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ANALYSIS OF SOFT SOIL CONSOLIDATION WITH THE APPLICATION OF PREFABRICATED VERTICAL DRAINS WITH PRELOADING METHOD USING FEM

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The article presents the application of finite element method for estimating settlements of road embankments founded on the soil reinforced with vertical drains and preloading method. The idea of the method was the transition from the solution of one-dimensional consolidation proposed for two-dimensional solution, while maintaining the same consolidation time and comparison with results obtained from measurements settlements of road embankment which is a part of planned Gdańsk Southern Ring Road near Przejazdowo site.

1. Introduction

Prefabricated vertical drains with preloading have been widely used for improving parameters of soil soil. The main task of vertical drains is to shorten drainage path and thereby speed up consolidation process. Barron (1948) provided solution for one-dimensional consolidation:

$$-\frac{\partial \varepsilon_z}{\partial t} = \frac{k_v}{\gamma_w}\frac{\partial^2 u}{\partial z^2} + \frac{k_h}{\gamma_w}\left(\frac{\partial^2 u}{\partial r^2} + \frac{1}{r}\frac{\partial u}{\partial r}\right) \tag{1}$$

Where k_v and k_h are respectively: vertical and horizontal filtration ratios, γ_w is bulk density of water, ε_x is vertical displacement, u is excess water pressure in pores, r is radial coordinate, z is vertical coordinate, t is time.

The solution presented by Barron includes a number of simplifications, among others, it assumes constant parameters for soil rigidity and permeability during entire consolidation process. It also does not take into consideration ”smearing” effect, i.e. damage of soil structure around the applied vertical drain.

Degree of consolidation U is defined as the level of dispersion of the excess water pressure in pores and is expressed in the form proposed by Carillo (1942):

$$(1 - U) = (1 - U_v) \cdot (1 - U_r) \tag{2}$$

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Where U is the degree of total consolidation, U_v is the degree of horizontal consolidation, U_r is the degree of radial consolidation.

This article presents the use of the finite element method and implemented visco-elastic-plastic model for calculating the settlements and dispersing excess water pressure in pores.

The calculations include the solution for one-dimensional consolidation in axisymmetrical system, which was presented by Barron, and then adapted to two-dimensional Plane Strain Condition for homogenized material with equal time of settlements.

The solution has been checked on the existing construction embankment located in the site of Gdansk Southern Ring Road.

2. CONSTITUTIVE MODEL

The analysis was made using constitutive model which allowed for propagation of strain caused by constant effective stress. Bjerrum (1967) suggested to divide consolidation into 2 parts: primary consolidation in which settlements depends on pressure dissipation in pores and change in porosity of the soil, and secondary consolidation in which soil compression takes place with stable level of stress.

In 1979 Butterfield proposed creep equation in the form of:

$$\varepsilon^H = \varepsilon_c^H + \mu^* \cdot \ln\left(\frac{\tau_c + t'}{\tau_c}\right) \tag{3}$$

where ε_C^H is the strain formed during consolidation. Modified creep ratio μ^*, describes secondary consolidation on the logarithmic time scale.

It needs to be differentiated that τ_c is not consolidation time t_c. Time τ_c is not a material parameter but depends on consolidation and geometry of tested sample. Janbu (1969) proposed geometrical interpretation which allows to determine τ_c i μ^* directly from empirical data shown in Figure 1.

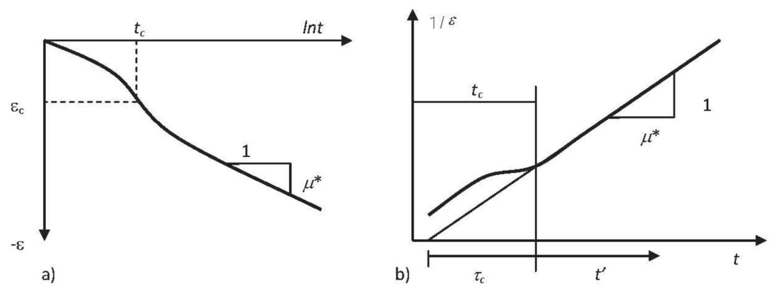

Fig. 1. Consolidation and creep of soil in standard oedometer survey.
Rys. 1. Konsolidacja i pełzanie gruntu w standardowym badaniu edometrycznym

Taking into consideration creep of soil, equation of total strain has the form of:

$$(4)\quad \varepsilon_v = \varepsilon_v^e + \varepsilon_v^{cr} = \varepsilon_{vc}^e + \varepsilon_{vc}^{cr} + \varepsilon_{vac}^{cr} = \kappa * \ln\left(\frac{p'}{p'_0}\right) + (\lambda * - \kappa *)\ln\left(\frac{p'_{pc}}{p'_{p0}}\right) + \mu * \ln\left(\frac{\tau_c + t'}{\tau_c}\right)$$

where ε_v is total volumetric strain caused by increase of mean stress p'_0 to the value p' within the time $t_c + t'$. Total volumetric strain can be divided into elastic part ε_v^e, and visco-plastic part ε_v^{cr}.

Equation (4) includes modified primary and secondary compressibility ratios, and modified secondary consolidation ratio, which are in the following relation to the standard ratios:

$$(5)\qquad \lambda^* = \frac{C_c}{2.3 \cdot (1 + e)}$$

$$(6)\qquad \kappa^* \approx \frac{2}{2.3}\frac{C_s}{1 + e}$$

$$(7)\qquad \mu^* = \frac{C_\alpha}{2.3 \cdot (1 + e)}$$

Classic Mohr-Coulomb criterion has been applied as the criterion for soil shear, where yield point is determined by the function $f(\phi, c)$, where ϕ – soil internal friction angle, c – cohesion.

Commercial program for finite elements Plaxis with implemented Soft Soil Creep model was used in FEM analysis.

3. The formula for transition from the solution of vertical drain consolidation in the in the axisymmetrical (1D) form to the solution in two-dimensional (2D) form

The idea of the method was the transition from the solution of one-dimensional consolidation proposed by Barron (1948) to two-dimensional solution, while maintaining the same consolidation time. The substitute filtration ratio for homogenized material, where $k_h = k_v$, is the unknown quantity.

In his solution, Barron (1948) divided consolidation into vertical and radial (horizontal):

$$(8)\qquad U_v = \left(1 + \frac{1}{2 \cdot T_v^3}\right)^{-\frac{1}{6}}$$

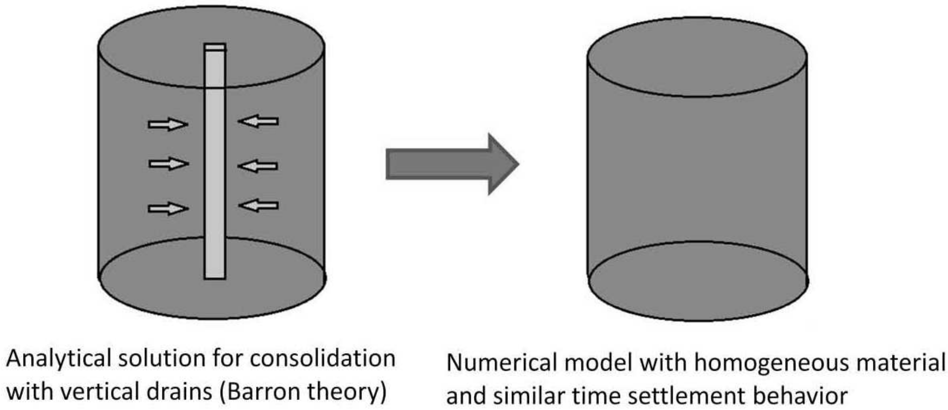

Fig. 2. The idea of transition from the analytical solution of axisymmetrical soil consolidation to two-dimensional consolidation in Plane Strain Conditions.
Rys. 2. Idea przejścia z analitycznego rozwiązania osiowosymetrycznej konsolidacji gruntu do dwuwymiarowej konsolidacji gruntu w płaskim stanie odkształcenia

(9)
$$T_v = \frac{C_v \cdot t}{H^2}$$

(10)
$$U_r = 1 - e^{\left(\frac{-8 \cdot C_h \cdot t}{D^2 \cdot F(n)}\right)}$$

Where: U_v i U_r are respectively: degrees of vertical and horizontal consolidation, T_v – time factor, C_v and C_h are respectively: vertical and horizontal consolidation ratios , H is thickness of drained layer (if it is possible to dissipate water pressure in pores at two edges of drained layer, then $H = 0.5 \cdot H$), D is the diameter of drained cylinder of soil, d is substitute diameter of drain, $F(n)$ is the function allowing for damage of soil structure around the drain due to its application in the soil.

Consolidation parameters (C_v and C_h) depend on soil rigidity, therefore substitute filtration ratio k, will be the function:

(11)
$$k_{(h,v)} = f\left(\lambda^*, k^*, F(n), D, H, C_v, C_h\right)$$

The figure below shows the results of numerical calculations:

The diagram shows analytical solution proposed by Barron (1948) which does not take into consideration “smearing” effect of the soil with numerical calibration. Numerical calibration puts emphasis on achieving biggest possible similarity of analytical solution with the numerical one for final consolidation time.

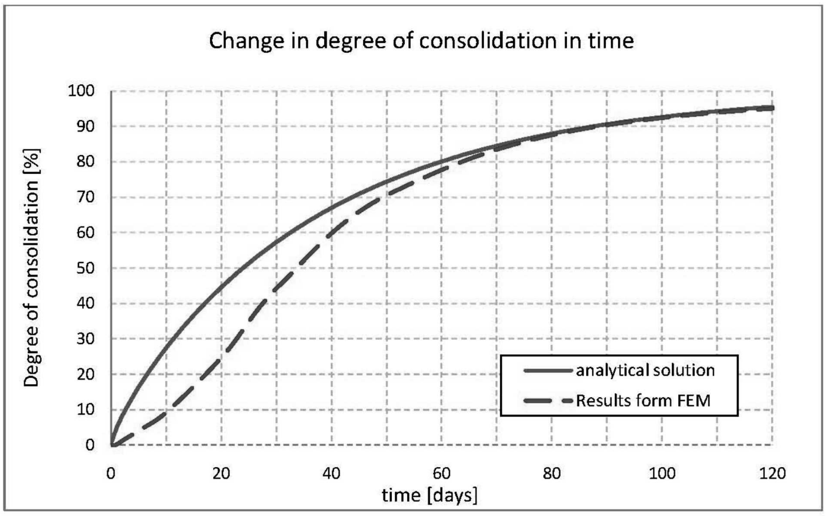

Fig. 3. Results of numerical simulation with analytical solution.
Rys. 3. Porównanie wyników numerycznej symulacji z rozwiązaniem analitycznym

4. Practical application

Road embankment, which is a part of Gdańsk Southern Ring Road, is located at 9+150 km, near Przejazdowo site. The foundation consists of a very compressible soil which contains organic elements at the level of 8.1%. Planned embankment height is 3.0 m high (above area level). Basis of the embankment is 40 m wide with embankment crown of 31 m. Planned reinforcement for the foundation is 2.5 m high preloading above the embankment for the approximate period of 6 months. Planned drain mesh is a square with a side of 1.0 m.

Soil profile was divided into 3 layers. First layer consists of fine and medium--compacted sand with thickness of 3.2 m. Then, here is poor soil layer with thickness of 5.3m on medium-compacted sands. Underground water can be found at ca. 0.7 m below level of the area. Parameters of the soil used in the analysis are presented in tables 2 and 3. Visco-elastic-plastic model (with OCR=1.3) was used in the analysis in order to reflect behavior of soft soil labeled with A1 code. Other layers were simulated using the Mohr-Coulomb's elastic-plastic model.

Digitizing of the model was carried out using triangular 15-nodal finite elements. Due to the fact that large strain was expected, updated Lagrange system was applied, in which mesh of finite elements deforms together with the material in successive calculation steps.

Material with modified filtration ratio (A1 - VD) was used below the embankment in order to reflect prefabricated vertical drains installed in it.

Table 1

Parameters of the soil used for calculations – elastic-plastic soil.

Parameter	Unit	Embankment	B1	IVc
γ_{unsat}	[kN/m^3]	17.8	17	19
γ_{sat}	[kN/m^3]	20	18	19
$\mathbf{k}_x$	[m/day]	1	1	1
$\mathbf{k}_y$	[m/day]	1	1	1
$\mathbf{e}_{init}$	[-]	0.5	0.5	0.5
$\mathbf{E}_{ref}$	[kN/m^2]	30000	15000	70000
ν	[-]	0.3	0.3	0.3
$\mathbf{c}_{ref}$	[kN/m^2]	2	2	1
φ	[°]	36.5	30	30
ψ	[°]	6.5	0	0

Table 2

Parameters of the soil used for calculations – visco-elastic-plastic model.

Parameter	Unit	A1	A1 - VD
γ_{unsat}	[kN/m^3]	8.62	8.62
γ_{sat}	[kN/m^3]	15.12	15.12
$\mathbf{k}_x$	[m/day]	8.60E-05	2.90E-03
$\mathbf{k}_y$	[m/day]	4.30E-05	2.90E-03
$\mathbf{e}_{init}$	[-]	1.97	1.97
Cc	[-]	0.65	0.65
Cs	[-]	0.11	0.11
$\mathbf{C}\alpha$	[-]	0.02	0.02
c	[kN/m^2]	1	1
φ	[°]	16	16
ψ	[°]	0	0
K_0^{nc}	[-]	0.815	0.815

Field measurements of settlements and results of the analysis were used to prepare comparative chart of measured and estimated settlements.

Estimated settlements, which take the creep of soil into consideration, correspond to the values of field-measured settlements. Results of traditional calculations of settlements, which do not take secondary consolidation into consideration, were presented as comparison.

Both calculation curves gave the same settlement delay mechanism as field-measured settlements. The reason of the above was the use of simplified homogenized material, which was similar to the use of vertical drains. However, with lapse of time, calculation settlements "joined" the measured settlements (comparison with Figure 3).

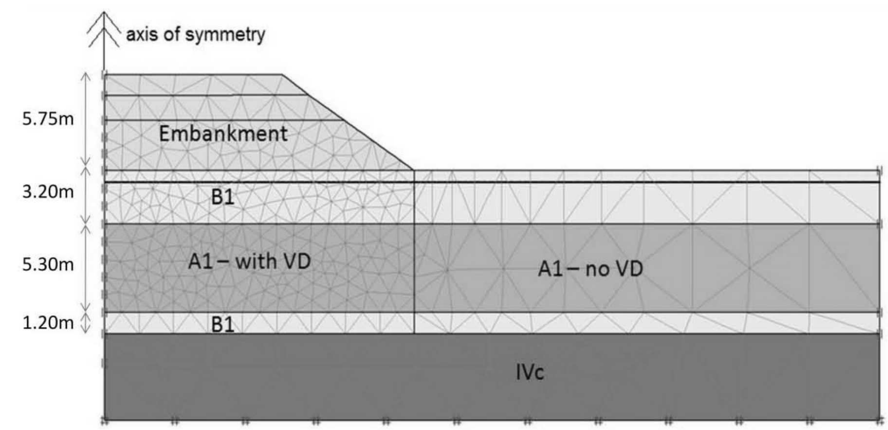

Fig. 4. Digitized calculation model.
Rys. 4. Zdyskretyzowany model obliczeniowy

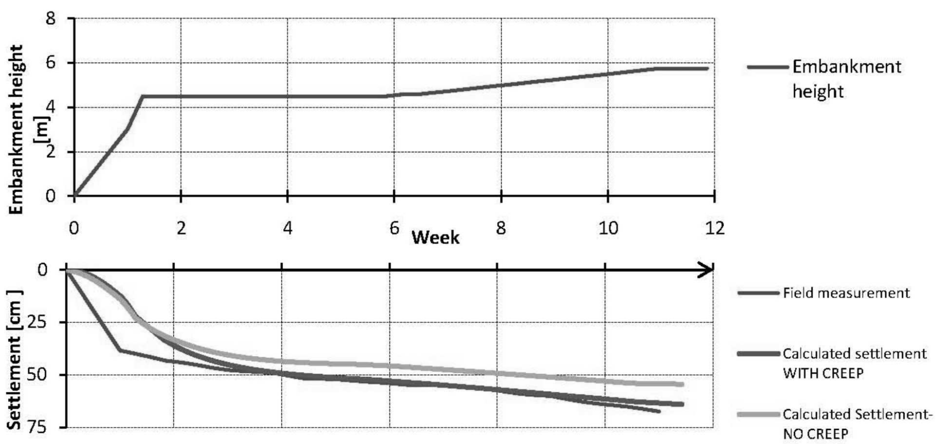

Fig. 5. Results of measurements of field and calculation settlements.
Rys. 5. Wyniki pomiarów osiadań terenowych oraz osiadań obliczeniowych

Figure 6 shows propagations of settlements after finishing dissipation of water excess in pores. This phenomenon is characteristic of poor soil whose viscosity causes additional plastic strains after finishing primary consolidation. The above-mentioned situation is shown in Figure 6, where the excess pore pressure is dispersed at settlement of ca. 0.52 m, and further strain takes places at constant pore pressure in the soil.

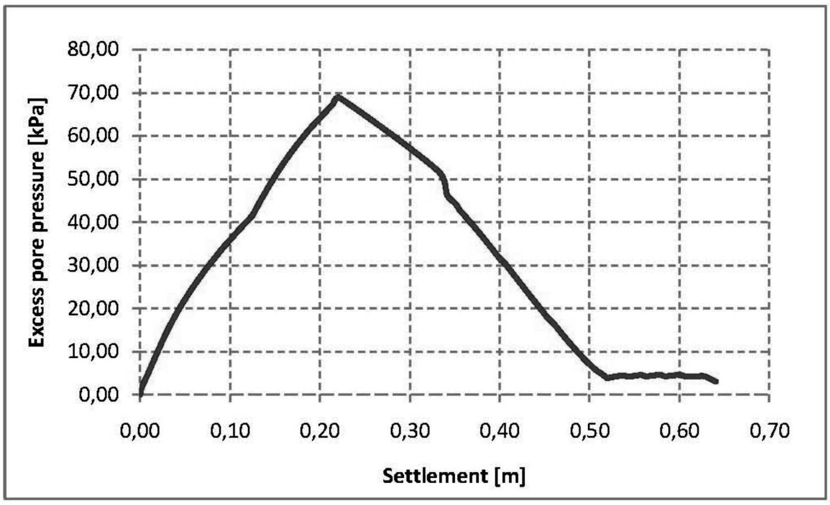

Fig. 6. Excess pore pressure vs Settlement – results of simulation using the finite element method for visco-elastic-plastic model.
Rys. 6. Zmiana nadwyżki ciśnienia porowatego w stosunku do osiadań – wyniki symulacji

5. Conclusions

The paper presents results of numerical simulation using finite element method for settlements of road embankment which is a part of a planned Gdańsk Southern Ring Road. Vertical drains used in the ground were included in the simplified scheme which consisted in homogenization of soil using the finite element method with constant consolidation time, which was in conformity with the analytical solution proposed by Barron.

Calculations of settlements using visco-elastic-plastic model correspond to the field-measured settlements. Discrepancies in the first stage of calculations are a result of applied methodology for simulation of vertical drainage in the ground.

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ANALIZA KONSOLIDACJI SŁABYCH GRUNTÓW PRZY UŻYCIU METODY ELEMENTÓW SKOŃCZONYCH

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

W artykule przedstawiono zastosowanie metody elementów skończonych przy przewidywaniu osiadań nasypów drogowych posadowionych na podłożu wzmacnianym przy użyciu nasypu przeciążającego i drenażu pionowego. Ideą metody było przejście z analitycznego rozwiązania jednowymiarowej konsolidacji do rozwiązania w Płaskim Stanie Odkształcenia z zachowaniem ekwiwalentnego czasu konsolidacji oraz porównanie z wynikami uzyskanymi z bezpośrednich pomiarów osiadań nasypu drogowego będącego częścią Obwodnicy Południowej Gdańska, zlokalizowanego w okolicach miejscowości Przejazdowo.

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