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COMPARATIVE ANALYSIS OF DESIGN METHODS OF TRANSVERSALLY LOADED DIAPHRAGMS

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Abstract. *Reinforced concrete diaphragms are in-built supporting structures constructed directly in the ground. They are intended for reception of lateral soil pressures, and due to the thickness-height ratio they belong to the group of deformable structures. The paper presents different design methods of transversally loaded diaphragms as well as constitutive soil models which can be used on this occasion. For comparison of the described methods, one example of design of reinforced-concrete diaphragm with the analysis of obtained results was done. The diaphragm is firstly treated using classical analytical methods, and then using the numerical methods based on the concept of problem discretization using finite differences method and the STRESS, TOWER and PLAXIS software. The goal of the paper is as accurate prediction of the diaphragm and surrounding soil behavior as possible, as well as finding of the relevant impacts required for the design.*

Key words: *diaphragm, soil, transversal load, interaction, modeling, discretization*

1. INTRODUCTION

A diaphragm is a thin wall in the ground, built according to a certain technology using special equipment. It is successfully used as a ground water barrier (anti-infiltration diaphragm), as an independent element of the deep foundations (wells, boxes), as a protection of deep foundation pits and as a retaining structure. Diaphragms are constructed from: raw clay, processed clay, clay-concrete, non-reinforced and reinforced concrete. Basic construction technology comprises trench excavation using machinery. The trench is simultaneously filled with the slurry. After completion of the excavation, the trench is filled with the material designed for construction of the diaphragm, which displaces the slurry. The advantages of such construction are as follows: the trench excavation does not disturb the existing equilibrium in the soil, and the excavation implements are attached to the

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standard backhoes, and the diaphragms can be built next to the already existing structures (it is possible to build the underground traffic lines below the existing streets without interrupting the traffic), ground water does not affect the diaphragm construction, etc.

2. DIAPHRAGM DESIGN

Diaphragm designs primarily depend on the adopted model of soil and implemented design method. Design methods of the diaphragms can be very different [1,3,6,10,12,13]. Regarding to the soil models the most frequently applied ones are:

- ideally-elastic models (one-parameter and two-parameter ones)
- ideally-elastoplastic and
- elastoplastic models with reinforcement.

As for the design methods, the following ones are applied:

- the methods starting from the differential equation of the elastic diaphragm line, and analytically solving the problem
- the methods based on the concept of discretization of the diaphragm and the surrounding soil

The majority of the methods encountered in the designing practice and literature deals with the soil using Winkler's one-parameter method as the simplest and the easiest to implement. Here the soil is represented by the system of the independent linearly elastic springs (fictitious members) where the deformations occur only in those springs where the loads occur as well. The soil is described with a single parameter – soil reaction coefficient (in vertical or horizontal direction) which is often taken as a constant or linearly progressive with the depth. By using this method of soil representation, a well known method of initial parameters which solves the problem in analytical manner was developed. Apart from that, the Winkler's one parameter model was widely represented in numerous software packages in the field of civil engineering (SAP, STAAD, TOWER, STRESS) based on the discretization principle.

The second group of methods is based on the assumption that the soil is linearly elastic, homogenous continuum using the two-parameter soil models in the analyses. Soil properties are defined by two parameters, modulus of elasticity Es (which also can be either constant or variable with depth) and Poisson coefficient ν_s. The analytical model of problem solving based on this more realistic, but considerably more complex model to use in practice, is not widely used exactly because of the complicated solving procedure. This model can often be encountered in the software packages having specialized – geotechnical use (PLAXIS, GEO-SLOPE) and general use (ANSYS, NASTRAN) which are based on the finite elements method.

Ideally –elastoplastic soil models (in geotechnics, the most common Mohr-Coulomb and Drucker-Prager models) are the integral part of the mentioned packages of specialized and general purpose, while the elastoplastic models with reinforcement (Cam-Clay model and similar) are present only in the geotechnical purpose software.

3. DIAPHRAGM DESIGN BASED ON THE SOIL REACTION COEFFICIENT

Soil reaction coefficient k_H defines the dependence between the horizontal movement of the points on the axis of the vertical deformable support $x(z)$ and reactive horizontal soil pressures $\sigma(z)$ at the depth z most frequently expressed in the form:

$$k_H = \frac{\sigma_z}{x_z} = f(z) \tag{1}$$

Where – $f(z)$ is the function of the depth related distribution of the soil reaction coefficient.

When solving the problem of transversally loaded diaphragms, the constant or linearly increasing value is most frequently at the starting point (Figure 1):

$$k_H = f(z) = k_h = const \tag{2}$$

$$k_H = f(z) = k_h \frac{z}{h_m} \tag{3}$$

Even though there are other, more complex depth related soil reaction coefficient distributions.

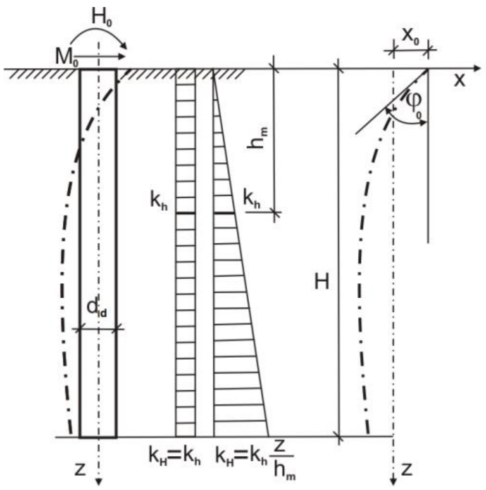

Fig. 1 Adopted distribution of soil reaction coefficient: constant or linearly increasing value

The relevant value of the soil reaction coefficient k_h and the corresponding depth h_m according to [2] can be determined on the basis of the expression:

$$k_h = m \cdot h_m \tag{4}$$

$$h_m = 2(d_d + 1) \tag{5}$$

whereby:

d_d – is the thickness of the diaphragm in meters and

m – is the characteristic dependent on the type of soil [kN/m^4] according to [2].

When used in the most frequently implemented software packages in this field, the coefficient of the horizontal soil reaction can be calculated based on the soil modulus of elasticity according to the Vesić expression:

$$k_h = \frac{0.65}{d_d} \sqrt[12]{\frac{E_s \cdot d_d{}^4}{E_b \cdot I_d}} \cdot \frac{E_s}{1-\nu^2} \tag{6}$$

where:

d_d – is diaphragm thickness
E_s – is soil modulus of elasticity
E_b – is concrete modulus of elasticity
I_d – is moment of inertia of the diaphragm cross-section
E_s – is soil modulus of elasticity
ν_s – is Poisson soil coefficient

If the ratio of the depth of the diaphragm h and its thickness d_d is higher than 10 ($h/d_d > 10$) the diaphragm belongs to the group of deformable vertical supports where the deformations due to the diaphragm bending are considerable, and significantly affect the function of soil resistance distribution, so the design is performed in accordance with it.

According to [8] and indicator of stiffness α can be introduced for the support width of 1.0 m:

$$\alpha = \sqrt[5]{\frac{m \cdot 1.0}{E_b I_d}} \tag{7}$$

For the values $\alpha \cdot h \geq 2.5$, where h is its depth, the diaphragm can be considered a deformable structure.

The diaphragm design methods which are most commonly encountered in our geotechnical practice will be represented further.

4. INITIAL PARAMETER METHOD

The method developed in [8], is known here as the initial parameter method. It deals with the problem in an analytical manner and provides the solution in a closed form which is convenient for practical usage. The soil is represented as a linearly deformable environment, characterized by the horizontal soil reaction coefficient $k_{H.}$ According to [8], this design method is recommended when $\alpha \cdot h \geq 2.5$.

When considering the problem, the starting point is the basic differential equation of the elastic diaphragm line:

$$E_b I_d \frac{d^4 x}{dz^4} + \sigma_z = 0 \tag{8}$$

Whereby $E_b I_d$ is the stiffness of the diaphragm to bending.

Using the Winkler assumption, given by the expression (1), the expression (8) can be written in the following form

$$E_b I_d \frac{d^4 x}{dz^4} + k_H x = 0 \tag{9}$$

For the constant coefficient of reaction in the horizontal direction, $k_H = k_h = const$ the expression (9) is:

$$E_b I_d \frac{d^4 x}{dz^4} + k_h x = 0 \tag{10}$$

While in case of the linearly increasing soil reaction coefficient it has the following form:

$$EI \frac{d^4 x}{dz^4} + k_h \frac{z}{h_m} x = 0 \tag{11}$$

The solution of the equations (10) and (11), in accordance with the corresponding boundary conditions are known in literature and provided in an analytical form.

For $k_H = k_h = const$ according to [2] is

$$x(z) = \frac{2\lambda}{k_h} e^{-\lambda z} \, [H_0 \cos\lambda\, z + M_0 \lambda\, (\cos\lambda\, z - \sin\lambda z\,)] \tag{12}$$

While for the linearly increasing soil reaction coefficient k_h according to [2] it is

$$x(z) = x_0 A_1 + \frac{\varphi_0}{\alpha} B_1 + \frac{M_0}{\alpha^2 EI} C_1 + \frac{H_0}{\alpha^3 EI} D_1 \tag{13}$$

Whereby

$$\lambda = \sqrt{\frac{k_h}{4EI}} \tag{14}$$

The remaining parameters in the forms (12) and (13) are:

A_1, B_1, C_1, D_1 – the influential functions dependent of $\bar{z} = \alpha \cdot z$

H_0, M_0, x_0, φ_0 – initial parameters.

For $z = 0$ the initial parameters H_0 and M_0 are defined and known (M_0=M, H_0=H), while the other two parameters, x_0, φ_0 must be determined from the support conditions of the lower end of the diaphragm. The lower end of the diaphragm can:

- Be either free in the soil or resting on the bedrock (boundary conditions $T_h = 0$, $M_h = M_R$)
- Be embedded in the bedrock (boundary conditions $x_h = 0$, $\varphi_h = 0$)

By the gradual differentiation of the expressions (12) and (13) the expressions for the cross section rotation angles $\varphi_{(z)}$, bending moments $M_{(z)}$, and transversal force $T_{(z)}$ are obtained.

5. Diaphragm Design Using Stress and Tower Software

There is a number of software packages starting from the Winkler assumption of soil behavior, and they are based on the concept of discretization of the vertical deformable support and replacement of the surrounding soil by the system of elastic supports (spring, fictitious members). One of them is the STRESS software, where from the conditions of equal base layer deformations (Δl_0), and contraction of fictitious members (Δl), the surface areas of the cross sections of fictitious members are determined:

$$A_{f\,i} = \frac{A \cdot k_{hi} \cdot l}{E} \tag{15}$$

where:

A – is the belonging surface area of one segment of the diaphragm
k_{hi} – is the coefficient of the horizontal reaction of soil for " i " - th fictitious member
l – is the length of the fictitious member (most frequently 1.0m)
E – modulus of elasticity of the diaphragm material

In the formed frame structure of the known dimensions and the given load, by calculation of the static impacts, the forces in the replacement members, and afterwards the reactive soil pressures, bending moments, transversal forces, displacements and cross-section rotation angles along the diaphragm axis are obtained.

In a similar way, the soil is modeled in the TOWER [11] software. Namely, here the soil is replaced by the support which can be either linear or planar, depending on the structure, and it is characterized by the soil reaction coefficient.

6. Design Based on the Application of Linearly Elastic, Homogeneous Continuum

Among the very commonly used soil models for diaphragm design is the model of linearly elastic, homogenous semi-space, and one of the design methods based on this model is the finite difference model. It reduces the problem defined by the differential equation and the set boundary conditions to the system of algebraic equations. In solving the problem, Stevanović [9] starts from the differential equation of the elastic line of the diaphragm having constant cross-section, that is, from the known differential dependences of the arbitrary supported and loaded beam:

$$\frac{dx}{dz} = tg\varphi \approx \varphi \qquad \frac{d^2x}{dz^2} = -\frac{M}{D} \qquad \frac{d^3x}{dz^3} = -\frac{T}{D} \quad \text{and eventually} \quad \frac{d^4x}{dz^4} = \frac{p_z}{D} - \frac{r_z}{D} \tag{16}$$

where: $D = E\,/\,(1 - \nu^2)$ – is the stiffness of the support to bending, E – is the modulus of elasticity of diaphragm material, I – is the moment of inertia of the diaphragm cross-section, ν – is the Poisson coefficient of the diaphragm material, p_z – external active load, r_z – reactive load (base soil resistance) according to the diaphragm depth.

Differential relations in the differential equation (16) are replaced by the appropriate differential quotients so that the system of algebraic equations is obtained, which can be briefly presented in the matrix form:

$$Ax = \frac{c^4}{D}(p - r) \tag{17}$$

where:

A – is the square matrix of the order $n+1$ (the matrix terms are the coefficients of the equation system)
$x = [x_0\ x_1 ... x_n]$ – is the ordinate vector of the diaphragm elastic line
$p = [p_0\ p_1 ... p_n]$ – is the vector of the external load by diaphragm depth
$r = [r_0\ r_1 ... r_n]$ – is the vector of the base layer resistance by diaphragm depth.

The external load, represented by the vector p, in a great number of cases is reduced only to the load on the top end of diaphragm M_0 and H_0. When the excessive unknowns are eliminated, by using the contour conditions on the top end, the replacement active loads are generated which correspond to the influences $M = M_0$ and $H = H_0$, so that the first two elements of the vector p are:

$$p_0 = -\frac{2M_0}{c^2} - \frac{2H_0}{c} \quad \text{and} \quad p_1 = \frac{M_0}{c^2} \tag{18}$$

where M_0 and H_0 are the set moment and transversal force in the zero node, which is the top of the diaphragm, and c is the length of the diaphragm division section.

The elements of the vector p are different from zero, in case that the diaphragm is anchored. Then the members of the vector p which correspond to the location of diaphragm anchoring are different from zero, and they are calculated using the procedure described for the beam on elastic foundation [9].

In the matrix equation, the vectors x and r remain unknown. In order to solve it, an additional equation is required, defining relation between these two vectors using the influence functions. The final form of the additional equation according to [9] is:

$$x = \frac{(1+\nu_0)c}{8(1-\nu_0)\pi \cdot E_0} F \cdot r \tag{19}$$

where F is the matrix of the relations between the soil pressures (r) at the points of division and the horizontal displacement (x) of those points (i.e. of the ordinates of the elastic line in those points). Here E_0 and ν_0 are the elasticity model and the Poisson base coefficient. An arbitrary element of the matrix F_{ki} represents the horizontal displacement of the point k when unit intensity uniform pressure is transmitted through the i-th sections the. The matrix F can be made assuming that the soil is a homogenous, elastic and isotropic semi-space and implementing solutions of the linear elasticity theory. The solution of the planar problem (assumption that the diaphragm is in the planar deformation state) was provided by Melan in 1932. Gorbunov-Posadov corrected the observed errors in the Melan's solution and in 1954 provided the definite expressions for component stresses and displacements of semi-space points.

The numerical values of the elements of matrix F for the division of the diaphragm in ten sections and the Poisson coefficient $\nu_0 = 0.3$ are provided in [12].

By the replacement of expression (19) in (17) the unknown soil resistance vector r can be calculated also in elementary transformations, and by the replacement in (19) the elastic diaphragm line displacement vector x.

The bending and transversal force moments in the arbitrary point of the diaphragm k can be calculated using the finite difference method using the well-known relations between these parameters and the calculated vector x, when the corresponding differential relations are replaced by the differential quotients:

$$M_k = -D\frac{d^2x}{dz^2} = -D\frac{x_{k-1} - x_k + x_{k+1}}{c^2}$$

$$T_k = -D\frac{d^3x}{dz^3} = -D\frac{x_{k-2} - 2x_{k-1} 2x_{k+1} + x_{k+2}}{2c^3} \tag{20}$$

7. DIAPHRAGM DESIGN USING PLAXIS SOFTWARE

This software package allows soil representation by the simple, as well as complex soil models, and here will be represented the Mohr-Coulomb model [12]. It belongs to ideally elastoplastic models, which were the first, in the historical perspective, to introduce the plastic properties into the description of material behavior. In the models, no working strengthening or softening is present, so all the yielding surfaces in the spatial stress state are blended into one – final failure plane. Inside the failure plane, the material behaves in a linear-elastic way, and when the stress state is located on the yield plane, what occurs is the plastic deformation whose direction and magnitude are determined on the basis of the potential function and failure conditions. These models are described by the classical failure theories (Tresca, Von Mises, Drucker-Prager and Mohr-Coulomb) and the approximate the curved failure envelope using bodies whose generatrices are straight lines. The intersection of the octahedron plane and the yield plane in Mohr-Coulomb model is hexagon, and circle in the Drucker–Prager model (Figure 2).

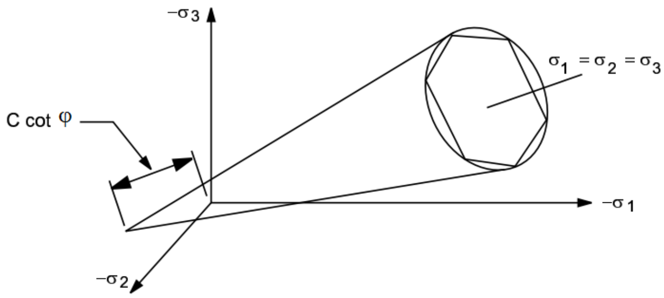

Fig. 2 Mohr-Coulomb and Drucker–Prager yield planes

The Mohr-Coulomb model is an improvement of the von Mises yield criterion because the yield plane is dependent on the stress hydrostatic component. In figure 2 it can be seen that the higher hydrostatic stress means the higher yield boundary. It is particularly suitable for presentation of the granular materials behavior, including soil.

The input parameters of this model of soil in PLAXIS are: soil density (γ), soil modulus of elasticity (E_s), Poisson coefficient (v), cohesion (c), shear resistance angle (φ) and dilatancy angle (ψ).

8. DESIGN EXAMPLE

For the sake of comparison of the described design methods, an example of the reinforced-concrete diaphragm was analyzed, having thickness 0.5 m, depth 7.5 m, modulus of elasticity $E_b = 2.10^7$ kN/m^2 and moment of inertia I_d = 0.0101 m^4.

The diaphragm is loaded on the top end with the horizontal force H = 90.3 kN and bending moment M = 163.8 kNm. The soil is constituted by three layers of 2.5 m whose moduli of elasticity are, respectively: $E_{01} = 1.10^4$ kN/m^2, $E_{02} = 2.10^4$ kN/m^2 and $E_{03} = 3.10^4$ kN/m^2.

The diaphragm and the surrounding soil are firstly treated using the initial parameter method with an assumption of the linearly increasing soil reaction coefficient, then using the finite difference method and by application of the STRESS, TOWER and PLAXIS software packages. For comparison of the used calculation methods, the bending moments and horizontal displacements of the diaphragm top were adopted.

When solving the initial parameter method, the adopted soil characteristic m is according to [6], an average value for three layers of $m = 6000$ kN/m^4 and on this basis was obtained $\alpha = 5.0$ i.e. $\alpha \cdot h = 3.5 > 2.5$ which means that the diaphragm has the final stiffness. For these input data and the adopted assumption of the linearly increasing soil reaction coefficient k_H, the maximum moment of 268.78kNm was obtained, as well as the diaphragm top displacement of 13.6mm.

For the purpose of applying the finite difference method and the model of homogenous isotropic continuum, the diaphragm was divided in ten sections having the length $c = 7.5/10 = 0.75$m. An average the soil characteristic was adopted to be $E_0 = 2.10^4$ kN/m^2 and $\nu_0 = 0.3$. The maximum moment of 324.1 kNm and displacement of 8.8 mm were obtained.

When implementing the STRESS software, the average adopted value for the soil characteristic was $E_0 = 2.10^4$ kN/m^2, by the diaphragm depth, i.e. the corresponding constant soil reaction coefficient k_H obtained according to the expression (6). For the input data adopted in this manner, and the division of the diaphragm in 10 segments the maximum moment of 213.83 kNm and horizontal displacement of the diaphragm top of 8.6 mm were obtained.

When applying the TOWER software, the soil is modeled using the linear support discretized in 15 segments of 0.5 m, whereby each segment is assigned the reaction coefficient calculated in the same way as in the STRESS software. The maximum moment of 209.44 kNm and horizontal displacement of the diaphragm top of 8.26 mm were obtained.

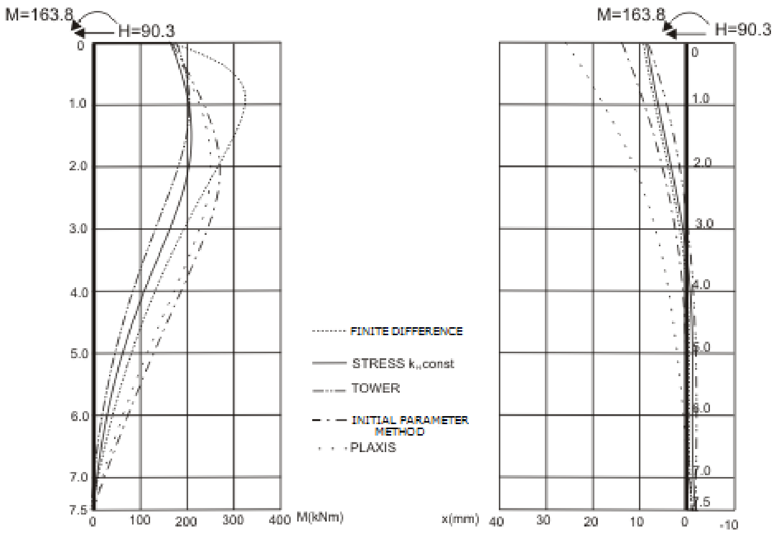

Fig. 3 Comparative diagram of bending moments and horizontal displacements of the diaphragm

9. CONCLUSION

There are notable differences in the results, both resulting from the adopted soil model and from the implementation of various methods. It is notable, that in accordance with the conclusions in [7], the differences are more pronounced in terms of the displacements than the bending moments of the diaphragms. It is observable that the best agreement was achieved for the results obtained according to the STRESS and TOWER software where a constant coefficient of the soil reaction with the depth is assumed. It could be expected because both methods are based on the Winkler soil model, and the same distribution of the soil reaction coefficient with depth was adopted. Good agreement of this soil model with the results of the field measurements could be found in the papers [4] and [5], which is very important, having in mind the large distribution of implementation of the Winkler model in the practical calculations.

In figure 3 one may also observe a very good agreement of the bending moment diagrams obtained by the initial parameter method and of a considerably more sophisticated calculation obtained through implementation of the PLAXIS software. If the obtained horizontal displacements are compared, significant differences can be observed, but still the initial parameter method is the closest to the result obtained with PLAXIS. The highest displacements were obtained by the implementation of this software and the Mohr-Coulomb soil model which is to be expected because that is the only elastoplastic model in relation to the remaining linearly elastic models.

As it was already mentioned, the usage of the initial parameter method, based on the soil characteristic *m* is advantageous for its relatively simple calculation and the potential of not too precise adapting of the *m* characteristic since it does not linearly affect the calculation results. On the other hand, according to [7], the obtained results show higher deviations in comparison to the data measured in the field in respect to the finite difference method and application of the STRESS software. Also the mentioned method provides a possibility to adopt only constant or only linearly increasing soil reaction coefficient with depth, while in reality, the mechanical characteristics of the soil can considerably vary with depth, so it is not a rare case that their value decreases in some deeper layers.

The finite difference method has a potential for usage of several soil models and of varying the soil characteristics by depth, which results in a more complex calculation. Apart from that experience in [4] show that for the convergence of the results, very "fine" division of 100 elements is required. For such a division, the Winkler soil model and the coefficient of the horizontal soil reaction adopted on the basis of the static soil penetration, the following results are obtained, which for the working load values are in very good agreement with the actual measured values.

Usage of the STRESS software is simple and quick, and offers potential to vary the soil characteristics by depth, and the obtained results, according to the experience of the authors of this paper [7] and for the working values of the loading, are very close to the measured values. In the process were used the coefficients of the horizontal soil reaction obtained on the basis of the diagrams of static soil penetration of the location in question according to [7]. Also, as opposed to other design methods, here the structure of the entire foundations (or entire building) can be modeled, which provides the possibility to take into consideration the foundations and soil interaction.

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UPOREDNA ANALIZA METODA PRORAČUNA POPREČNO OPTEREĆENIH DIJAFRAGMI

Armiranobetonske dijafragme su potporne konstrukcije građene direktno u tlu. Namena im je da prihvate poprečne pritiske tla, i posmatrajući odnos debljine i visine dijafragme pripadaju grupi deformabilnih konstrukcija. U ovom radu su dati različite metode proračuna poprečno opterećenih dijafragmi kao i modeli tla koji se mogu primeniti za ovu problematiku. Za poređenje opisanih metoda dat je i jedan primer proračuna armiranobetonskih dijafragmi, kao i analiza dobijenih rezultata proračuna. Najpre je dat proračun dijafragme klasičnim postupkom proračuna, a potom i numeričke metode proračuna zasnovane na diskretizaciji problema koristeći metod konačnih razlika i STRESS, TOWER i PLAXIS softvere. Cilj ovog rada je određivanje što tačnijeg načina proračuna i predviđanja ponašanja dijafragme i okolnog tla, kao i nalaženje relevantnih uticaja potrebnih za dimenzionisanje.

Ključne reči: *Dijafragma, tlo, poprečno opterećenje, interakcija, modeliranje, diskretizacija*