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PIT PROTECTION FOR DEEP EXCAVATIONS - MODERN METHODS OF CONSTRUCTION AND DESIGN

Povzetek

V urbanih območjih, bolj je potreba po gradnji večjega števila podzemnih etažah. Gradnja zaščite pit globokih izkopov v urbanih razmerah, poleg obstoječih objektov v zahtevnih geotehničnih pogojev so zapleteni, drago in zamudno. Zato se v zadnjih letih uporabljajo posebne metode za izvedbo del, kot je sistem "od zgoraj navzdol". V tem prispevku je predstavljen različne metode projektiranja konstrukcij za temeljenje jame.

Ključne besede: fundacija pit, "top down" konstrukcije, numerične metode izračuna.

Summary

In urban areas, increasingly there is a need for building of a higher number of underground floors. Construction of pit protection for deep excavations in urban conditions, next to existing objects in the complicated geotechnical conditions are complex, expensive and time-consuming. Therefore, in recent years specific methods of execution of works such as "top down" system has been applied. In this paper the different methods of design of protection structures for foundation pits is presented.

Keywords: foundation pit,"top down" construction, numerical calculation methods.

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1. INTRDUCTION

When performing deep excavations, to ensure the stability of the sides of the excavation, there is a need for protective structures of the foundation pit. This is especially true for urban applications where space is limited around the pit and where adjacent buildings are located next to the excavation pit.

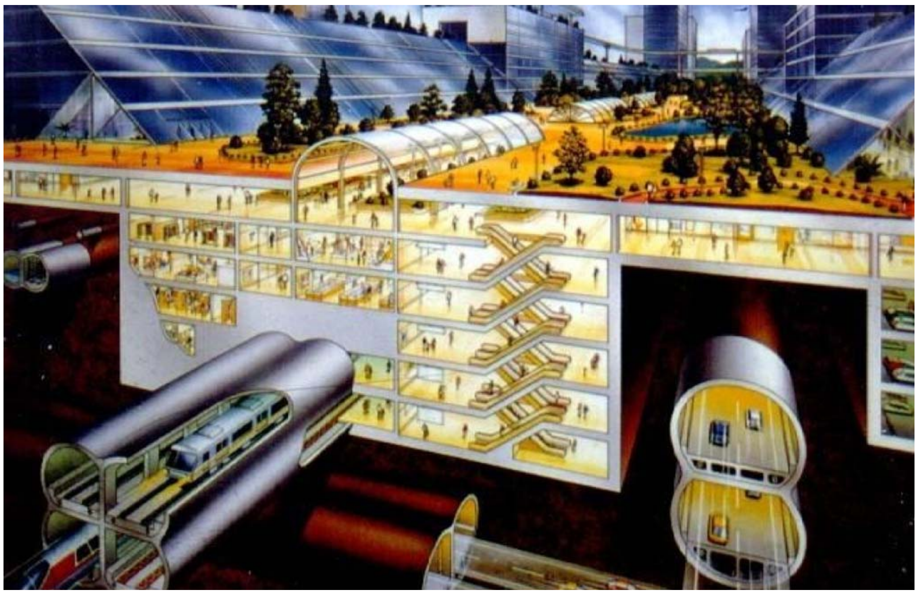

Figure1. Various facilities of underground structures

The figure no.1, presents various facilities in large cities are increasingly constructed under the ground. In addition to infrastructure facilities, underground roads, railway tunnels, underground stations, subways, underground spaces in the building and shopping centers are built, sports and recreation as well as warehouse and garage spaces. Therefore, it is increasingly necessary to perform deep excavation work and their protection in urban city areas.

Depending on the depth of the excavation, the dimensions of the foundation pit and soil different systems of protection of the foundation pit are applied. If the depth of the excavation is large (several underground floors) sheet of drilled piles or concrete diaphragm wall is usually applied. If the soil is incoherent and if the groundwater level is high then the protective structure of reinforced concrete diaphragm is performed. Protective structures should satisfy two conditions, namely:

- That they can accept lateral pressure of soil and water with a sufficient factor of safety;
- That changes in the stress strain conditions in the soil around the foundation pit during the construction are relatively small and that they do not cause damage to adjacent buildings and installations around the pit.

For depth of excavation greater than 5-6 meters, such cantilever construction elements which are elastically wedged into the soil can not accept lateral pressure of the soil. Depending on the composition of the soil, displacements become large and threaten to induce damage to adjacent buildings. Therefore, for greater depths of the excavation it is necessary to perform shoring of protective perimeter wall construction of the foundation pit. Depending on the dimensions of the pit, bracing can be done by performing some temporary steel structure inside the pit or by the geotechnical anchors. For greater depth shoring of pits can be done in several levels. Such works are complex, relatively expensive and long lasting. Therefore, in recent times pit shoring is performed with the construction of the underground part of the building, which is being constructed at the same time. One way of performing this kind of work is the so-called "top down" method which is often applied in the world while with us this method is at an early stage. This method of construction allows the parallel construction of underground and above-ground part of the building.

In addition to the analysis of methods of performing the protection of foundation pits, this paper provides a review of design methods of these structures. Design of protective structures is basically analysis of the interaction between the structure and soil. Such calculations are complex because they need to properly cover behavior of the soil, which is non-linear and the construction phase. As an illustration of the presented methods of calculation at the end of the paper a few examples are presemted.

2. METHODS OF CONSTRUCTION OF FOUNDATION PITS

2.1. METHODS OF CONSTRUCTION OF PIT PROTECTION

There are many different ways to construct the protective structure of foundation pits. When choosing solutions designer explores the composition and properties of soil, groundwater levels and water permeability of soil layers, depth of foundation pit and its dimensions, the existence of neighboring objects and installations and their condition, the structure of the object which is being built inside the foundation pit, the available machinery and equipment of potential contractors, cost of works to protect the foundation pit and the time required for execution of the work.

Protective structures of foundation pits consist of two important elements, such as circumferential wall and construction for shoring of wall.

On the choice of circumferential wall mostly influence the composition and properties of the soil layers and the groundwater in the soil. These walls can be run in the form of: curtain of bored piles, reinforced concrete diaphragms, Berlin talps and other methods. In the next section we will give a short description of these methods with a description of their characteristics.

Protective wall of foundation pits can be constructed in the form of a curtain wall of the drilled piles. The diameters of the piles depend on the magnitude of the forces that can occur in piles. The axle distance between the piles depends on the kind of soil and the soil water. If the soil is cohesive and if water inflow is small, then the maximum distance between piles is not supposed to be greater than three pile diameters. Doing so may lead to the collapse of the relieving arch in the soil behind the piles and soil leaking into the pit. If soil is non-coherent and under water then the piles are performed adjacent to each other.

Protective wall of foundation pits can be constructed from reinforced concrete diaphragms. These walls are thick 40, 60, 80 and 100 cm, depending on the size of forces in them. The diaphragms can be monolithic concreted on site or can be made from prefabricated elements. Such a method of construction of protective wall is high-quality and reliable, the disadvantages are the high costs and large thicknes of protective structures.

Also, there are different ways to construct a protective wall such as Berlin method, the walls of the Larsen talps or precast concrete elements that can be combined with parts of the walls that are concreted on the spot.

Depending on the depth of the excavation for foundation pit, the walls can be derived as a cantilever elements which are elastically wedged into the ground. This can be applied to the excavation depth not exceeding 5.00m. If the depth of the excavation is larger, then the deformation of these structures are large, which could cause problems in adjacent buildings. Therefore, if the depth of foundation pit is large, it is necessary to carry out their bracing, in order to reduce stress and forces in them.

The way of supporting perimeter walls of foundation pits depends on the size of the force that they must accept and the dimensions of the foundation pit. Figure no. 2 presents the foundation pit bracing with steel construction. It can be seen that the bracing is performed in multiple levels with very strong steel elements. These structures are not easy to construct, cost a lot, and at the end of the construction works it is necessary to make their dismantling. Also, these structures can significantly complicate the performance of the excavation inside the pit and taking out of the excavated material.

As an alternative to supporting the protective wall of the pit, ingineers often apply ground anchors. Depending on the depth of foundation pit and the size of the load acting on the circumferential wall, geotechnical anchors can be performed in one or more rows in height. In the figure no. 3, it is shown a relatively deep foundation pit in which bracing of perimeter wall was performed with geotechnical anchors in several levels.

Reliability and safety of this method of supporting perimeter walls of the foundation pit is largely dependent on the characteristics of the soil layers that are used for anchoring the anchor. It is not uncommon that in time there is a loosening of the anchors and as a result major deformation of the supporting structure. Moreover these works are expensive and take a long time.

Figure 2. Shoring of the support structure

Figure 3. Anchoring of the support structure

Therefore, in recent times, a lot of work is done on how to design different ways of protecting fundation pits that would be safer and would cost less, especially bearing in mind that this is a temporary structure, whose function practically ceases when the construction is carried out inside the pit.

2.2 BASIC PRINCIPLES OF "TOP DOWN" CONSTRUCTION

As mentioned in the introduction in the design of protective structures for deep excavations, structure of the building itself is often used for shoring of the protective wall. Works start with a relatively small excavation and formation of the working plateau. Then a protective structure around the perimeter of the future building is performed. In order to enable working together of performed piles or diaphragm, above them a reinforced concrete coupling beam is peformed. It is recommended that this beam has a high stiffness to allow the spatial work of protective structure, which is necessary at certain stages of the works.

In the following step excavation is caried out to the depth to which the protective structure can be performed as the console element, which is elastically wedged into the ground, to accept lateral pressure of the soil, while keeping its deformations small and do not cause damage to adjacent buildings . From prepared plateau, on future colum positions or other suitable places temporary supports – poles are constructed. These supports are performed as bored piles which are concreted up to the level of the lower edge of the foundation slab, and from that point up in the form of steel sections or pipes. To avoid buckling of thesee elements the space around steel elements in the borehole is filled with earthy material, usually sand.

Next, on the prepared surfaces or formwork ceiling is cast in place with concrete. Derived ceiling represents also the horizontal strut for protective structure of foundation pit. In the horizontal plane, this structure has high stiffness, so that it can accept lateral pressure of the protective structure. In this ceiling a required number of openings is left for the formation of the ramps or vertical transportation of excavated soil underneath. By performing this ceiling conditions for the simultaneous performing of work on the down and up are created.

On the up the construction of the object is performed in the usual way. Depending on the capacity of bored piles and bearing capacity of steel elements adopted in column positions the number of floors above ground that can be performed prior to the completion of the underground part of the building's structure is determined. Most often these elements are adopted so that the construction of the underground part and the above ground part of the structure takes about the same time. When the concrete hardens, and performed ceiling receives sufficient strength, next construction phase can be accessed, ie. excavation carried below the ceiling.This excavation is performed by machine. To allow for mechanical excavation smaller excavators are used and excavation is carried out simultaneously for two floors. With a grid of temporary supports between 7:00 and 8.00m height of the excavation of two floors around 6.00m, working conditions for medium-sized excavator are just a little slow in relation to the excavation in the open.

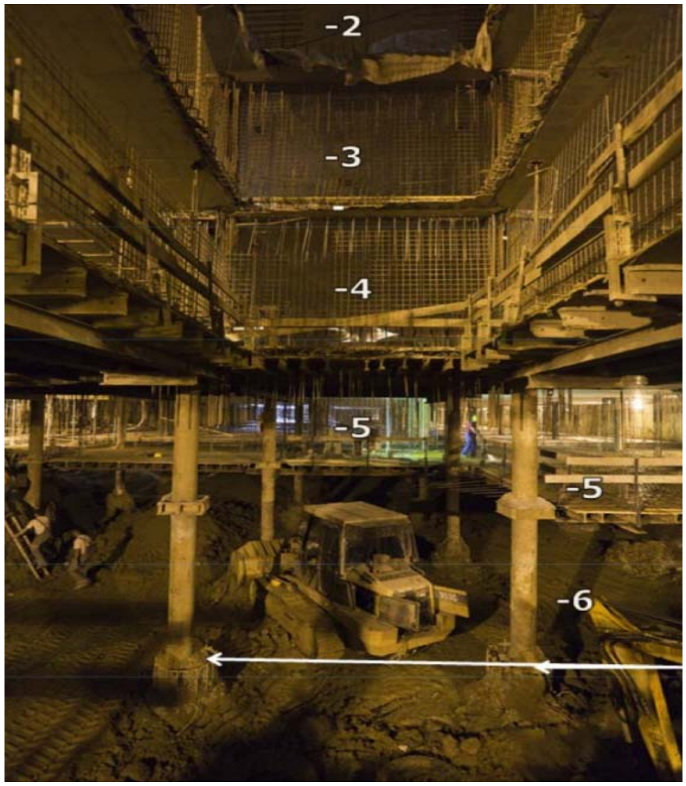

Figure 4. An example of "top down" construction

When the specified excavation is completed next cealing is casti in place with concrete below the first already performed cealing. Such a method of deepening foundation pit continues the same way until the final excavation. Often, it is necessary that the excavation is carried out with simultaneous decline of groundwater. Depending on the geotechnical characteristics of the soil layers and groundwater level this dewatering can be performed in different ways. To minimize the water inflow in foundation pit it is recommended to perform diaphragm wall around the perimeter and to penetrate deeper layers of lower permeability [7].

Upon completion of the excavation to final depth, surface preparation is carried out for execution of the foundation slab. This slab accepts the load of the building together with the bored piles in column positions. If the bearing capacity of these piles is not sufficient to accept the total load of the building together with the foundation slab, then the slab can have openings for additional piles which are driven later [1,10]. These piles are driven into the soil with hydraulic presses which use as ballast weight of the structure of the object and the weight of the base plate for which the construction for anchoring is anchored.

In carrying out this work, special attention should be paid to the correct solution and the performance of the waterproofing of the underground part of the structure of the object. Isolation can be performed simultaneously with the execution of the works on the construction or at the end in the form of various penetrates. At the end of works on the underground part of the structure temporary steel suports are modified to designed columns.

The above described method of creating protective structure allows to excavate foundation pits to large depth, with sufficient security. Economic analyzes show that these solutions are significantly better than other methods where the shoring of pit is performed with temporary structure or with anchors. The proposed method of works significantly shorten the construction. This is particularly important when the works are in urban conditions and they interfere normal traffic. One of the characteristic features in these works is that the foundation of building is being performed on the combined piles and foundation slab. This combined work off shallow and deep foundations is not applied frequently in practice [2, 3, 4, 8, 9, 11, 12]. The calculations of these structures are complex, because there is interaction between piles, foundation slab and soil.

3 METHODS OF ANALYSIS OF CONSTRUCTIONS FOR PIT PROTECTION

3.1. INTRODUCTION

Calculations of deformable protective structure of foundation pits, by their nature, are very complex. Typically, such structures are analyzed as beams which are located in the mass of soil, where it acts as a load, but which are supported by the ground. They are a classic example of the interaction between the structure and soil. From the outside of the protective structure soil pressure acts, the arrangement and intensity of this load is not known in advance and can change depending on the deformation of the protective structure. On the inner passive side protective structure rests on the ground. Arrangement and the intensity of this load depends on the size of the deformation of the protective structure and the stiffness of the soil. Also, the load of the groundwater acts on the protective construction. This load can be hydrostatic, and if you establish a flow of water around the protective constructiones then be taken as hydrodynamic load. Constitutive modeling of soil behavior is highly complex because the behavior of the soil depends on its geological history as well as to the stress change - load history. Also, stress in the protective structure and the surrounding soil are largely dependent on the phase of execution of the works and ways of shoring the supporting structure. Shoring can be performed with different struts, geotechnical anchors or the construction of the underground part of the building.

There are several groups of calculations and analysis to determine the stress in the protective structures of the foundation pits and the surrounding soil. In the next section we will present: the method of ultimate limit analysis, the equivalent frame method and modern numerical methods.

3.2. METHOD OF ULTIMATE LIMIT ANALYSIS

Ultimate limit analysis methods are based on the assumption that the soil reached the border state of stress in the horizontal direction [13,14]. This means that it is assumed that on a supporting structure act on the outer side active pressure, and on the inner side passive pressure. In addition to these horizontal loads on the protective construction of foundation pit can act loads from water. This load is usually calculated as the hydrostatic pressure on both sides of the structure, depending on the water level. If the water establishes its flow, then hydrodynamic pressure of water should be takken into acount. This load is considerably less than the hydrostatic load of water.

Since it is assumed that the soil is in a ultimate limit state, this means that the size of the load does not change depending on the deformation of the supporting structure. The problem of this calculation comes down to satisfying the force equilibrium conditions, without taking into account the deformation.

If it is assumed that the deformation and displacement of protective structures are such that on the outside the pressure of the soil dropped and reached its minimum value ie. active earth pressure, from the equilibrium condition it is necessary to determine the pressure of soil from the internal side of the supporting structure. As an unknown size the depth of soil to which there has been a full plasticity as well as the value of earth pressure at the bottom of the protective structure are adopted. By solving the equilibrium conditions of horizontal forces and moments, it is possible to determine the unknown values.

In the numerical example at the end of the work, it is presented the results of the calculation of the wall that is treated as a console wedged into the ground. It should be noted that this type of calculation can be applied only in simple systems which do not analyze the construction phase or changes in the static system during the construction of protective structures of the foundation pit. Such calculations can be applied in the analysis of the global stability of the supporting structure.

3.2. THE EQUIVALENT FRAME METHOD

One of the methods of calculation which is often used in the calculation problems of interaction of structure and soil is the equivalent frame method [5, 6]. In these methods, the protective structure is treated as a beam which is supported with a system of elastic springs which simulate the effect of the soil. To solve these problems computer program for the calculation of static actions in the frame structure are used. Because of their simplicity, these calculations are very often used. The accuracy of the obtained results largely depends on the manner of modeling the behavior of the soil as well as the proper modeling of the system at different phases of the protective structure.

If in the soil flexibility matrix diagonal elements are ignored, the flexibility matrix of the soil and the stiffness matrix of the soil are reduced to a diagonal matrix. In this way, the behavior of soil is described with Winkler's model.

Using methods of equivalent frame, it is possible to describe quite well the behavior of the protective structures of the foundation pit, if we properly describe soil behavior. Based on these methods there have been many proposed different types of calculations. The author of this paper proposed a procedure for the analysis of deformable retaining structures, which will be described in more detailed way in the following section.

As mentioned in the introduction, calculations of protective structures are complex, because of the non-linear behavior of soil. At each change of stress in the soil elastic and plastic deformation ocure. In addition, the deformation in the soil depends on the stress trajectory in the soil, which means that calculation of deformable retaining structures should be aligned with the phases of the works.

3.2.1. The proprsed numercal method

As is mentioned before, design of embedded cantilever or propped retaining wall is very complex, and must follow the constructing stages. The soil is assumed to be fully drained. The water pressure is hydrostatic or hydrodynamic, below the water table. The incremental and iterative method of analysis should be applied because of the nonlinear stress-strain relations in soils.

The in situ state of stress in soil is defined in terms of the current values of effective vertical stress σ'_{vo} and effective horizontal stress σ'_{ho}. For horizontal, level ground, the in situ vertical stress is:

$$\sigma_{vo} = \sum h_i \cdot \gamma_i \tag{1}$$

However, the horizontal stress is more difficult to evaluate. The stress ratio K_0, which is the at rest coefficient of horizontal soil stress, is defined as ${\sigma'_{ho}}/{\sigma'_{vo}}$. For normally consolidated soil, the simplified Jaky equation provides reasonable estimates for K_0, as is given below:

$$K_0 = 1 - \sin\varphi \tag{2}$$

Many factor affect the in situ state of stress in soil, including: overconsolidation, aging, chemical bonding, etc. Overconsolidation is probably most influential for the majority of soil. For the overconsolidated soils, the general relationship for K_0 is often expressed as:

$$K_0 = (1 - \sin\varphi) \cdot OCR^n \tag{3}$$

In some case, close to the margins of the excavation there are existing structures. The foundation pressures generate additional stresses in the soil. The stresses in an elastic and isotropic half-space produced by a uniform vertical load, over a flexible form:

$$\sigma_z = \frac{p}{\pi} \cdot \left(arctg\frac{b-x}{z} + arctg\frac{b+x}{z} \right) - \frac{2 \cdot p \cdot b \cdot z \cdot (x^2 - z^2 - b^2)}{\pi \cdot [(x^2 - z^2 - b^2)^2 + 4 \cdot b^2 \cdot z^2]} \tag{4}$$

$$\sigma_x = \frac{p}{\pi} \cdot \left(arctg\frac{b-x}{z} + arctg\frac{b+x}{z} \right) + \frac{2 \cdot p \cdot b \cdot z \cdot (x^2 - z^2 - b^2)}{\pi \cdot [(x^2 - z^2 - b^2)^2 + 4 \cdot b^2 \cdot z^2]} \tag{5}$$

Before excavation, retaining wall and soil are making statically an equilibrium system. This static system can be represented by an equivalent frame.

The retaining wall is discrete by two nodded beam elements. The influence of soil is substituted by the horizontal springs (boundary element) at nodal points, at both sides of wall. The system is subjected to horizontal ground pressure at rest and hydrostatic pressure below ground water level. This loading is in equilibrium. In this paper the finite element method is employed to calculate tangential stiffness of the boundary elements. It was assumed that soil behaves as linearly elastic material. Every soil layer is defined with two parameters E_s and ν_s, determined for the stress level in the middle of every layer. Using finite element formulation the soil flexibility matrix F_s is numerically evaluated.

The coefficients in F_s are horizontal nodal displacements due to external applied until horizontal nodal forces. By using the principle of superposition the horizontal displacements of the nodal point, due to horizontal soil pressure at rest, may be written in the matrix form as:

$$U = F_s \cdot P \tag{7}$$

To model the soil behavior an hypoelastic Duncan-Cheg model is used. The hypoelastic concept can provide simulation of constitutive behavior in the smooth manner and hence can be used for hardening or softening geological materials. Use of the hyperbola for representing stress-strain curves for soil was proposed by Kondner. To incorporate this aspect, Duncan and Chang used the hyperbola in conjunction with the relation between initial modules and confining pressure by Janbu. The following expression for the tangent modules can by obtained as:

$$E_t = K \cdot \left(\frac{\sigma_3^{'}}{p_a}\right)^n \cdot \left[1 - \frac{R_f \cdot (\sigma_1 - \sigma_3) \cdot (1 - sin\varphi)}{2 \cdot (\sigma_3 \cdot sin\varphi + c \cdot cos\varphi)}\right]^2 \tag{8}$$

In that expression the Mohr-Columb failure criterion is incorporated. For unloading the initial modulus is used. To correct the evaluated horizontal nodal displacements a diagonal matrix *D* is formed. The coefficients in D are ratios between modulus of elasticity and tangent modulus at every nodal points. The stiffness of the boundary elements may be evaluated as a ratio between nodal forces and corrected nodal displacements.

$$K_i = \frac{\sigma_{h,i} \cdot \Delta H}{U_i^*} \tag{9}$$

The excavation was simulated by sequentially removing the thin soil layers slices in front of the wall. Removal of slices was simulated by first calculating the equivalent nodal forces arising from the stresses acting within these slices and then applying those which acted on slices remaining in their opposite sense as boundary conditions for further increments of the analyses. Correct account was taken of both the initial stress and those stress changes which occurred during the excavation process. At the base of the excavation the soil is subjected to passive stress relief.

As a result of applying nodal forces on the equivalent frame, horizontal displacements of the nodal points towards the excavation are obtained. The consequence of this displacements are changes of horizontal stresses in the soil. In the front of the wall ,passive site, the pressure in the soil increase and is given by:

$$\sigma_{hp}(I) = \widehat{\sigma_{hp}}(I) + \frac{P_a}{\Delta H} - \Delta\sigma_{h0}(I) \qquad (10)$$

Where are:

σ_{hp}- horizontal stress in nodal i, $\widehat{\sigma_{hp}}$- stress in the same nodal before applying incremental load, P_a- force in the boundary element on the active site.

After all needed calculation are performed, it is necessary in all nodal points on both sides of wall, to calculate the safety factors according to next expression:

$$F_s = \frac{2\cdot(\sigma_3\cdot sin\varphi + C\cdot cos\varphi)}{(\sigma_1 - \sigma_3)\cdot(1 - sin\varphi)} \qquad (11)$$

If this factor is larger than previously calculated (before excavation slice), it means that this node have undergone unloading. In this nodes the initial modulus have to be used. Also, it is necessary to check whether the stress in the nodes are greater than active, are smaller than passive. In the nodes where it is not satisfied, the boundary elements are removing and replacing bay active and passive pressure. This procedure is used iteratively within every incremental loading to monitor the plastic zone development at the interface of wall and soil. All nodal stress and deformations obtained at the end of iteration process, within the considered increment, are stored. This procedure is repeated for the next increment of excavation and obtained results are added to the already stored from previous one. If the anchors or supports are designed, they can be also incorporated in the calculation process. If the excavation takes under the level of ground water table, the increments of hydrostatic pressure are applying. According to above explained procedure the computer program is made. Using this program efficient calculation can be easily performed.

3.3. MODERN NUMERICAL METHODS

A more accurate calculation procedures for protective structures of foundation pits and changes in stress-strain conditions in the soil around them, require the application of numerical calculation method. One of the methods that are commonly applied is the finite element method FEM. In this method, solving the equations that describe a problem within a restricted domain is reduced to solving a large number of linear equations in which the unknown parameters of the network nodes, which is done discretization domain in finite elements.

In the next section we will give a brief description of the FEM code calculation of deformable protective structures. In order to solve these problems successfully it is required to correctly choose the way of analysis, as well as the appropriate model of the soil.

Soil properties largely depend on the geological history. They can vary considerably and therefore must be measured. In recent years there has been significant progress in constitutive modeling of soil behavior. The main objectives of the constitutive model of the soil are that they describe well the behavior of the soil and that the parameters for their description can be obtained on the basis of conventional experiment, also that their change in the numerical analysis is not complicated. Such a model that meets all those requirements has not yet been found.

In elastoplastic modeling of soil behavior, it is necessary to properly select surfaces of yield, which separates the state of stress that causes only elastic deformation of the stress condition which causes the elastic and plastic deformation. In addition to these functions the surfaces of the plastic potential are itroduced which define the distribution of increments of plastic deformation during plastic yield. Also, in addition to these surfaces in plasticity theory the law of hardenig is introduced, that is used to define the spread of yield surface as a function of accumulated plastic deformation. To model shear and volumetric plastic strain there is a need for defining models with more surfaces of loosening. Therefore, in addition to the surfaces of loosening, which define the state of stress that leads to breakdown due to shear stress, the surface of loosening which depends on the volumetric plastic deformation is intoduced. Usually, these models are called models with the cap. This area, describing the constitutive behavior of the soil is significantly improved. However, formation of such a model requires a very large number of material parametrara. These parameters can be determined only in laboratories that are well equipped and in which experiments can be run by defined stress paths. Due to problems with determining the necessary parameters to describe the behavior of soil with most computer programs, it is proposed to use simpler models which can be described with smaller number of parameters. Because of the non-linear behavior of the soil, using FEM such problems can not be solved in one step. Consequently for solving this problem incremental formulation is used. In this formulation, the load is applied in a number of increments, and in each increment the problem is linearized. This means that the solution of the nonlinear problem is reduced to solving a sequence of linear problems. To improve the accuracy of solutions within increments iterations are made, and therefore solving nonlinear material problems is achieved by using incremental-iterative methods. The next section gives an overview of the basic equations of FEM for solving the problem of incremental-iterative procedures.

$$^{t}K^{(i-1)} \cdot \Delta u^{(i)} = {}^{t+\Delta t}R - {}^{t+\Delta t}F^{(i-1)} \tag{12}$$

$$^{t}K^{(i-1)} = \int_V B^T \cdot {}^{t}D_{EP}{}^{(i-1)} \cdot B \cdot dV \tag{13}$$

$$^{t+\Delta t}F^{(i-1)} = \int_V B^T \cdot {}^{t+\Delta t}\sigma^{(i-1)} \cdot dV \tag{14}$$

$$^{t+\Delta t}\sigma^{(i-1)} = \int_{^{t}\varepsilon}^{^{t+\Delta t}\varepsilon^{(i-1)}} {}^{t}D_{EP}.d\varepsilon \tag{15}$$

To get an accurate calculation results at the contact of different materials where there is a large change in stiffness the use of contact elements is recomended. With these elements in a relatively thin contact zone we define different behavior of materials from basic materials in contact. Thus, the recommendation is to use contact elements between reinforced concrete structures and soil, which allow relative movement in of material over another.

Today a great number of high-quality programs that can sucessfully solve these problems is available. Here are just a few of them: PLAXIS, ADINA, DIANA, ABAQUS, SOFISTIK. Figure 5, presents a solution of a supported protective structure using the program PLAXIS. In addition to the finite element mesh the figure shows the size of the calculated horizontal displacements of the protective structure and soil.

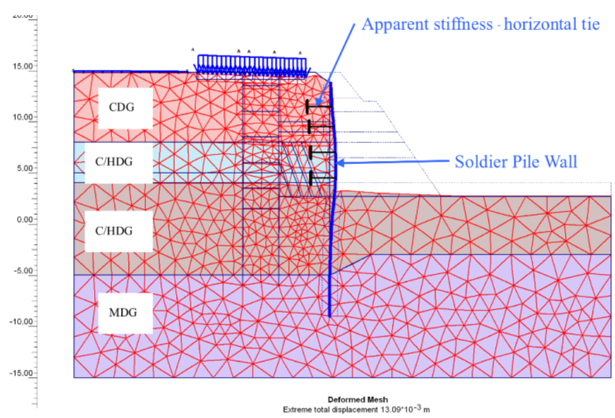

Figure 5. Finite element mesh in computer program Plaxis

In addition to calculation of stress-strain conditions in the protective structure of foundation pit and surrounding soil, such calculations can be successfully solve the problem of water flow around the protective structure.

In certain cases, when the dimensions of the foundation pit are relatively small ande the depth of the pit is greater, instead of solving plane problems in typical cross sections, it is necessary to treat the problem as a spatial.

The next chapter presents simple examples based on which it may be concluded that the results of the calculations change depending on the applied calculation methods as well as the phase of execution of works.

3.4. NUMERICAL EXAMPLES

In order to illustrate how the methods adopted by calculation affect on the accuracy of the results calculation have been made for two simple examples. In both examples a reinforced concrete diaphragm that is made in a layer of sand is considere

EXAMPLE 1

In this example, an analysis of the results of calculations depending on the chosen methods for the design of protective structures is made. We analyzed only the first phase of excavation where the excavation in front of the diaphragm was performed to a depth of 3.00

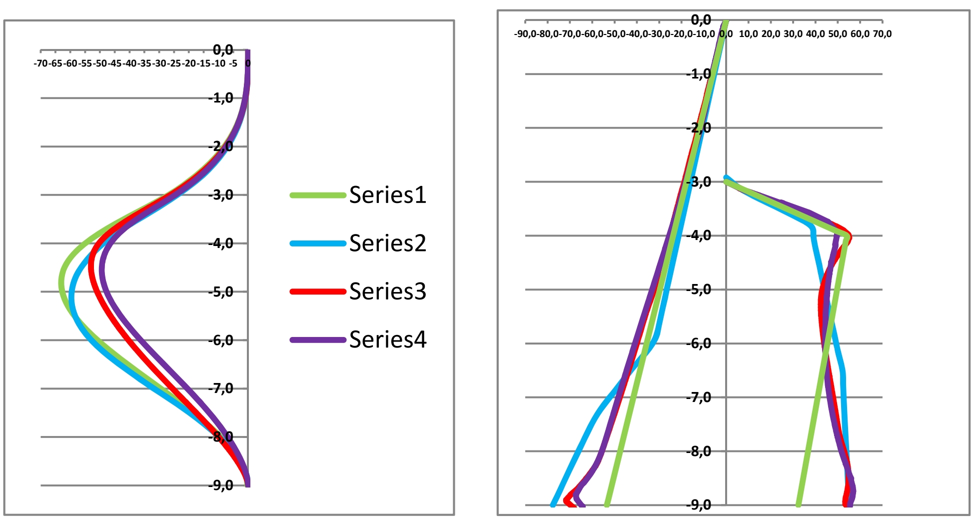

Figure 6. Moments and horizontal loads in diferent methods of calculations

EXAMPLE 2

In this case, it is shown how phases of work affect the results. The first part presents previously mentioned diaphragm at which excavation was performed to a depth of 3.00m, and then at 0.00m the cealing slab was made. In the second part, the order of execution of works was changed, so the ceiling was made first at a level of 0.00m, and then excavation to a depth of 3.00m was performed.

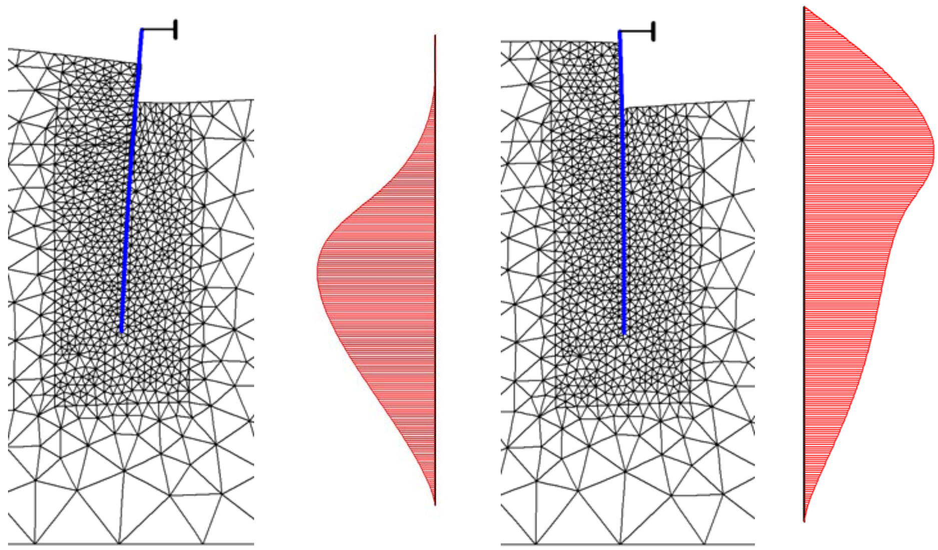

Figure 7. Deformed mesh and moments in diferent order of phases

4. CONCLUSIONS

Based on everything sad we can draw the following conclusions:

1. This paper analyzes different ways of performing work on the protection of foundation pits. With deep foundation pits, preference should be given to top-down construction method in which the protective structure is shored with the building structure, which provides the necessary security during construction. Changes in stress and strain in soil are relatively small, so there is no additional subsidence that may cause damage to adjacent structures. These solutions are rational.
2. As far as the calculation, it is important to properly model soil in which the works are carried out and that the calculation include the execution phases of the works. With the use of modern numerical methods we can successfully solve the problems of the calculation of protective structure of

foundation pits.

5. LITERATURE

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