SLOVAK JOURNAL OF CIVIL ENGINEERING

2009/2 PAGES 26 – 37 RECEIVED 7. 5. 2008 ACCEPTED 10. 6. 2009

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MODELLING OF PRIMARY CONSOLIDATION

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Research Field: Material and geometrical nonlinear FEA, Settlement of saturated clays, Foundations of High-Rise Buildings

ABSTRACT

In this paper the time-dependent settlement of high-rise buildings is solved. The analysis is made with the results of experimental measurements performed by an oedometric unit and a mathematical model. The mathematical model was analyzed by the Finite Element Method and the ANSYS software system and compared with the experimental measurement of the settlement.

KEY WORDS

- *subsoil behaviour*
- *finite element method*
- *settlement*
- *consolidation*

1 INTRODUCTION

Loads which are transferred from a superstructure through a foundation to soil cause stresses in subsoil, which then result in deflections. The construction of high-rise buildings in Bratislava is presently increasing on a large scale, and the subsoil is overstressed. In this situation it is necessary to develop new progressive calculations of the structure – soil interaction and the settlement of a structure.

The final value of settlement is not immediately achieved after additional loading. The settlement depends on the properties and stress state of the soil and should achieve its final value over a certain time period. The final value of the settlement of frictional soil is obtained very quickly, even during the period of construction. Nevertheless, the process of clay settlement is considerably slower; it takes several months or years. The final value of the settlement of impermeable clay is often achieved over a long time and depends on the properties of the layers in the ground. Such settlement continues for several decades. This is especially true in saturated clays because their hydraulic conductivity is extremely low, and this causes the water to take an exceptionally long time to drain off the soil.

Usually consolidated material is material that has never experienced a load greater than the current load. Over-consolidated material is material that has previously experienced a load greater than the current load. Over-consolidated material is stronger than the same normally consolidated material and is less compressible. The engineer, therefore, needs to know the magnitude of the maximum past pressure, which can be obtained from the laboratory test results.

2 REASONS FOR SETTLEMENT

Soil consists of a multiphase aggregation of solid particles, water, and air. This fundamental composition gives rise to unique engineering properties, and the description of the mechanical behaviour of soils requires some of the most sophisticated principles of engineering mechanics. The terms "multiphase" and "aggregation" both imply unique properties. As a multiphase material, soil exhibits mechanical properties that show the combined attributes of solids, liquids, and

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gases. Individual soil particles behave as solids and show relatively little deformation when subjected to either normal or shearing stresses. Water behaves as a liquid, exhibiting little deformation under normal stresses, but deforming greatly when subjected to shear. Being a viscous liquid, however, water exhibits a shear strain rate that is proportional to the shearing stress. Air in the soil behaves as a gas, showing appreciable deformation under both normal and shear stresses.

When dry soil is subjected to a compressive normal stress, the volume decreases nonlinearly; that is, the more the soil is compressed, the less compressible the mass becomes. Thus, the more tightly packed the particulate mass becomes, the more it resists compression. The process, however, is only partially reversible, and when the compressive stress is removed, the soil does not expand back to its initial state. When this dry particulate mass is subjected to shear stress, an especially interesting behaviour owing to the particulate nature of the results of the soil solids, if the soil is initially dense (tightly packed), the mass will expand because the particles must roll up and over each other in order for shear deformation to occur. The specific initial density (the critical density) at which the material will display zero volume changes when subjected to shear stress. The term "dilatancy" has been applied to the relationship between shear stress and volume change in particulate materials. Soil is capable of resisting shear stress up to a certain maximum value. Beyond this value, however, the material undergoes large, uncontrolled shear deformation.

The other limiting case is saturated soil, that is, a soil whose voids are entirely filled with water. When such a mass is initially loose and is subjected to compressive normal stress, it tends to decrease in volume; however, in order for this decrease in volume to occur, the water must be squeezed from the soil pores. Because water exhibits a viscous resistance to flow in the microscopic pores of fine-grained soils, this process can require considerable time, during which the pore water is under increased pressure. This excess pore pressure is at a minimum near the drainage face of the soil mass and at a maximum near the centre of the soil sample.

We can distinguish several types of settlements (fig. 2.1):

- **Immediate settlement** – is in progress directly after loading without a change in volume.
- **Consolidated settlement** – is caused by an increase in effective pressure stresses. It occurs as a result of the consolidation of the soil, which is related to draining the water from the pores.
- **Long - term settlement** – continues when the hydraulic consolidation is finished. It refers to the plastic deformations of a structure.

The deformation of soil is a time-dependent process. The following processes can be regarded as rheology processes: When a change in stress occurs, the effective stress within the soil does not change

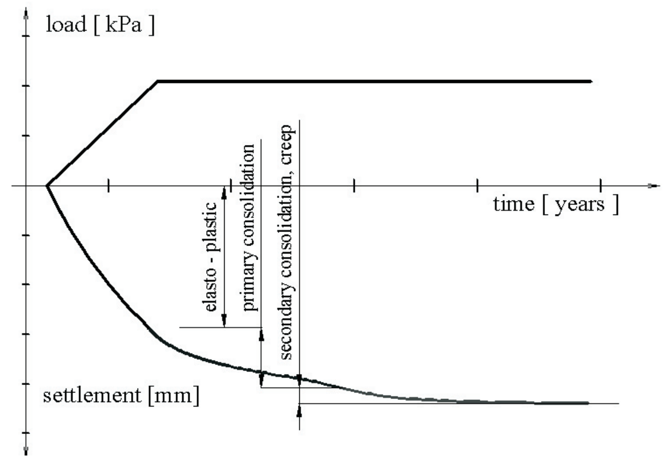

Fig. 2.1 *Settlement of a building*

compared to its pre-construction value. The total stress in the soil is increased by the change in stress due to construction (e.g. building a structure on a ground surface), and the pore pressure increases by the same amount. When this occurs in a soil which is saturated with water, water will be squeezed out of the soil. The speed of water movement in the ground is related to Darcy's law. With the gradual draining off of the water, the neutral stresses are decreased. The load shifts to contacts between the solid grains – the effective stress is rising. The deformation is finished when the neutral stresses decrease to the level of the hydrostatic pressure, and the whole additional load shifts to solid particles.

Consolidation is regarded as a rheological process of the gradual adaptation of the pore volume and the structural skeleton of soil in an active loading state, which is connected with the squeezing out of the excess water from the pores. This is the process of the progressive hardening of the soil.

Such types of soil are consolidated when the whole load is spread to solid grains as an effective stress. In partly or unconsolidated soils, the load is split, and some portion of this load is transmitted to the pore water. These stresses are denoted as neutral stresses.

The correct definition of the consolidation process has important practical significance. It informs us about the development of deformation in subsoil, the splitting of effective and neutral stresses and the increase in the shear strength of the soil over the course of time.

Whenever a foundation is loaded, a pressure (stress) increase occurs in the soil underlying the raft. Actually, the pressure spreads laterally to a certain degree as well. The intensity of the pressure decreases with the depth until it eventually becomes too small and is of little concern. It is the increase in pressure that causes settlement to

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occur in the soil below the foundation. The zone where the pressure increases is significant with respect to settlement, which varies with the width of the foundation. In clay, the zone is also influenced by the intensity of the effective overburden pressure (the pressure due to the effective weight of the soil lying above the point in question) with respect to the foundation stress.

Differential equation of primary consolidation

Each change in stress causes a new deformation in the subsoil. According to the presented mathematical fundamentals, the consolidation presents a case of unsteady water movement in the ground pores. The transient flowing of water is generally represented by a continuity equation [6]:

$$\frac{\partial G_W}{\partial t} = \left[\frac{\partial}{\partial x}(\gamma_w v_x) + \frac{\partial}{\partial y}(\gamma_w v_y) + \frac{\partial}{\partial z}(\gamma_w v_z) \right] \qquad (2.1)$$

γ_w – unit weight of water [kN/m^3]
v_x, v_y, v_z – velocity vector of water [m/s]
$\frac{\partial G_W}{\partial t}$ – Water quantity in the term, if the volume of the pore is constant

Based on a study of the principal law of water filtration through sand filters, Darcy derived a fundamental law of water movement in a porous environment:

$$v = k.i \qquad (2.2)$$

v – Permeability velocity [m/s]
k – Factor of permeability
i – Hydraulic gradient

When we substitute Darcy's law into the continuity equation [6]:

$$\frac{\partial G_W}{\partial t} = \left[\frac{\partial}{\partial x}\left(\gamma_w k_x \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(\gamma_w k_y \frac{\partial h}{\partial y}\right) + \frac{\partial}{\partial z}\left(\gamma_w k_z \frac{\partial h}{\partial z}\right) \right] \qquad (2.3)$$

The left part of the equation introduces changes in the water weight in a certain volume at time *dt*. When we express these changes in relation to the reacting load and adapt the equation to the boundary conditions, we can derive a differential equation of a one-dimensional consolidation [9]:

$$\frac{\partial u}{\partial t} = \frac{kE_0}{\gamma_w} \frac{\partial^2 u}{\partial z^2} \qquad (2.4)$$

u – Neutral stress [kPa]
E_0 – Oedometric modulus [kPa]

We suppose that it is a consolidation of a horizontal compressible layer, which is uniformly loaded, and that the water from the sample will be drained off in the shortest possible way, i.e., only in a vertical direction. Total stress is induced by the external loading, and it is constant over time. The value of the consolidation can be investigated by many different methods.

$$c_v = \frac{kE_0}{\gamma_w} \qquad (2.5)$$

c_v – Factor of consolidation [m^{2}s^{-1}]

In cylindrical co-ordinates [9]:

$$\frac{\partial u}{\partial t} = c_v \left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2} + \frac{\partial^2 u}{\partial z^2} \right) \qquad (2.6)$$

When the convection is radial and the perpendicular component is zero $\delta^2 u / \delta z^2 = 0$, we can write a differential equation of a radial primary consolidation in this form [9]:

$$\frac{\partial u}{\partial t} = c_v \left(\frac{\partial^2 u}{\partial r^2} + \frac{1}{r}\frac{\partial u}{\partial r} \right) \qquad (2.7)$$

In this case it is necessary to describe the value of the consolidation factor c_v in consideration of the direction of the water movement and conditions of the subsoil's deformation. Radial flow occurred when the vertical drains were constructed. Those drains accelerate the consolidation process of weakly saturated subsoil layers. Usually they are constructed as drilled or vibrated stone columns.
Equation (2.4) could be solved using several methods. One is the Bernoulli method of a series of expansions in an infinite Fourier algorithm, but the solution is suitable only for a simple boundary condition [6]. Another solution, which is presented in this article, is a solution using the finite element method.

Multidisciplinary tasks in FEM

Coupled-field analyses are useful for solving problems where the coupled interaction of phenomena from various disciplines of physical science is significant. Several examples of this include: an electric field interacting with a magnetic field, a magnetic field producing structural forces, a temperature field influencing fluid flow, a temperature field giving rise to thermal strains and the usual influence of temperature-dependent material properties. The latter two examples can be modelled with most non-coupled-field elements, as well as with coupled-field elements [1].
There are basically two methods of coupling which are distinguished by the formulation of finite element techniques used to develop

the matrix equations. These are illustrated here with two types of degrees of freedom ($\{x_1\}$, $\{x_2\}$).
Most physical tasks deal with one way coupling, which means, for example, the effect of the thermal field will cause deformations (displacements, rotations); however, displacements do not cause modifications of a thermal field.

2.3.1 Strong (simultaneous, full) coupling

The coupled effect is accounted for by the presence of the off-diagonal sub matrices [K12] and [K21]. This method provides for a coupled response in the solution after the first iteration.
Where the matrix equation is of the form [1]:

$$\begin{pmatrix} [K_{11}] & [K_{12}] \\ [K_{21}] & [K_{22}] \end{pmatrix} \begin{bmatrix} \{x_1\} \\ \{x_2\} \end{bmatrix} = \begin{bmatrix} \{F_1\} \\ \{F_2\} \end{bmatrix} \tag{2.8}$$

One of the tasks where a strong formulation is used is a piezoelectric analysis. In this case the values of the deformations are dependent on the values of the electrical current. The matrix equation [1]:

$$\begin{pmatrix} [M] & 0 \\ 0 & 0 \end{pmatrix} \begin{bmatrix} \left\{\frac{\partial^2 a}{\partial t^2}\right\} \\ \left\{\frac{\partial^2 V}{\partial t^2}\right\} \end{bmatrix} + \begin{pmatrix} [C] & 0 \\ 0 & 0 \end{pmatrix} \begin{bmatrix} \left\{\frac{\partial a}{\partial t}\right\} \\ \left\{\frac{\partial V}{\partial t}\right\} \end{bmatrix} + \begin{pmatrix} [K] & [K_Z] \\ [K_Z]^T & [K_d] \end{pmatrix} \begin{bmatrix} \{a\} \\ \{V\} \end{bmatrix} = \begin{bmatrix} \{F\} \\ \{L\} \end{bmatrix} \tag{2.9}$$

2.3.2 Weak (sequential) coupling

The coupled effect is accounted for in the dependency of [K11] and {F1} on {X2} as well as [K22] and {F2} on {X1}. At least two iterations are required to achieve a coupled response.
Where the coupling in the matrix equation is shown in the most general form [1]:

$$\begin{pmatrix} [K_{11}]_{(\{X_1\},\{X_2\})} & 0 \\ 0 & [K_{22}]_{(\{X_1\},\{X_2\})} \end{pmatrix} \begin{pmatrix} \{X_1\} \\ \{X_2\} \end{pmatrix} = \begin{pmatrix} \{F_1\}_{(\{X_1\},\{X_2\})} \\ \{F_2\}_{(\{X_1\},\{X_2\})} \end{pmatrix} \tag{2.10}$$

One of the tasks where a weak formulation is used is a thermal-structural analysis. In this case the values of the deformations are dependent on the value of the thermal field. The matrix equation [1]:

$$\begin{pmatrix} [M] & 0 \\ 0 & 0 \end{pmatrix} \begin{bmatrix} \left\{\frac{\partial^2 a}{\partial t^2}\right\} \\ \left\{\frac{\partial^2 T}{\partial t^2}\right\} \end{bmatrix} + \begin{pmatrix} [C] & 0 \\ 0 & [C_t] \end{pmatrix} \begin{bmatrix} \left\{\frac{\partial a}{\partial t}\right\} \\ \left\{\frac{\partial T}{\partial t}\right\} \end{bmatrix} + \begin{pmatrix} [K] & 0 \\ 0 & [K_t] \end{pmatrix} \begin{bmatrix} \{a\} \\ \{T\} \end{bmatrix} = \begin{bmatrix} \{F\} \\ \{Q\} \end{bmatrix} \tag{2.11}$$

2.4 Transaction between transient heat conduction and primary consolidation

When we look closely at a differential thermal analysis equation and compare that equation with a differential equation of primary consolidation, we can find a connection between these two different physical phenomena. This conditional transaction results from the same physical background as the differential diffusion equation.
Differential equation of transient heat conduction in an isotropic body [11]:

$$\frac{\partial}{\partial x}\left(\lambda \frac{\partial T}{\partial x}\right) = \frac{\partial T}{\partial t} \tag{2.12}$$

Differential equation of primary consolidation in an isotropic body [10]:

$$\frac{\partial}{\partial z}\left(c_v \frac{\partial u}{\partial z}\right) = \frac{\partial u}{\partial t} \tag{2.13}$$

The primary consolidation could be characterized as a hydraulic flow in a porous media.
The premises and substitutions in the solution are:

1. $T = u$ temperature = pore pressure
2. $\lambda = c_v$ coefficient of thermal conductivity = coefficient of primary consolidation
3. Other premises could be characterized by a theory of elasticity
   3.1 Structural stress - effective pressure

$$\sigma_{ef} = E\varepsilon_{ef} = E\frac{\Delta h}{h} \tag{2.14}$$

   3.2 Thermal stress – pore pressure

$$u = E\varepsilon_u = E\alpha_t \Delta t \tag{2.15}$$

   3.3 Transaction condition – total pressure

$$\sigma = E\left(\varepsilon_{ef} + \varepsilon_u\right) \tag{2.16}$$

4. Boundary condition - coefficient of transaction α_t
   4.1 Starting point: $t_1 = u_1 = \chi\sigma$, $t_{ref} = 0$, χ = ratio of saturation

$$\sigma_1 = E\left(\varepsilon_{ef1} + \varepsilon_{u1}\right) = E\alpha_t\left(t_1 - t_{ref}\right) = E\alpha_t\chi\sigma_1 \tag{2.17}$$

$$\alpha_t = \frac{1}{E\chi} \tag{2.18}$$

   4.2 Ending point: $t_2 = u_2 = 0$, $t_{ref} = 0$, χ = ratio of saturation

$$u_2 = E\varepsilon_{u2} = E\alpha_t\left(t_2 - t_{ref}\right) = E\alpha_t 0 \tag{2.19}$$

$$\alpha_t = 0 \tag{2.20}$$

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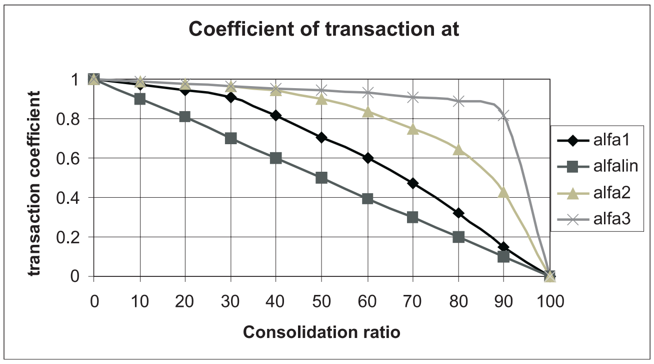

Fig. 2.4.2 *Coefficient of transaction α_t*

The correct determination of the coefficient of transaction α_t is important, because this coefficient estimates the ratio of the consolidation's settlement. Few behavioural functions have been characterized. With the oedometric experimental tests we verified those behavioural functions. For the clay investigated, the most optimal function could be declared as function α_3.

3 EXPERIMENTAL METHODS

3.1 Oedometer test

A well-known laboratory test is the oedometer test. It is a model test of one-dimensional consolidation when the subsoil below the foundation is pressed only in a vertical direction (the lateral strains are considered negligible). These assumptions are acceptable in most cases of foundations, and they approximate a real condition a fortiori; the foundation area is larger, and the relative thickness of the compressible subsoil layer is smaller. This method assumes that the consolidation occurs in one dimension only. Laboratory data are used to generate a graph of the settlement versus effective stress where the effective stress axis is on a logarithmic scale.

We prepared a specimen test with a diameter of 40 mm and a depth of 30 mm from an undisturbed soil sample. Thereafter, we inserted it into the rigid metallic cylinder of an oedometer, where it was gradually loaded between the rigid, porous plates in successive steps.

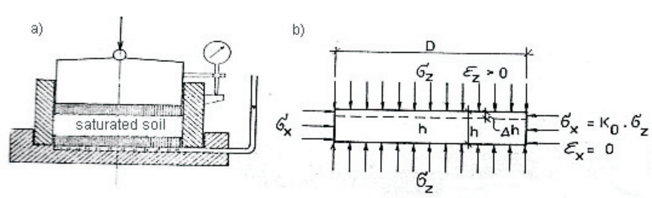

Fig. 3.1.1 – *Oedometer test*

The relation of the final values of the deformations in particular load levels can be presented graphically as a *soil compressibility curve*, which is used for the principal expression of soil deformation properties.

In order to predict the amount of settlement that is likely to occur in a clay stratum, an engineer must have knowledge of the past history and engineering properties of the clay. This can be achieved by retrieving an "undisturbed" sample of the clay and testing it to measure its consolidation characteristics.

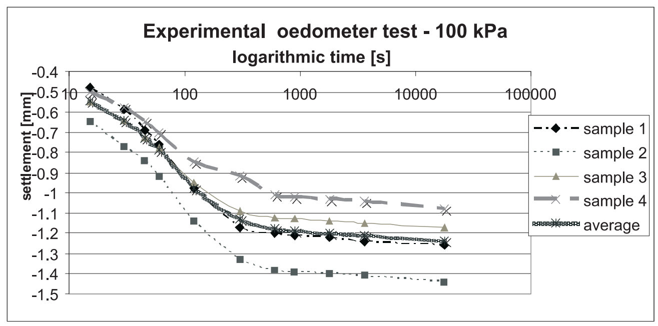

Fig. 3.1.2 *Oedometer test results, Soil - F6 - c_v = 1.48 10^6 m^2/s*

The results of the laboratory testing program are presented on a series of semi-log plots. One of these plots shows the decrease in the void ratio or strain (vertical axis) in relationship to the increased pressure of the load. From this data the engineer can obtain the significant engineering properties of the soil, which are then used to predict the magnitude of the settlement.

The results of the laboratory testing also provide information on the rate of consolidation, which provides a basis for predicting how quickly the clay will consolidate (i.e. the end of the primary consolidation). An engineer will often conduct the tests in such a manner so that the primary consolidation and secondary consolidation are separated. In this case, the calculations are conducted for each component and summed to provide a prediction of the total long-term settlement. At other times, the tests are conducted and include the results of both the primary and a component of the secondary consolidation.

The determination of soil compressibility by using an oedometer test is a basic method for obtaining the quantitative characteristics of soil deformation properties.

3.2 Triaxial test

During a triaxial test a cylindrical sample is put between rigid plates and covered with a thin, elastic watertight membrane, which is tied to

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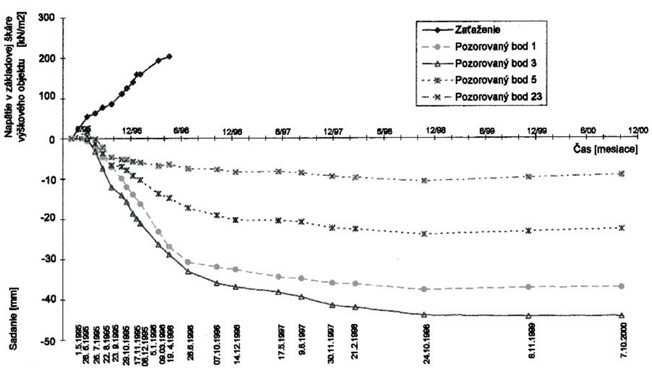
those plates. In the first phase of the examination, the pressure state is created through the fluid. The pressure is equal in all directions $\sigma_1 = \sigma_2 = \sigma_3$. In the second phase, an additional load is applied on top of the plate, which initiates the cylindrical symmetrical state of the stress $\sigma_1 > \sigma_2 = \sigma_3$. The sample is thus deformed in the vertical and horizontal directions. In the test, it is possible to measure the neutral stresses; at the same time we can also monitor the changes in the volume of the sample.
With the raised difference between σ_1-σ_3 at a constant pressure σ_3, the shear force might cause damage to the sample, which would not be possible in an oedometer.
This examination will then investigate the real state of stress in the corresponding point of the subsoil.

3.3 In-situ tests – settlement control

According to Euro Code EN 1997, all buildings categorized as difficult should be observed after finishing the construction. These results are most appreciated, because we can predict from those results the real behaviour of the subsoil. A few results from the settlement measurements of two high - rise buildings currently being constructed in Bratislava are shown in the following figures.



Graph 3.3.1 *Measurements of VUB building settlement according to [5]*

Geological investigation

To obtain information about soil conditions below a surface, some form of subsurface exploration is required. Then the foundation design can continue successfully.
The north-western edge of the Danube lowland is characterized by the specific geological layout of Post-Tertiary and Neogene sediments. Post-Tertiary soil layers represent a gravel of various granularities (G2, GP) with a sand region. Neogene deposits inside the region of Bratislava are situated according to different site investigations from a depth of 11.0 to 15.0 m below terrain level. On the basis of the results of the experimental tests, the Neogene subsoil can be categorized as F8 CH or high plastic clay soils with a limit of plasticity at 100kPa.
The ground water is approximately 4-5m below the terrain. This ground water is in a hydraulic relation to the Danube at water level.

4 METHODS OF GROUND IMPROVEMENT

In spread stiff raft foundations of high-rise buildings with over 30 floors above ground and few underground floors, contact stresses could rise in the most loaded places around the central cores of stiffness to a value of around 300 kPa. Those contact stresses could vary depending on the shape and materials used in the proposed structure.
The bearing capacity of soil is the maximum average contact pressure between the foundation and the soil which will not produce shear failure in the soil. The ultimate bearing capacity is the theoretical maximum pressure which can be supported without failure.
On soft soil sites, large settlements may occur under loaded foundations without actual shear failure occurring; in such cases, the allowable bearing capacity is based on the maximum allowable settlement.
Our preliminary calculations according to STN 73 1001 shows that the soil-bearing capacity is around a value of Rd = 850 kPa, but the immediate settlement of the subsoil has been calculated to a value of s_1= 85 mm. However, this deflection could increase by 10 – 20 % due to rheological processes in the subsoil and due to the influence of the primary consolidation.
The significant parameters of the settlement process are the mechanical characteristics of the soil, the level of the groundwater, permeability and the order of the subsoil layers.
The boundary values of the settlement in framed concrete buildings are, according to the valid standard STN 731001, s_{lim}=60 mm. An improvement in subsoil has been considered to decrease deformations. As the most suitable method to increase the mechanical properties of the subsoil, it has been suggested to improve the subsoil by stone columns with a diameter of Ř 1000mm. As an optimal basic raster a square net of 3x3 m has been selected. The pile length has been optimalised to 6m.
This type of ground improvement is particularly suitable for saturated clays. Stone columns increase the value of the soil's resistance and decrease the time for the primary consolidation and final settlement of the subsoil.
The natural process of the consolidation of compressible soils

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can be accelerated by improving the drainage conditions within the soils, so as to assist in the outward migration of the water. If such soils are loaded on their surface by a temporary surcharge or permanent structures such as an embankment, the water pressure in the underlying soil will increase. If the water cannot escape sufficiently quickly, a dangerous stability condition could arise.

Alternatively, this condition can be anticipated and exploited to accelerate the drainage of the compressible soils by installing vertical columns of sand or strips of preformed permeable material within the ground to enable the excess water to escape more rapidly. These columns are called sand drains, vertical drains or wick drains, and are frequently considered for improving the strength of the soil.

There are a few construction methods for the construction of stone columns. There are some world-renowned companies that use special methods of constructing foundations, such as *KELLER* or *FRANKI*.

Sand drains or stone columns can be formed by one of several methods. In small installations, the borings can be made using bored pile equipment and a casing to line the hole. For larger schemes the holes can be formed by driving a steel tube fitted with an expendable shoe to displace the soil away from the tube, or a special casing can be jetted down to the required depth.

At the groundwater surface a horizontal blanket is placed to connect the sand drain heads so that the expelled water can be drained away.

The soil to be strengthened by these methods requires an investigation of its strength characteristics before and after draining the sand. Piezometers should be installed in the ground to measure the change in water pressure as the drainage proceeds. The extent of sand drainage required can be established by calculation, and if the scheme is large, a small pilot test scheme will provide valuable data on the drain size centres.

The permeability of the soil should be measured in situ to confirm the feasibility or economic desirability of a vertical drain installation, because experience has shown that values of permeability coefficients obtained from laboratory tests can be wildly misleading.

Construction methods of stone columns with gravel fill include:

- Vibro-compaction: a classical sand compaction process which relies on the rearrangement of cohesionless soil particles into a denser state under the action of a specially designed vibrator.
- Vibro-replacement: soil improvement is achieved by reinforcing weak soils with densely compacted granular columns. Silt and soft cohesive soils are not suited to vibro compaction because of their cohesive properties, which prevent particle re-alignment under vibration.
- Bored stone columns

5 FINITE ELEMENT ANALYSES WITH ANSYS SOFTWARE

5.1 3D analysis of Oedometric test

The oedometer test was modelled in 3D as well. We used the symmetry of the model by inputting the boundary support conditions in a beneficial form. The model is defined as a quarter cylinder with a radius of 40mm and a depth of 30mm. The size of the mesh is 1mm in axis z and 5mm in axis x, y. The model was created through the use of a SOLID 5 element, which also allows solving coupled field analyses of flow in porous media and the structural analysis.

Boundary conditions:

1. Restraint of displacement ux, uy, uz,
2. Effective stress 100 kPa
3. Neutral stress 100 kPa
4. Definition of flow boundary
5. Time discretization of solution

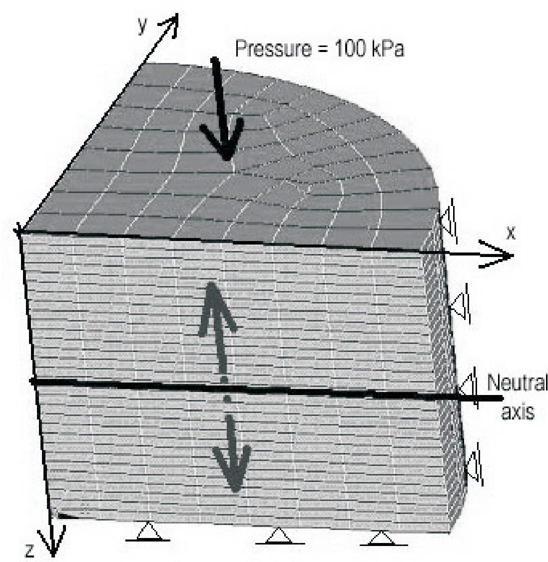

Fig. 5.1.1 *3D stress state – Oedometer test*

5.2 FEM simulation of consolidation settlement in saturated clays

The behaviour of uniformly loaded sub-soil can be studied on a numerical model, see Fig.5.2.1, using the ANSYS software. The sub-soil investigated was solved in a plane strain state, modelled from 4-node PLANE 13 elements.

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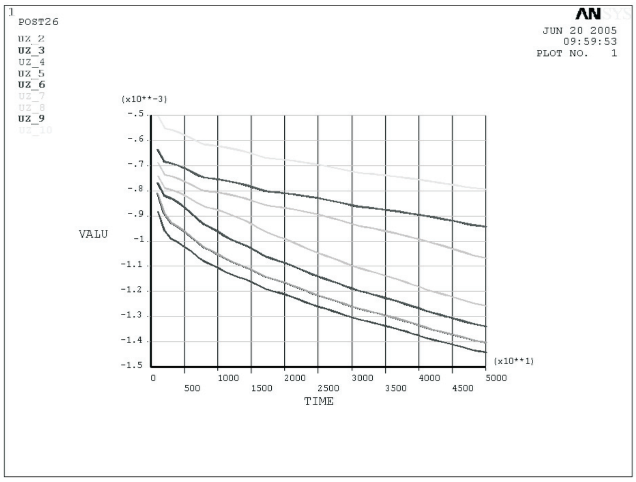

Fig. 5.1.2 *Vertical displacement over time*

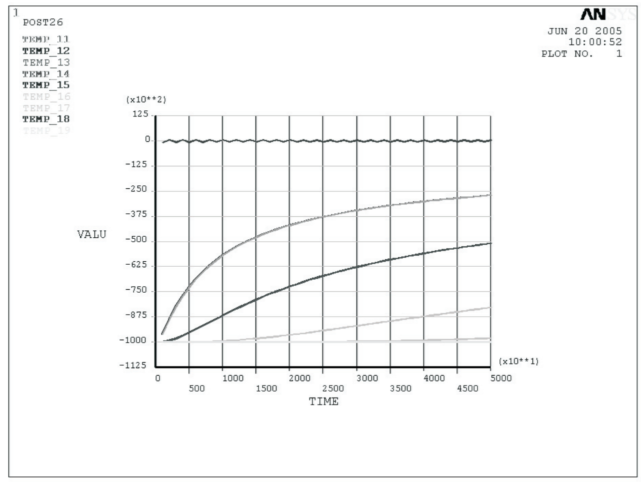

Fig. 5.1.3 *Neutral stresses over time*

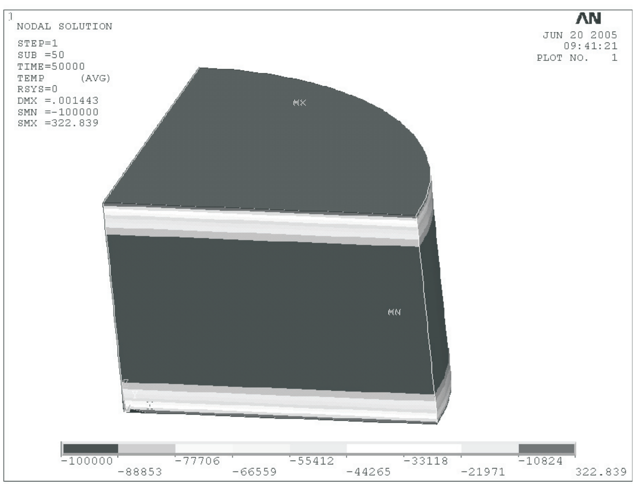

Fig. 5.1.4 *Neutral stress in time t = 50.10^3 s*

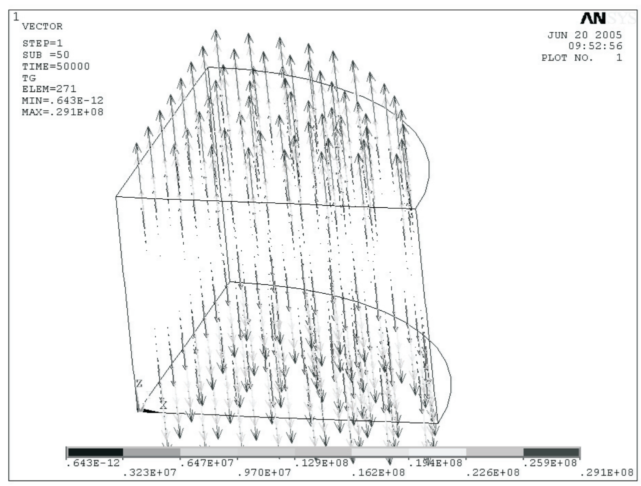

Fig. 5.1.5 *Water flow gradient*

The region investigated with dimensions of 30 x 40 m was divided into elements of the sizes 0.5 x 0.5 m. We achieved the symmetry of the model by inputting the boundary support conditions.

The interaction between the foundation structure and the subgrade was modeled using CONTACT12 contact elements.

The material characteristics of the subsoil were defined according to this particular model.

The linear elastic model of the homogeneous isotropic material is defined by the oedometer modulus of elasticity E_O = 8 MPa and Poisson's Ratio ν = 0.37. Additional characteristics defining the primary consolidation that were considered in the calculations are: the consolidation factor, permeability and the coefficient of the transaction that resulted from the experimental tests.

Several high-rise structures, both constructed and proposed, were analyzed. In this paper a solution of the subsoil settlement of the VUB high – rise office building is presented. The process of the settlement of this shallow slab, which was experimentally measured, was taken from reference [5]. The settlement analysis was made using the method which has been described above.

The settlement calculations were made using a 2D model, see *Fig.5.2.1*. The contact stress in the foundation's bottom was considered to be a constant value of 200 kPa. A time solution was made using direct integration over time. Several points were selected and analyzed in the foundation plate as well as in the subsoil. In Fig.

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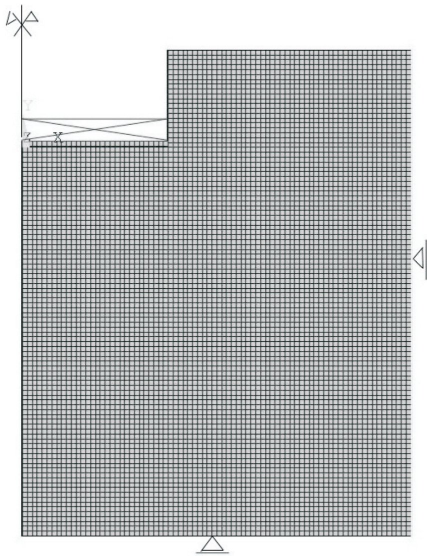

Fig. 5.2.1 *FEM sub-soil model*

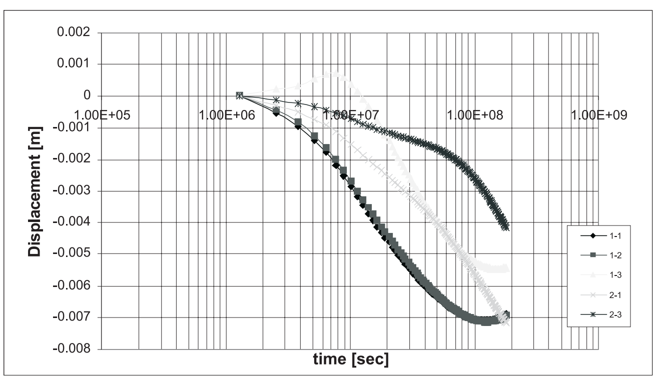

Fig. 5.2.2 *Calculated time dependent settlement of a high-rise building in specific nodes*

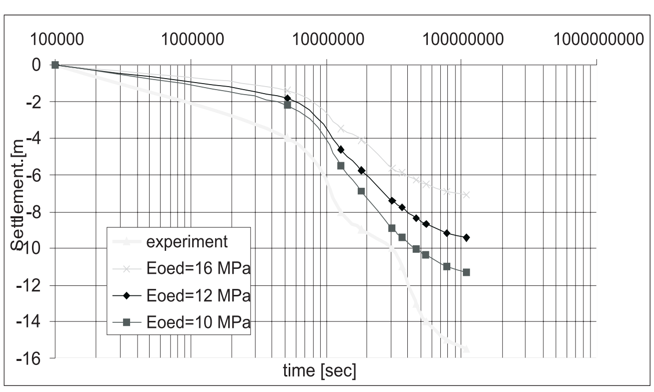

Fig. 5.2.3 *Calculated maximal time dependent settlement of high-rise building*

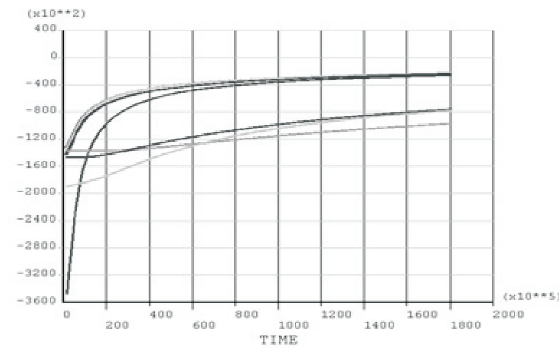

Fig. 5.2.4 *Calculated values of pore pressure in specific nodes (P_8 to P_13)*

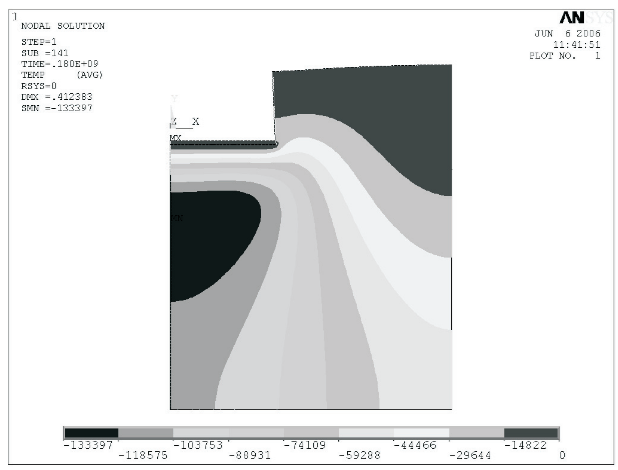

Fig. 5.2.5 *Isochors of pore pressure in sub-soil in time t = 6 years*

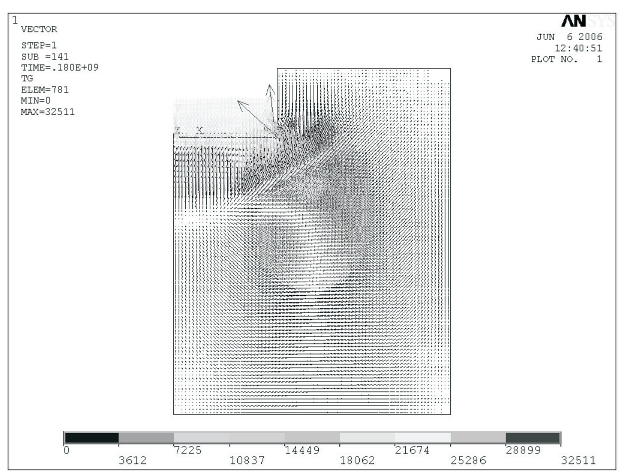

Fig. 5.2.6 *Vectors showed water spread out from soil pores*

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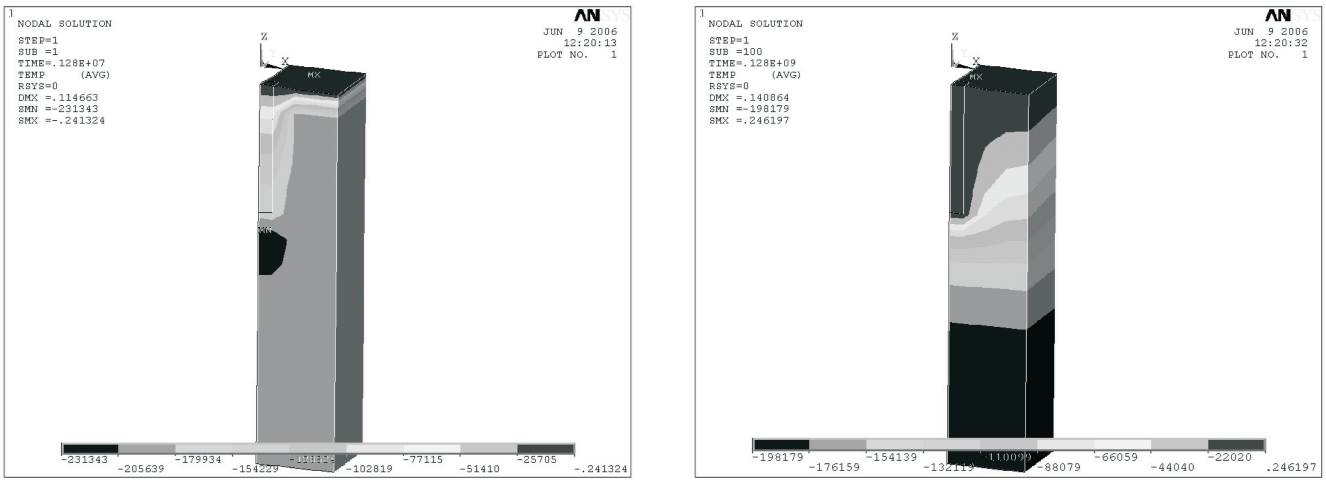

Fig 5.3.1a,b *Isochors of pore pressure over time start t_0 = 11 days and end t_2 = 1389 days*

5.2.2, nodal vertical displacements over time are presented. In Fig. 5.2.3, the values of the consolidation settlement are presented over time t = 6 years. A parametric study with various elastic modul is also presented. The results are compared with the experimental measurements. In Fig. 5.2.4, the values of the pore pressure in the specific nodes are plotted.

5.3 FEM modelling of ground improvement – stone columns

The behaviour of uniformly loaded sub-soil can be studied on defined numerical models using the ANSYS software. The sub-soil investigated was solved in a plane strain state, modelled from PLANE 13 4-node elements, and as a full 3D model modelled from SOLID5 8-node elements.

We were advantaged by the symmetry of the model, and through the input of the boundary support conditions, we decreased the required computer time for solving the analysis.

The material characteristics of the sub-soil were defined according to this particular model:

- The linear elastic model of homogeneous isotropic material is defined by the oedometer modulus of elasticity E_O = 12 MPa and Poisson's Ratio ν = 0.37. Additional characteristics defining the primary consolidation that were considered in the calculations are: the consolidation factor, permeability and coefficient of the transaction that resulted from the experimental tests.

The behaviour of subsoil improved by stone columns loaded with 280 kPa contact stress was studied on a simplified 3D model of a part of one pile with the related subsoil. The boundary condition of the primary consolidation, i.e., the gradual draining of the water from the pores, was defined just on the top area of the model. In the same area the contact pressure was defined. *Figs.5.3.1a,b* show the isochors of the pore pressure start time t_0 and end time t_2 of the investigation. A noticeable decrease of pore pressure is recognized. This change follows the theory of primary consolidation as the cause of the creep of the subsoil, which means additional deformations.

The second model set up for an examination of subsoil behaviour improved by stone columns was a 2D model solved in a plane strain state, which was modelled from PLANE 13 4-node elements. The region investigated with dimensions of 40 x 60 m was divided into elements of the sizes 0.5 x 0.5 m. The excavation pit for the

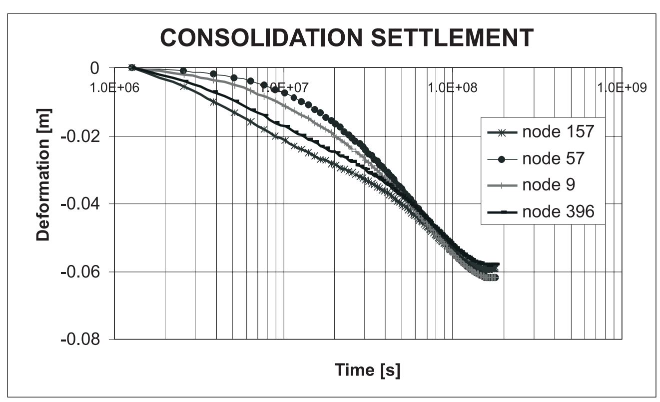

Fig 5.3.2 *Values of calculated consolidation settlement in a logarithmical time scale.*

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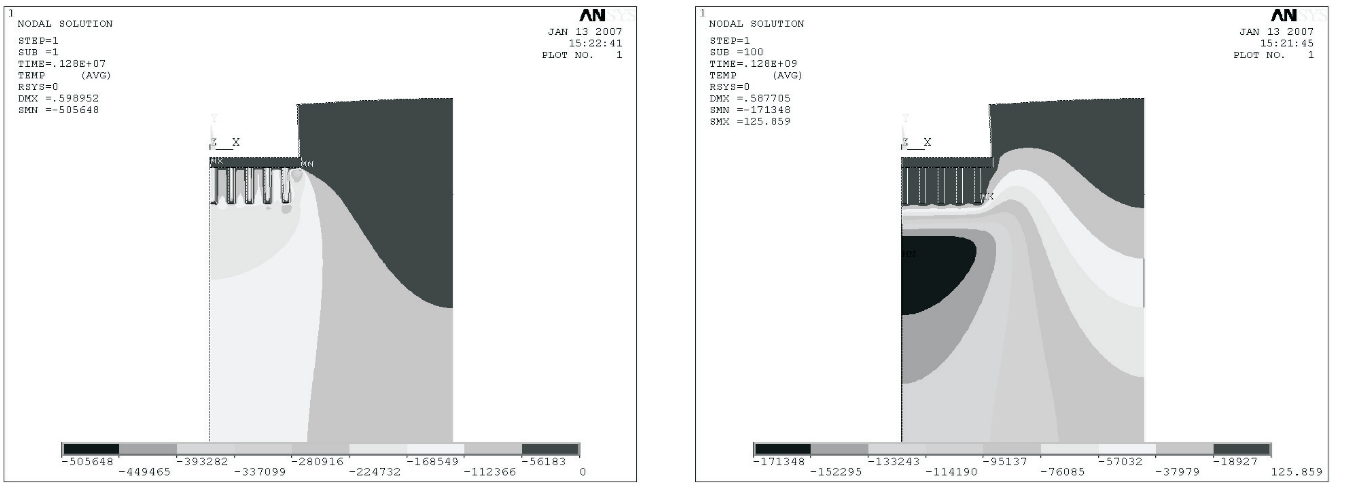

Fig 5.3.3 *a,b Isochors of pore pressure in time start t_0 = 11 days and end t_2 = 1389 days*

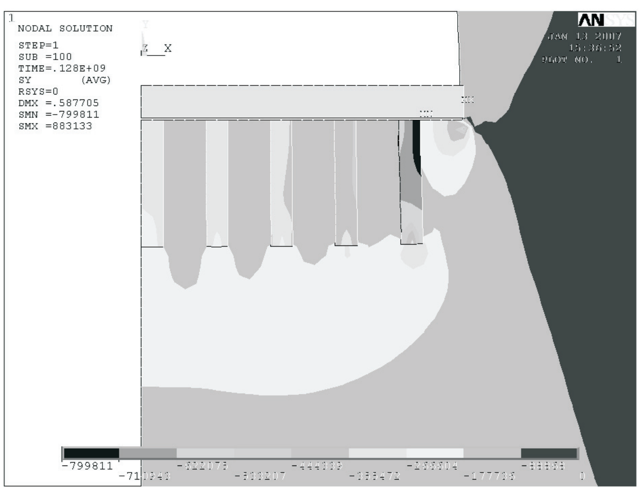

Fig 5.3.4 *Vertical pressure σ_z in time t_2 end of investigation*

foundation was considered to have a size 10 m deep and 15 m long with a foundation raft 2 m deep.

6 CONCLUSION

In this paper the possibility of the simulation of the real time behaviour of saturated sub-soil after a construction was finished by using the finite element method was presented.
For debugging the mathematical calculations and a proper understanding of the effects of primary consolidation, the actual experimental results from the ground investigation were used. The actual building settlement and calculated results were introduced.
As shown in the figures, the vertical strain and neutral pressure were analyzed over time. A comparison of the experimental measurements and FEA solution was introduced. The FEA results are consistent with the experimental oedometer test. The issue of small discrepancies is at the beginning of the calculations. That happened due to the larger time steps of the Newton – Raphson solution; however, the solution converged to the results of the experimental measurement very well. Significant results were obtained concerning the vertical deformations over time. The time-dependent settlement of the foundations of buildings can be calculated with the proper application of this FEM model to geological subsoil conditions.
A calculation of the primary consolidation of the improved subsoil was introduced. The results from the set up models showed a decrease in the required time for primary consolidation as well as reducing the deformation zone and total settlement. Stone columns concentrate the stresses and loading of the clayed subsoil in a beneficial form.

ACKNOWLEDGMENT

This work was supported by VEGA 1/2147/05 fund. Additionally, a grateful acknowledgment is also due to the significant support of assoc. prof. Ing. Norbert Jendželovský, PhD., for his valuable advice and proposals.

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REFERENCES

[1] **Kohnke, P. (2005)** *ANSYS*, User´s Manual for Revision 8.1, Vol. I-IV, Swanson Analysis System, Inc.
[2] **Attkinson, J. (2004)** Consolidation – teaching paper, City University London, www.fbe.uwe.ac.uk
[3] **Burland, J.B., Roscoe, K.H. (1969)** Local Strains and Pore Pressures in a Normally Consolidated Clay Layer During One-Dimensional Consolidation, In: Geotechnique 19, No.3
[4] **Georgiadis, K., Potts, D. M., Zdravkovic, L. (2003)** The Influence of Partial Soil Saturation on Pile Behaviour, In: Geotechnique 53, No.1, 2003
[5] **Hulla, J., Turček, P. (1998)** Zakladanie Stavieb (Building Foundations), Jaga group, Bratislava, 310 pp. (in Slovak)
[6] **Jesenák, J. (1982)** Mechanika zemín (Geomechanics), SVŠT Bratislava, 423 pp. (in Slovak)
[7] **Smith, G.C. (2003)** Numerical Analysis of Infiltration into Partially Saturated Soil Slopes, PhD – Thesis, Imperial College London, 350 pp.
[8] **Šejnoha, J. et al. (2001)** Structure - Subsoil Interaction in View of Transport Processes in Porous Media, CTU Reports 1/2001, Vol. 5., Prague, 81 pp.
[9] **Šuklie, L. (1969)** Rheological Aspects of Soil Mechanics, Wiley- Interscience, 522 pp., SBN 47183550 1
[10] **Terzaghi, K. (1943)** Theoretical Soil Mechanics, John Wiley & Sons, Ltd, New York, 311 pp.
[11] **Zienkiewicz, O.C. (1977)** The Finite Element Method, Mc Graw – Hill, London, 551 pp.