]	^{o}	[m^{-1}]
M1	56 000	0.35	18	0.47	2	5	34	5
M2	80 000	0.25	16.5	0.61	3.75	0	35	10
M3/M7	180000	0.25	18.5	0.43	700	0	38	10
M4	80000	0.32	20.0	0.32	0.0001	59	13	0.5
M5	10000	0.45	11	1.4	0.01	10	10	1

4.1. Cutoff wall effectiveness

Designing length of the cutoff wall is not so straightforward as it must take into account current technical state of the dike, potential existing imperfections in the structure and material properties of the dike and its subsoil. To assess the cutoff wall effectiveness, by taking exclusively the effect of reduction of the outflow, the following two measures are proposed

$$E^{\max} = \left(1 - \frac{Q_{\max}}{Q^*{}_{\max}}\right) \times 100\% \tag{16}$$

$$E = \left(1 - \frac{\int_{t_o}^{t_{\text{end}}} Q(t)dt}{\int_{t_o}^{t_{\text{end}}} Q^*(t)dt}\right) \times 100\% \tag{17}$$

where

$$Q_{\max} = \max\left(Q(t)\right) \quad \text{for} \quad \forall\, t \in < t_o, t_{\text{end}} > \tag{18}$$

These measures can be applied to the outflow through the embankment only (then $Q(t) = Q_k(t)$), to the outflow through the ground surface (then $Q(t) = Q_p(t)$) as well as to the sum of the outflows through the embankment and ground surface (then $Q(t) = Q_k(t) + Q_p(t)$). The corresponding outflows for case of zero length cutoff wall are denoted by Q^*. The t_o is the initial time of flood wave transition while t_{end} is the time when the steady state is achieved.

4.2. Parametric analysis

The dike cross section, shown in Fig. 8, was considered as a template one, and the main goal of carried out parametric analysis was to assess the cutoff wall effectiveness assuming eight different lengths for the cutoff wall, specified in the Table 2. In the considered case the coupled total stress analysis was carried out starting from $t_o = 1$ [d] to $t_{\text{end}} = 25$ [d]. In the period $t = (1.0...6.25)$ [d] constant time steps were used with $\Delta t = 0.1$ [d], while later on, up to time $t_{\text{end}} = 25$ [d], a variable time steps were used by using the following recurrence formula $\Delta t_N = \Delta t_{N-1} \times 1.05$ with the initial $\Delta t_o = 0.1$ [d].

4.3. Results

The outflow time histories through embankment only $Q_k(t)$, through ground surface only $Q_p(t)$ and the overall one $Q(t) = Q_k(t) + Q_p(t)$ are shown in Fig. 10, Fig. 11 and Fig. 12 respectively. These results confirm reaching the steady state condition before $t_{\text{end}} = 25$ [d]. The

Table 2

Design variables

Cutoff wall elevation [m.a.s.l]	Cutoff wall length [m]
242.7	0
237.3	5.4
236.4	6.3
235.1	7.6
234	8.7
233	9.7
232	10.7
230.6	12.1

variation of the cutoff wall effectiveness measures with respect to the cutoff wall length are shown in Fig. 13 and Fig. 14. One may notice that the two proposed measures give different magnitudes of the outflow reduction. As it could be expected the measure $E_{\max}$ yields larger values than the E one for any length of the cutoff wall. It is also worth to mention that in the considered case safety factors, computed by means of $c - \phi$ reduction method, at the state of the flood wave culmination ($t = 1.67$ [d]) and after flood transition ($t = 6.25$ [d]) are not affected by the length of the cutoff (see Fig. 15).

5. Conclusions

The authors of the paper regard that the following conclusions can be drawn on the basis of the material presented above

1. The FEM software applied by them in the analysis of the dike structures is fully capable and suitable to such civil engineering problems as rehabilitating and strengthening them by internal cutoff walls constructed inside their bodies.

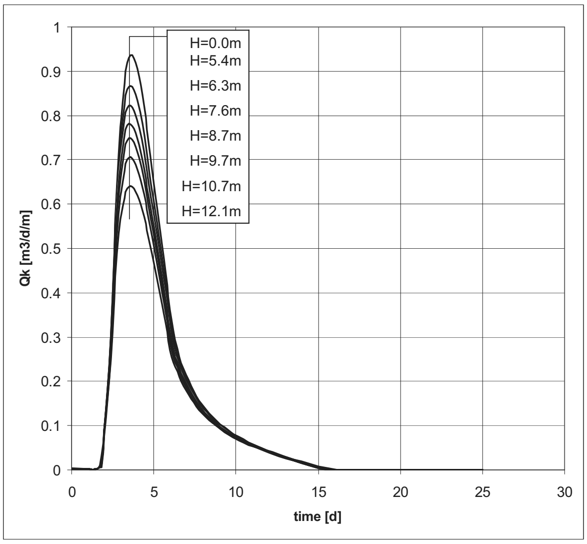

Fig. 10. Outflow time history $Q_k(t)$ [m^3/d/m]
Rys. 10. Przebieg czasowy objętości przepływu $Q_k(t)$ [m^3/d/m]

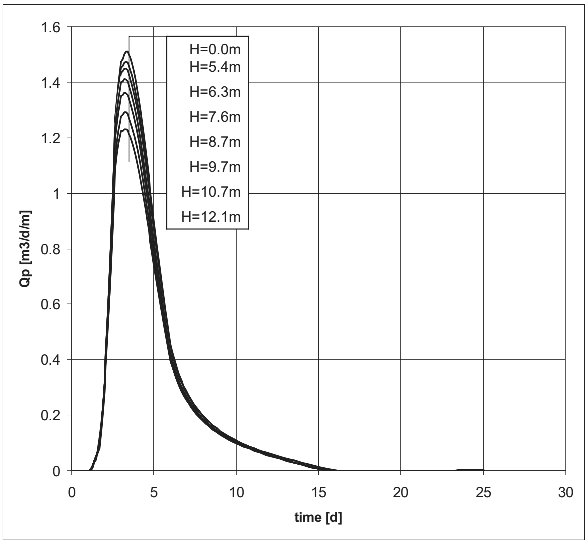

Fig. 11. Outflow time history $Q_p(t)$ [m^3/d/m]
Rys. 11. Przebieg czasowy objętości przepływu $Q_p(t)$ [m^3/d/m]

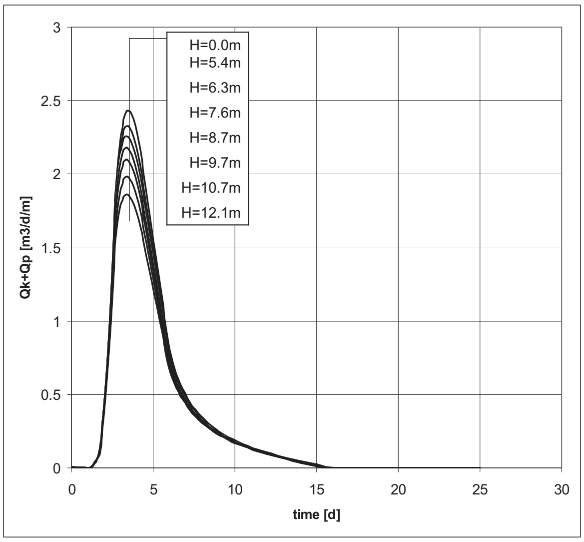

Fig. 12. Outflow time history $Q_{k+p}(t)$ [m^3/d/m]

Rys. 12. Przebieg czasowy objętości przepływu $Q_{k+p}(t)$ [m^3/d/m]

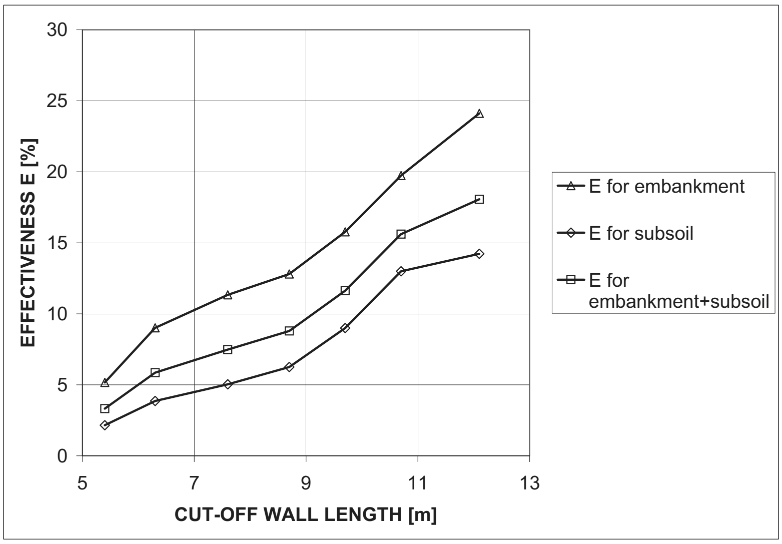

Fig. 13. Cutoff wall effectiveness $E(H)$

Rys. 13. Efektywność przesłony $E(H)$

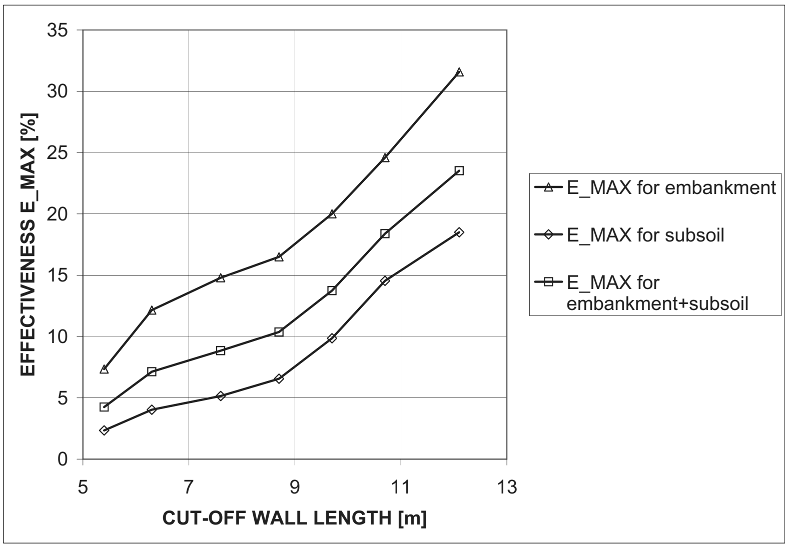

Fig. 14. Cutoff wall effectiveness $E_{\max}(H)$
Rys. 14. Efektywność przesłony $E_{\max}(H)$

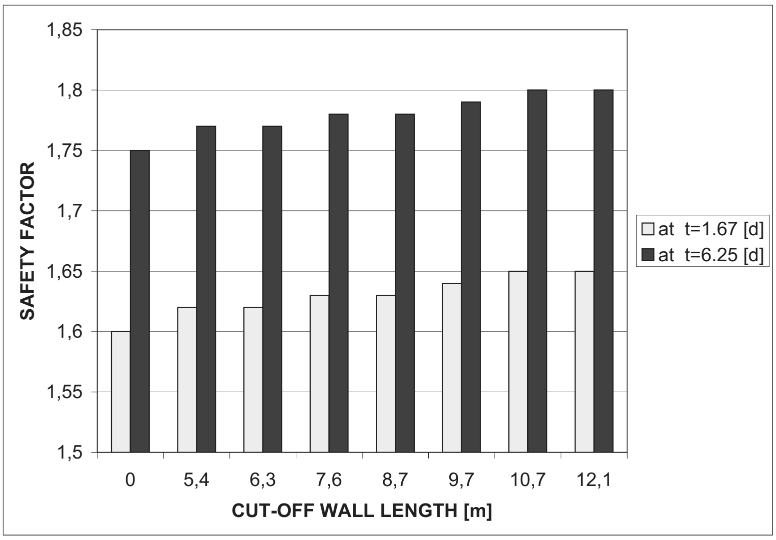

Fig. 15. Evolution of safety factors with cutoff wall length
Rys. 15. Zmiany wartości współczynników bezpieczeństwa wraz z długością przesłony

2. The formulae and procedures elaborated originally by the PK team within the PRODICON project to determine quantitatively effectiveness of the cutoff walls as a function of their heights, for any period of time (e.g. short term river floods or long term seepage through embankments of the water reservoirs), are useful and can be applied in the engineering practice.

3. The minimum cutoff wall length should be larger than the embankment height to eliminate potential imperfections in its interior (damages caused by mole like animals etc.).

4. In the particular case the optimal cutoff wall length should be selected in the range where the gradient of the effectiveness parameters is the steepest, however they cannot cutoff the inflow of the groundwater from the outside to the river bed.

References

[1] Grunty budowlane. Posadowienie bezpośrednie budowli. Obliczenia statyczne i projektowanie. PN-81/B-03020.

[2] Aubry D., Ozanam O., *Free-surface tracking through non-saturated models*, Numerical Methods in Geomechanics, Innsbruck, 1988, 757–763.

[3] Van Genuchten, *A closed form equation for predicting the hydraulic conductivity of unsaturated soils*, Soil Sciences Am. Soc., 1980, 44:892–898.

[4] Irmay S., *On the hydraulic conductivity of unsaturated soils*. Trans. Am. Geophys. Union, 1956, 35:463–468.

[5] Lewis R. W., Schrefler B. A., *The Finite Element Method in the Static and Dynamic Deformation and Consolidation of Porous Media*, John Willey and Sons Ltd., Baffins Lane Chichester, West Sussex PO19 1UD, England, 1998.

[6] Truty A., *On certain classes of mixed and stabilized mixed finite element formulations for single and two-phase geomaterials*, Zeszyty Naukowe Politechniki Krakowskiej, Seria Inżynieria Środowiska 48, Kraków, 2002.

[7] Truty A., Szarliński J., Ziobroń P., *Land protection by improvement of dike construction: Report. mathematical model*, Report 01 in project PRODICON COOP-CT-2006-032847, Institute of Geotechnics, Cracow University of Technology, May 2007.

[8] Hong Yang, Harianto Rahardjo, Eng-Choon Leong, Fredlund D.G., *Factors affecting drying and wetting soil-water characteristic curves of sandy soils*. Canadian Geotechnical Journal, 2004, 41:908–920.

[9] Zimmermann Th., Truty A., Urbański A., Podleś K., *Z_SOIL.PC 2007 3D user manual: Theory, Tutorials and benchmarks, Data preparation*, Elmepress International & Zace Services Ltd, Switzerland, 2007.