Studia Geotechnica et Mechanica, Vol. XXVI, No. 3–4, 2004

A NUMERICAL MODEL FOR THE SOFT SUBSOIL – LOADBEARING CUSHION SYSTEM

JERZY SĘKOWSKI

Silesian Technical University, Chair of Geotechnics,
ul. Akademicka 7, 44-101 Gliwice, Poland.

Streszczenie: Jednym z zabiegów poprawiających warunki posadowienia obiektów budowlanych i inżynierskich są poduszki wzmacniające. Autor przedstawił własną propozycję modelu obliczeniowego dla układu słabe podłoże–poduszka wzmacniająca opartą na założeniu, że do opisania właściwości poduszki i podłoża należy wykorzystać ciała sprężysto-idealnie plastyczne, których prawa płynięcia plastycznego są stowarzyszone z warunkiem Coulomba–Mohra. Taki dwustrefowy model oznaczono symbolem CM/CM. Propozycja była wynikiem określonej procedury postępowania i obejmowała przesłanki teoretyczne i doświadczalne, studia parametryczne MES i badania weryfikacyjne (te ostatnie będą przedmiotem oddzielnego artykułu).

Abstract: One of the methods of improving soil under foundations of building and engineering structures is a loadbearing cushion. In this paper, the author presents his own numerical model for the soft subsoil–loadbearing cushion system. It is based on the assumption that elastic-perfectly plastic bodies and a plastic flow rule associated with Coulomb–Mohr's yield condition can be used for describing the properties of the cushion and the subsoil. Such a two-zone system is designated as the CM/CM. This concept is a result of a specific procedure, which comprised theoretical and empirical premises, FEM parametric studies and verification (a separate paper will be devoted to the latter).

Резюме: La méthod des coussin renforcés c'est une de méthods qui ameliore les conditions de la fondation des bâtiments et des ouvrages d'art. L'autheur a présenté sa proposition d'un modèle numérique pour le système sol faible – coussin renforcé. Cette conception profit des corps élasto – idéal plastiques dont les lois de la fluence plastique sont associés avec les conditions de Coulomb–Mohr. On a nommé ce modèle de deux-zones comme CM/MC. La proposition presentée c'est le résultat de la procédure déterminée contenant les premisses théoriques et expérimentales, les études paramétriques FEM et la validation expérimentale dont les résultats seront discutés dans une autre article.

1. INTRODUCTION

Cases, where a structure cannot be founded directly, or where excessive costs of such foundation concept must be taken into account, are symptomatic of soft subsoil.

One of the methods of improving the soft subsoil is to replace it with a finite layer of the stiffer soil (weak subsoil replacement), to form the so-called *loadbearing cushion.* Its dimensions are close to those of the structure.

The procedure of loadbearing cushion designing includes: the selection of the replacement material and the technology of its formation as well as determination of the dimensions of loadbearing cushion, i.e. its height and width at the base (*Hp*, *Bp*).

The requirements for the material and cushion formation technology, the principles of performing control tests and the criteria for assessing cushion quality do not cause any major controversies. However, we can raise objections to cushion dimensioning methods (SĘKOWSKI [26]). After brief reference to the existing principles of the loadbearing cushions dimensioning, the author will present his own numerical model for the soft subsoil–loadbearing cushion system.

2. DIMENSIONING OF LOADBEARING CUSHIONS – CURRENT METHODS

2.1. DIMENSIONING CRITERIA AND NUMERICAL MODELS

The dimensioning of a loadbearing cushion is a procedure for determining its height (*Hp*) and width at the base (*Bp*). An appropriate selection of both values depends on the limit state conditions, i.e. bearing capacity and displacement conditions. In practice, this means that the values of *Hp* and *Bp* should be such that they exceeded neither the bearing capacity of the layer directly under the cushion nor the bearing capacity of the cushion itself and the foundation settlement should not be greater than the values permitted for the founded structure. These conditions, *the selection criteria for loadbearing cushion dimensions*, have the following form:

- bearing capacity criterion

$$Q_r < m \cdot Q_{fn} \, , \tag{1}$$

- displacement criterion

$$s < s_p \, . \tag{2}$$

Consequently, the principles for determining the capacity of subsoil bearing and its displacements, and specifically their theoretical basis are of importance. The numerical methods applied in practice so far include: *a rigid-perfectly plastic model* and *a linear elastic model.*

For the bearing capacity of the "cushion–subsoil" system, either simple limit equilibrium methods combined with empirical corrections that take into account the layering of subsoil, or the methods of the limit analysis of the layered subsoil are employed (FLORKIEWICZ [10]; ŁĘCKI and FLORKIEWICZ [18]). Recently, for limit analyses general numerical procedures, such as Gussman's kinematic element method [15], have been applied.

So far, the linear elastic model has been used for analysing displacements of the "cushion–subsoil" system. It allows the use of widespread *strain and stress methods* corresponding to triaxial and uniaxial strain states, respectively. However, in these methods, the heterogeneity of the layered soil is treated in a very simplified manner. Namely, for the layers with different E_{0i}, E_i deformation moduli, the solutions for linear elastic and homogeneous half-space equilibrium problems are used.

Heterogeneity of layered subsoil was given a detailed consideration solely in the more advanced linear elastic analyses of the "cushion–subsoil" system (GRYCZMAŃSKI and FEDYNYSZYN [13]; GRYCZMAŃSKI and SKIBNIEWSKA [14]; GRYCZMAŃSKI [12]). As a consequence, the method of finite elements had to be used.

2.2. CURRENT CUSHION DIMENSIONING PRACTICES

The existing cushion dimensioning practices are based on the two following assumptions:

1. The bearing capacity or admissible stress in soft soil below the base of the cushion cannot be exceeded and the height *Hp* is determined accordingly. The condition of admissible settlement, although obvious, is treated as secondary.

2. The width of the cushion at the base (*Bp*) is determined by using an arbitrarily evaluated angle of stress distribution β in the subsoil (figure 1).

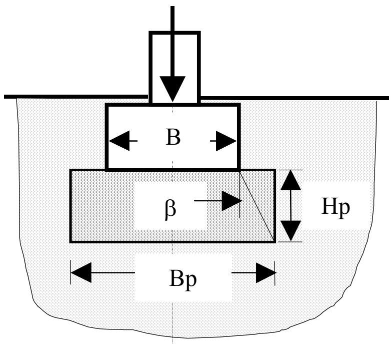


Fig. l. Calculation scheme

The variety of cushion dimensioning methods results, primarily, from different values of the angle β (0–45°) adopted by different researchers, and secondly, from different interpretations of the bearing capacity condition. As a consequence, over the last forty years, we have dealt with more than ten cushion dimensioning concepts. Some of them are represented by nomographs, tables or charts, which substantially facilitate the design process (e.g. BRYL et al. [4]; BIERNATOWSKI [2]; GLINICKI [11]).

The behaviour of subsoil strengthened with a gravel and sand cushion has also been the subject of a large-scale model testing in a laboratory (among other TOCHKOV [29], KEZDI [16], SZECHY [27], CICHY and ODROBIŃSKI [7] as well as SĘKOWSKI [23]–[25]).

Oriented numerical analyses of loadbearing cushions have also been conducted. In the majority of them, the finite element method has been employed. The analyses have been carried out first of all in order to determine the distributions of displacements and stresses in subsoil improved with a cushion or a loadbearing layer. They fall into three categories: the analyses based on the linear elastic theory of heterogeneous layered bodies (GRYCZMAŃSKI and FEDYNYSZYN [13]; GRYCZMAŃSKI and SKIBNIEWSKA [14]; GRYCZMAŃSKI [12]), those based on the non-linear elastic theory (MITCHELL and GARDNER [19]) as well as elastoplastic analyses (SĘKOWSKI [24], BRZĄKAŁA and NGUYEN [5]).

To summarise, the results of the experimental research and theoretical analyses dealing with the basics of cushion dimensioning as well as the aforementioned different designing methods prove that:

- the applicability of the bearing capacity and settlement analysis methods used so far for cushion dimensioning is limited,
- the use of different generations of elastoplastic models for describing the soft subsoil–loadbearing cushion system is substantiated.

These two conclusions confirm the necessity for developing a rational theoretical basis for cushion dimensioning which would reflect the current theory of soil mechanics. They also indicate a general direction of the search for such a model.

Thus, the basis should be a numerical model consisting of two bodies with different elastoplastic properties: a cubicoidal cushion, which is symmetrical to the footing, and whose dimensions are *Lp*, *Bp*, *Hp*, and the surrounding soft subsoil whose external dimensions are large enough compared to the foundation footing.

Moreover, the model should be reasonably simple. This channels the search for the model that describes cushion and subsoil properties into the use of elastic-perfectly plastic bodies with plastic flow rule associated with Coulomb–Mohr's yield condition. In the author's opinion, such a two-layer model is a numerical implement by which the realistic assessment of foundation settlement within a wide range of load can be ensured. The model is further designated as the CM/CM.

3. CONCEPT AND SELECTION OF THE "SOFT SUBSOIL–LOADBEARING CUSHION" SYSTEM MODEL

3.1. A CONCEPT OF THE SOFT SUBSOIL–LOADBEARING CUSHION SYSTEM MODEL

The author believes that the following procedure is optimal for developing a rational numerical model for the foundation–cushion–soft subsoil system:

• pre-selection of a set of the models examined, based on general theoretical and experimental premises,

• FEM parametric studies of representative characteristics of the models selected, preferably the foundation's "loading–settlement" relationship in a broad range of load,

• selection of the rational model, whose representative characteristic exhibits the best consistency with experimental data, provided that optimal parameter estimates have been ensured,

• comparison of the resultant set of numerical characteristics with corresponding relationships obtained experimentally.

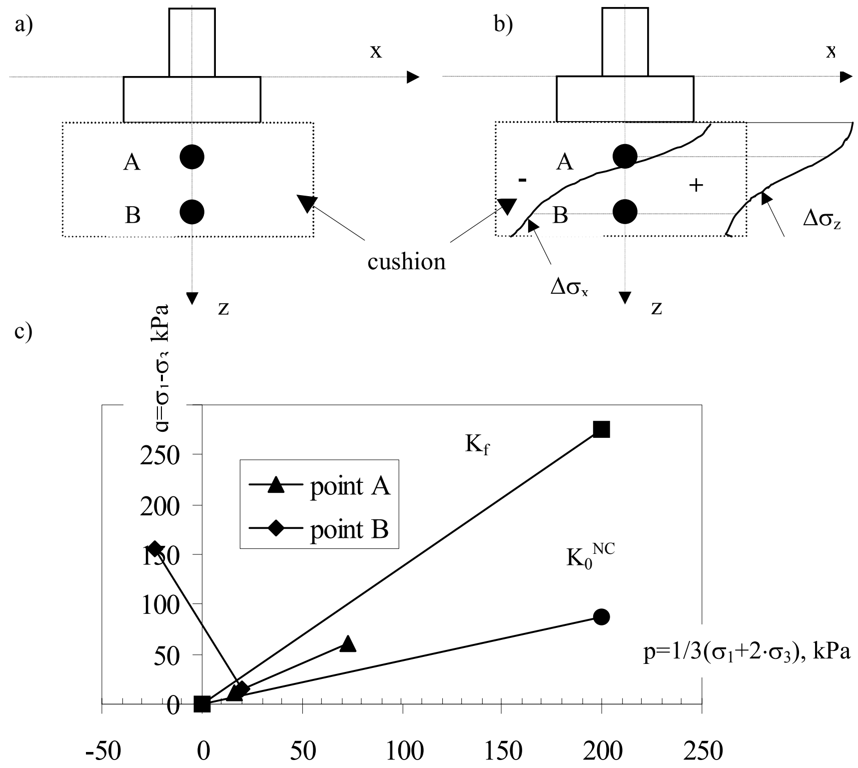


Fig. 2. Theoretical premises allowing pre-selecting the model of soft subsoil–loadbearing cushion system

When estimating this procedure, the key issue is the lack of a sufficient database with data acquired from model testing in laboratory and in the field scales as well as the lack of measurements of actual settlements for building structures. Therefore, when selecting a rational numerical model, it would be easier to concentrate on the "load–settlement" theoretical characteristics for a model arbitrarily found to be appropriate for this role. This base model should be more sophisticated than the schemes examined. Results of experimental studies stored in the experimental database could be progressively compared with conclusions from theoretical studies. Eventually, such procedure has been adopted. After pre-selection of the models examined and parametric studies of their characteristics, a numerical model of soft subsoil–loadbearing cushion was selected. This model was then verified on the basis of "load–settlement" relationships obtained by the author in laboratory tests and in field investigations. Note that the paper does not detail any results of the last stage of the procedure presented.

Theoretical premises that allow pre-selection are individual properties of the stress paths in the system analysed, especially in the lower part of the cushion, which is illustrated by the example of points *A* and *B* of the cushion (figure 2a).

The linear elastic analysis provides the distributions of stress increments caused by the load of the structure shown in figure 2b. Its characteristic feature is substantial horizontal stretching in the bottom layer of the cushion. Corresponding stress paths are shown in figure 2c. The positions of their starting points (in situ stress states) in the *p*, *q* system result from preconsolidation of the cushion material by means of mechanical compaction (impact or vibration). It is clear that the path in the point *B*, which represents the bottom layer, soon reaches the limit state line. The lines in figure 2 show stress paths for points *A* and *B* of the sand cushion ($Hp = 0.6\ B$ and $Bp = 1.2\ B$) at the depths of 1.2 and 1.5 m, respectively. The cushion was formed under a strip foundation whose width *B* reaches 1.0 m and which is founded at the depth $D = 1$ m and loaded with the pressure $q^* = 100$ kPa (see GRYCZMAŃSKI [12]).

From the above considerations the following conclusions can be drawn:

1. Application of the linear elastic model to a cushion and subsoil (LE/LE) is limited, especially to the cushion.

2. The elastic-perfectly plastic model with Coulomb–Mohr's yield surface fairly adequately describes the behaviour of granular material, at least in the bottom layer of the cushion.

Weak, soft subsoil, however, should be modelled in such a way as to take account of its non-linearity and the effect of loading history before reaching the limit state. The simplest models to meet these requirements are critical state models, e.g. Modified Cam Clay model (MCC).

Consequently, the discrete CM/MCC model (Coulomb–Mohr's model for the cushion and Modified Cam Clay for the subsoil) has been arbitrarily selected for the base model.

It will be interesting to analyse the applicability of simple models, i.e. LE/LE, LE/CM and CM/CM in terms of the base model. This can be done by comparing the "load-settlement" curves that represent these models with the corresponding characteristic of the base model. This relationship is selected as a basis for comparison and analyses because it represents the overall response of the soil to the load applied.

3.2. CHARACTERISTICS OF NUMERICAL MODELS SELECTED FOR THE SYSTEM DESCRIPTION

Three soil models have been chosen for further reasoning which includes the selection of a numerical model for the loadbearing cushion–soft subsoil system. They are: the linear elastic model (LE), the elastic-perfectly plastic model with Coulomb–Mohr's yield surface (CM) and the elastic-plastic Modified Cam Clay isotropic hardening model (MCC). Each of them will be briefly presented below.

3.2.1. LINEAR ELASTIC MODEL

Adapting this model means that a given layer (a cushion, subsoil) is treated as a solid, continuous, isotropic and homogeneous body, and the constitutive equation is as follows:

$$\boldsymbol{\sigma} = \mathbf{D} \cdot \boldsymbol{\varepsilon}, \tag{3}$$

where $\boldsymbol{\sigma}$, $\boldsymbol{\varepsilon}$ are the current vectors of stress and strain, and **D** is an elastic matrix.

The elastic matrix **D** for isotropic media is defined as:

$$\mathbf{D} = \frac{E}{(1+\nu)\cdot(1-2\cdot\nu)} \begin{bmatrix} 1-\nu & \nu & \nu & 0 & 0 & 0 \\ & 1-\nu & \nu & 0 & 0 & 0 \\ & & 1-\nu & 0 & 0 & 0 \\ & \text{(sym)} & & \frac{(1-2\nu)}{2} & 0 & 0 \\ & & & & \frac{(1-2\nu)}{2} & 0 \\ & & & & & \frac{(1-2\nu)}{2} \end{bmatrix}. \tag{4}$$

Equation (4) and its characteristic parameters, namely, elastic modulus E and Poisson's ratio ν, are commonly used.

3.2.2. COULOMB–MOHR MODEL

The equation representing this model's yield surface, identical with the limit state surface, has the following form in the p', q', Θ system:

$$F(q', p', \Theta) = p' \cdot \sin\phi - \frac{1}{3} \cdot q' \cdot (\sqrt{3} \cdot \cos\Theta + \sin\Theta \cdot \sin\phi) + c \cdot \cos\phi = 0\,. \tag{5}$$

In these equations, p', q', 0 are effective stress state invariants, whereas ϕ, c are the model's parameters.

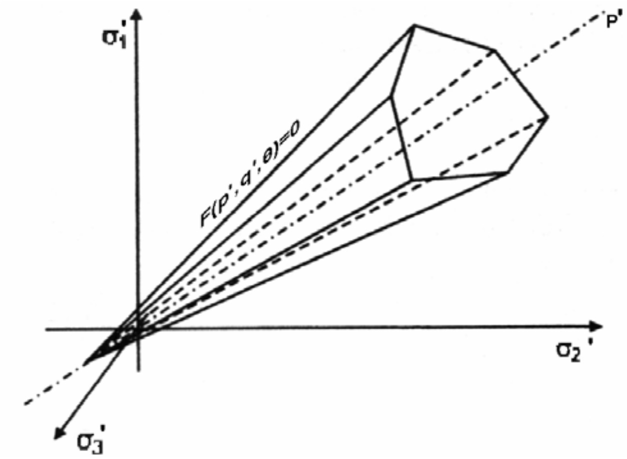


Fig. 3. Yield surface for Coulomb–Mohr's model

The yield surface that occupies a constant position in the effective principal stress space σ_1', σ_2', σ_3' is illustrated by a hexagonal, equilateral, but non-equiangular, pyramid, whose central axis is the hydrostatic axis (figure 3).

A general form of the constitutive relationship, i.e. the relationship between stress and strain increments, is as follows:

$$\delta\boldsymbol{\sigma}' = \mathbf{D}^{ep} \cdot \delta\boldsymbol{\varepsilon}, \tag{6}$$

where $\mathbf{D}^{ep}$ denotes the elastic-plastic matrix

$$\mathbf{D}^{ep} = \mathbf{D} - \frac{\left(\mathbf{D} \cdot \frac{\delta f}{\delta \boldsymbol{\sigma}'}\right) \cdot \left[\frac{\delta f}{\delta \boldsymbol{\sigma}'}\right]^T \cdot \mathbf{D}}{\left[\frac{\delta f}{\delta \boldsymbol{\sigma}'}\right] \cdot \mathbf{D} \cdot \frac{\delta f}{\delta \boldsymbol{\sigma}'}}, \tag{7}$$

whereas **D** stands for the isotropic elastic matrix defined by the matrix equation (4).

It is assumed that the flow rule for this model, is associated with Coulomb–Mohr's yield condition $\{F(\delta) = 0\}$. Therefore, Coulomb–Mohr's elastic-perfectly plastic model is characterised by four material parameters, i.e. elastic modulus E, Poisson's ratio v, angle of internal friction ϕ and cohesion c. They are obtained in laboratory tests, e.g. in the triaxial compression apparatus, or in field studies, e.g. trial loading tests, cone penetration tests and dilatometer tests (ZADROGA [30]; TSCHUSCHKE [29]; LECHOWICZ and RABARIJOELY [17]).

3.2.3. MODIFIED CAM CLAY MODEL

This model represents a generation of isotropic hardening models. It is commonly considered a basic model for the critical state theory.

The description of the Modified Cam Clay model is based on the following postulates:

• The yield surface is an ellipsoid of revolution presented in the principal stress space in figure 4. It is given by the following equation:

$$F = q^2 + M^2 \cdot p' \cdot (p' - p'_c) = 0\,, \tag{8}$$

where p', q, p_c denote respectively: mean stress, stress intensity and preconsolidation pressure (parameter M is defined later).

• The isotropic hardening parameter is the total plastic change of the Δe^p void ratio according to the following equation:

$$p'_c = p'_{c0} \cdot \exp\left(\frac{\Delta e^p}{\lambda - \kappa}\right). \tag{9}$$

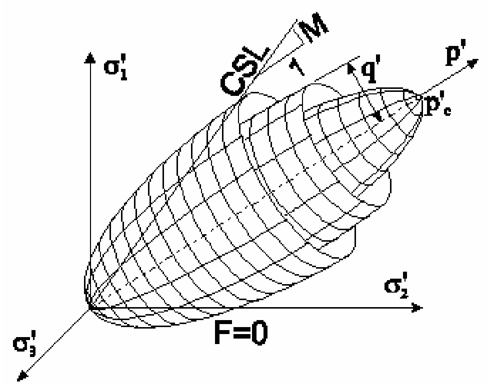


Fig. 4. MCC model yield surface

• A general form of incremental "stress–strain" characteristics is as follows:

$$\delta\boldsymbol{\sigma} = \mathbf{D}^{ep} \cdot \delta\boldsymbol{\varepsilon},$$

where (10)

$$\mathbf{D}^{ep} = \mathbf{D} - \frac{(\mathbf{D}\cdot\mathbf{n}_F)\cdot(\mathbf{n}^T{}_F \cdot \mathbf{D})}{\mathbf{n}_F^T \cdot \mathbf{D}\cdot\mathbf{n}_F + \mathbf{K}_F}.$$

In the Modified Cam Clay model, we deal with five material parameters:

$$\lambda, \kappa, M, \Gamma, G(\nu)\,. \tag{11}$$

These parameters quantify material functions defining the model itself, and their physical interpretation is shown in figure 5.

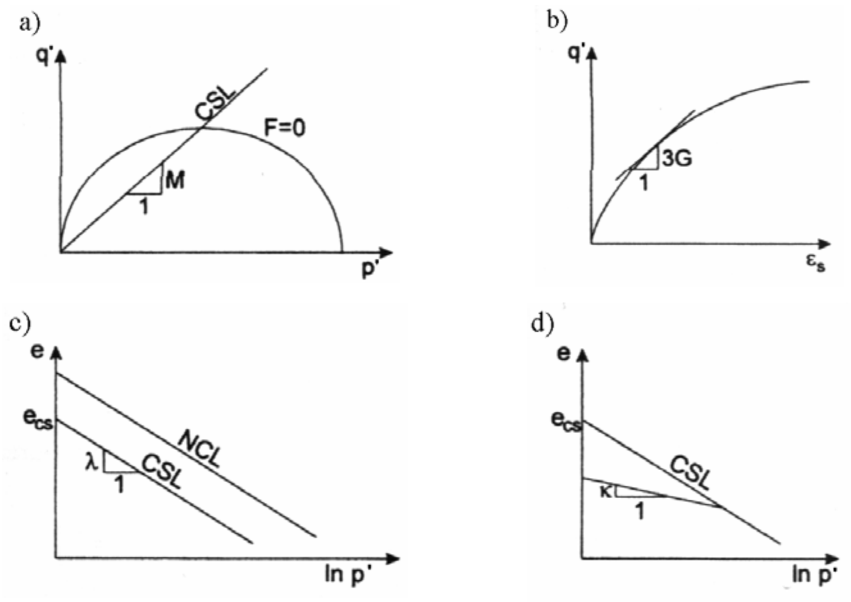


Fig. 5. Physical interpretation of Modified Cam Clay model parameters

M, λ and κ are, respectively, interpreted as: the slope of critical state line (CSL) in the system of p', q invariants (figure 5a), the slope of a normal isotropic consolidation line (NCL) in the semi-logarithmic scale (figure 5c) and the isotropic swelling line (SL) in the same system (figure 5d). The fourth parameter is the shear elastic modulus G (figure 5b).

Further, e and e_0 denote, respectively, the void ratio and its initial value, p'_{co} – the initial precompression pressure value, ε_v and ε_s – volumetric and deviatoric strains (strain intensity).

3.3. NUMERICAL ANALYSIS OF THE APPLICABILITY OF SELECTED NUMERICAL MODELS

3.3.1. PRELIMINARY NOTES

Carefully planned 2D FEM analyses were the basis for evaluating the applicability of the selected numerical models in describing the behaviour of the soft subsoil –

loadbearing cushion system. In these analyses, real engineering structures founded on a very weak subsoil strengthened with a sand cushion were examined. This, despite practical objections, provided an opportunity for comprehensive analysis of the problem, including the parametric study of the soft subsoil constitutive model (section 3.3.2) and the parametric study of the cushion numerical model (section 3.3.3).

In both cases (the axisymmetric problem and the plane strain problem), theoretical "load–settlement" curves determined for the base model (CM/MCC) and other schemes (models) (LE/LE, LE/CM, CM/CM) have been compared. In the first case, the effect of weak subsoil (clay) parameters on the load–settlement relationship has been subjected to more extensive numerical analysis, while in the second case, such analysis has been carried out for the numerical model of the loadbearing cushion.

3.3.2. PARAMETRIC STUDY OF SOFT SUBSOIL CONSTITUTIVE MODEL

A reinforced concrete tank of the diameter *d* = 4.0 m was founded directly on a circular plate of the diameter *D* = 6.0 m at the depth h_f of 1.0 m. The soft subsoil was soft silty clay (*G*π, I_L = 0.75), and dense coarse sand (*Pr*, I_D = 0.7) was used as the strengthening material. The loadbearing cushion had the following dimensions: *Hp* = 0.5·*D* = 3.0 m and *Bp* = 1.5·*D* = 9.0 m. Load unit at the tank base equalled *q** = 250 kPa, which produced the unit pressure of *q** ≅ 110 kPa on the level of foundation footing. The scheme of the system analysed is presented in figure 6.

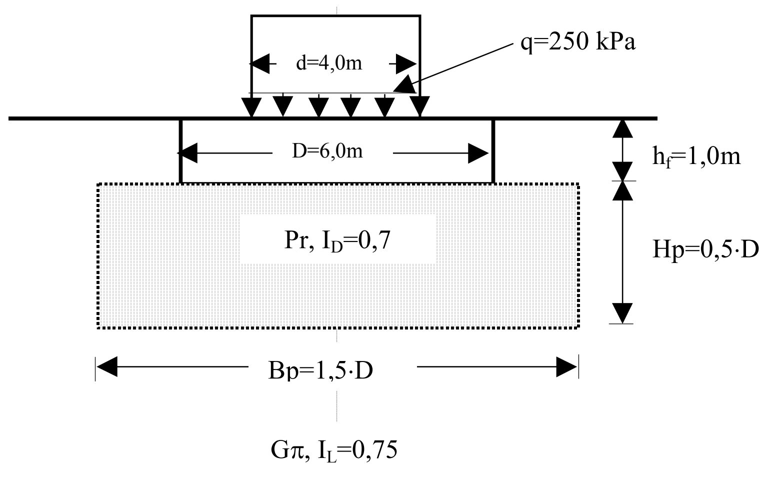


Fig. 6. Numerical scheme

The reinforced concrete foundation slab was represented by the linear elastic model with the parameters: E_b = 28500 MPa; v = 0.167. Silty clay in the base model for soft subsoil was represented by the elastoplastic isotropic Modified Cam Clay hardening model (MCC), and coarse sand cushion was represented by the elastic-perfectly plastic Coulomb–Mohr's model (CM). For the purpose of comparative analysis, also other schemes (LE, CM) were used to model both soils. Parameters of the models analysed are shown in table 1.

The discrete geometrical model of the system is shown in figure 7. The mesh consists of 198 rectangular eight-nodal elements arranged in the system of 11 rows and 18 columns. Its dimensions (53 m×18.5 m) related to the foundation width (B^*/D, H^*/D) are, respectively, 8.83 and 3.1.

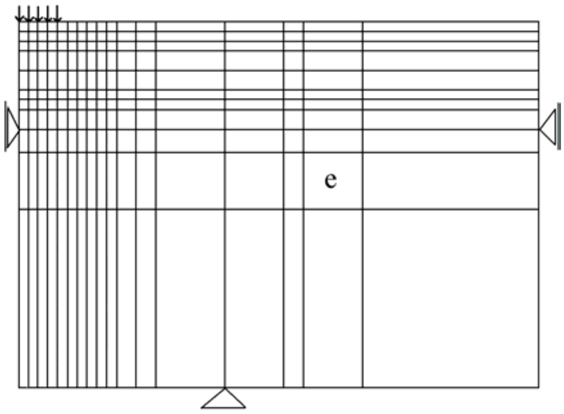


Fig. 7. Discrete model of the system

Table 1

Input soil parameters for the numerical models implemented

Soil model	Type of soil	
	Silty clay	Coarse sand
Linear elastic model (LE)	E v 6000 0.45	E v 130000 0.25
Elastic-perfectly plastic Coulomb–Mohr's model (CM)	E ϕ c v 6000 6 5 0.45	E ϕ c v 130000 34 0 0.25
Modified Cam Clay (MCC)	λ κ M $\Gamma-1$ v 0.046 0.021 0.217 0.5 0.45	
Important: E, c values are in [kPa], ϕ in [°], the other values are dimensionless		

In the case of the base model (CM/MCC), the "load–settlement" curve is considered to be the reference line in the analysis performed. Similar relations have been defined using the LE/LE; LE/CM and CM/CM models for the parameters given in table l. Further, the effect of each parameter in the simpler models (E, v in the LE model and E, v, ϕ, c in the CM model) on the "q–s" characteristics ($E^* = E/\alpha - \alpha = 2$, 3, 5, 10; $\phi = 3°, 4°, 5°$; c = l, 2, 3, 4, 5 [kPa]; v = 0.25; 0.35; 0.45) has been analysed. The changes in parameter values in the simpler models occurred only in a weak layer. The foundation model did not change. A package of FEM programmes, CRISP'93, was implemented in numerical analysis (BRITTO and GUNN [3]).

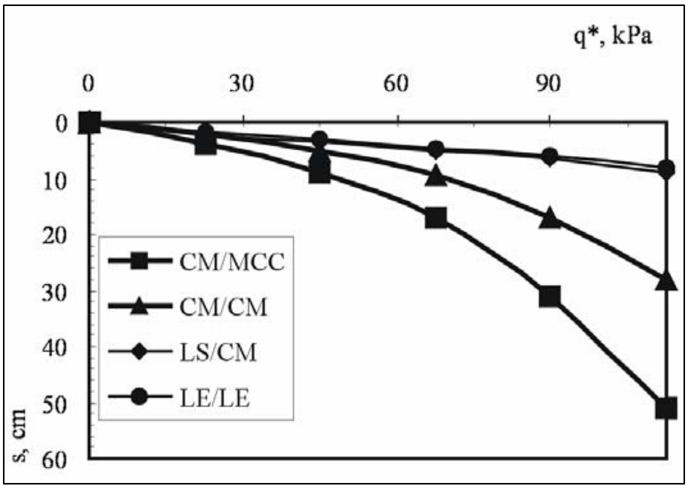


Fig. 8. "Load–settlement" characteristics for the schemes analysed

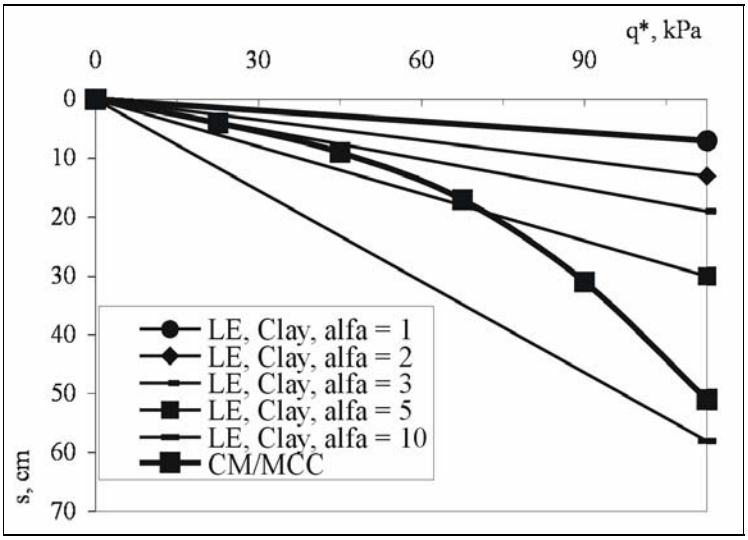


Fig. 9. The effect of the clay elastic modulus on the "load–settlement"

characteristics in the LE/LE scheme

Our calculations lead to the following conclusions:

• The effect of plasticity on the stress and strain state in the subsoil appears as the load is applied. Naturally, this effect is then transmitted to the displacement field as shown in figure 8. Here, the settlement of a point under the centre of the plate due to a unit load is presented for individual model combinations, including the base model. This impact is made progressively stronger along with an increase in unit loads. It needs to be emphasised that of all the simpler models analysed, the "load–settlement" relationship for the base model is most effectively approximated by the CM/CM scheme. This is particularly noticeable for the loads over 50 kPa.

• LE/LE scheme. The elastic modulus for the soft layer has a much greater effect on foundation settlement than Poisson's ratio, also used to describe a weak layer in the LE model. However, due to the different character of "load–settlement" characteristics for LE/LE and CM/MCC models, their relative agreement can be considered only within the range up to approx. 50 kPa (figure 9).

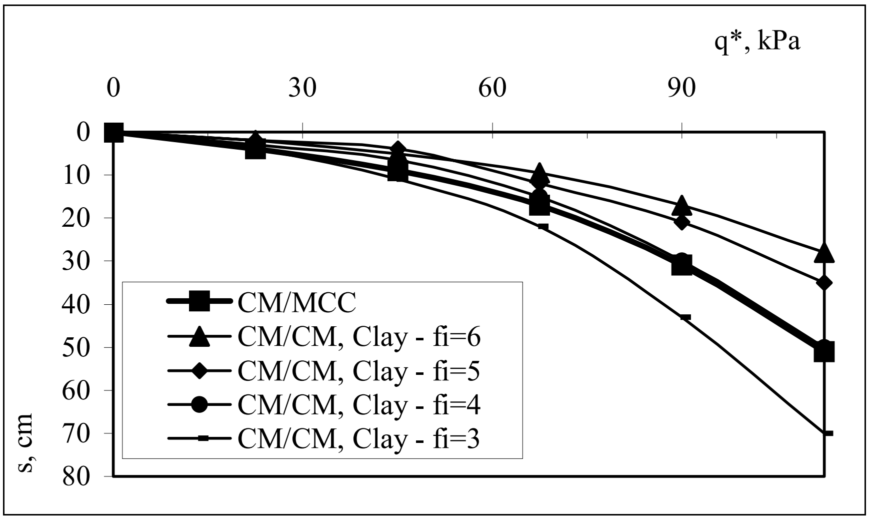


Fig. 10. The effect of an internal friction in clay on the "load–settlement" characteristics in the CM/CM scheme

• CM/CM scheme. The effect of the CM model parameters on the "load–settlement" characteristics is extremely important, particularly in the case of the modulus E and the angle of internal friction ϕ. This is demonstrated in figures 10 and 11. They show the effect of the clay parameters – the angle of internal friction (figure 10) and the elastic modulus (figure 11) – on the "load-settlement" characteristics in the CM/CM scheme compared with the "load–settlement" characteristic in the base scheme.

Figures 10 and 11 show that parameter values of the CM/CM model can be selected in such a way that the "1oad–settlement" relationships will produce results identical with, or very close to those obtained for the base model.

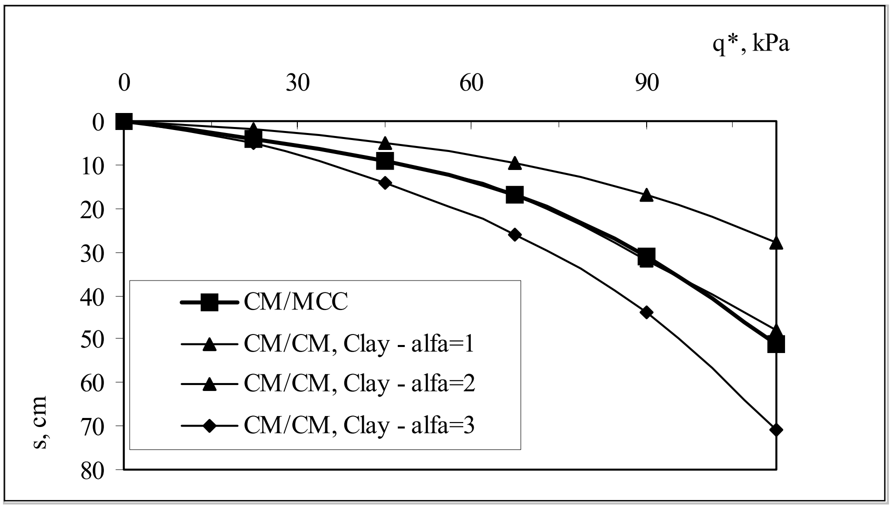


Fig. 11. The effect of the clay elastic modulus on the "load–settlement" characteristics in the CM/CM scheme

Regardless of the numerical model, the effect of Poisson's ratio on the characteristics in question is minor. It needs to be added that also the LE/CM model, with the cushion described by the linear elastic model and a subsoil – by the Coulomb–Mohr's model, did not furnish us with definite results.

3.3.3. PARAMETRIC STUDY OF THE CUSHION NUMERICAL MODEL

A strip foundation of the width B = 1.0 m, strengthened with a sand cushion, has been founded at the depth of D = 1.0 m below terrain level, directly on soft soil. In the analysis, different pairs of cushion dimensions, *Hp* and *Bp*, were introduced at different stages, as shown in table 2. Unit load on the subsoil was q^* = 50 kPa. Analogously to the previous analysis, the weak subsoil was soft silty clay ($G\pi$, I_L = 0.75) and for the loadbearing cushion, dense coarse sand (Pr, I_D = 0.7) was used.

The discrete geometric model of the system is illustrated by figure 12. The mesh consists of 198 rectangular eight-nodal elements in the system of 11 rows and 18 columns, and its dimensions related to the foundation's width are respectively: H^*/B =

7.45; B^*/B = 13.25.

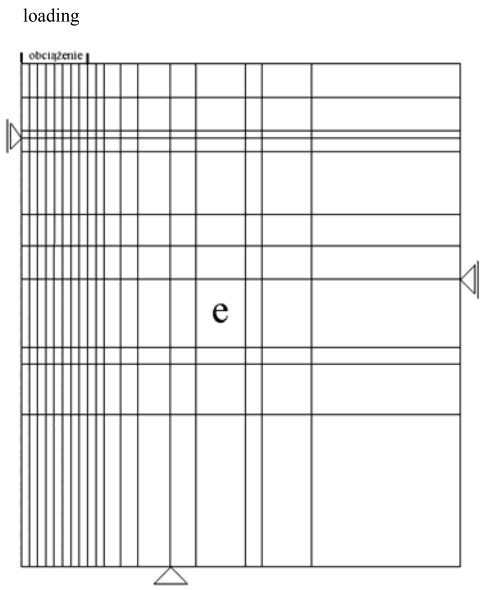


Fig. 12. The discrete model of the system

Table 2

Geometric characteristics of the strengthened cushion

Strip foundation width	Cushion dimensions	
	Hp/*B*	*Bp*/*B*
B = 1.0 m	0.5	1.0
	0.5	1.5
	0.5	3.0
	2.0	1.0
	2.0	1.5
	2.0	3.0

Also in this example, the base line for the comparative analysis was the "load–settlement" curve, determined numerically for the base model (CM/MCC). This relationship was also obtained for simpler models, i.e.: LE/LE; LE/CM and CM/CM, whose parameters were presented in table l. Next, the effect of each parameter in simpler models (E, v in the LE model and E, v, ϕ, c in the CM model) on the relationship "q–s" was analysed. This time, however, not only the parameters of the weak layer ($E^* = E/\alpha$ – α = 2, 3, 5, 10; ϕ = 2°, 4°; c = 0, 2, 4, [kPa]; v = 0.25; 0.35; 0.45), but also the parameters of the loadbearing cushion ($E^* = E/\alpha$ – α = 2, 3, 5, 10; ϕ = 30°, 20°; ν = 0.25; 0.35, 0.45) were being changed. For this reason and also due to variability of loadbearing cushion dimensions (table 2), the scope of the cases analysed was considerably wider.

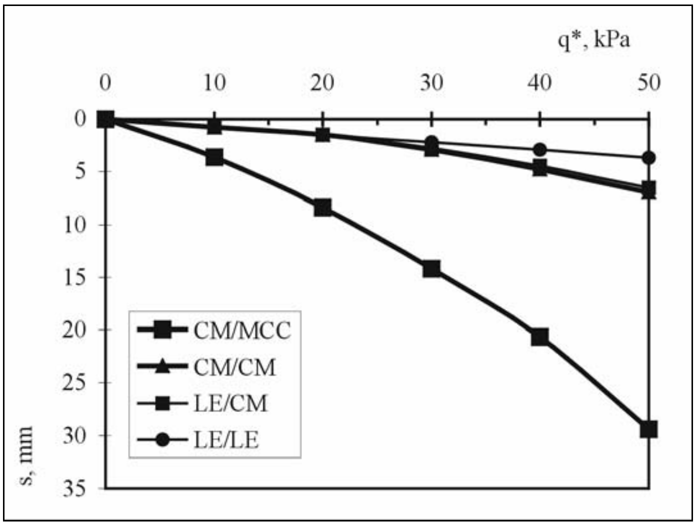


Fig. 13. "Load-settlement" characteristics for the schemes analysed: *Hp* = 0.5*B*; *Bp* = *B*

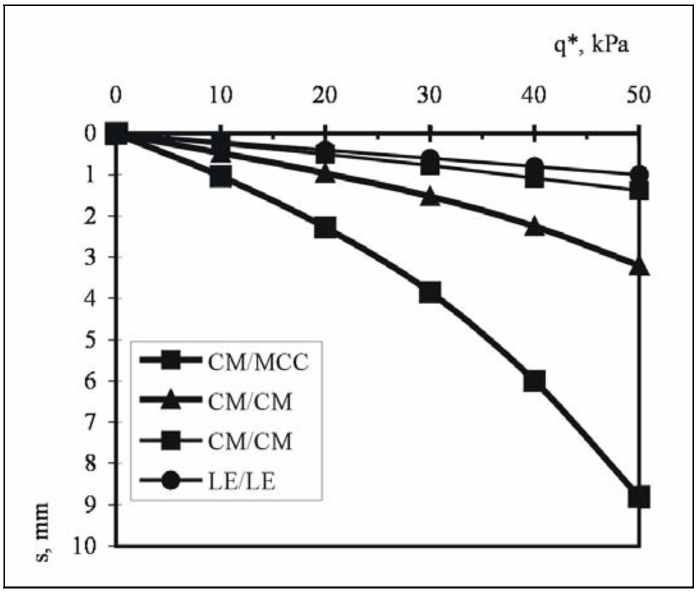
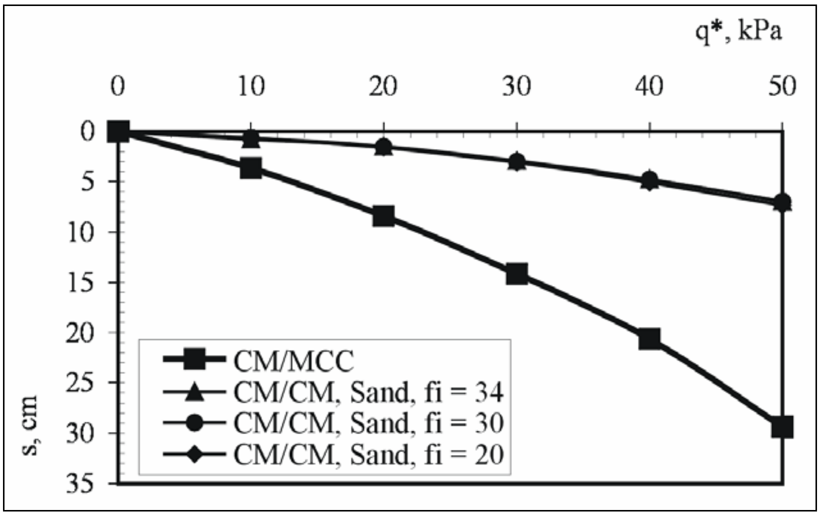


Fig. 14. "Load–settlement" characteristics for the schemes analysed: *Hp* = 2*B*; *Bp* = 3*B*

The results obtained fully confirm the conclusions drawn from the first example and those can now be supplemented as follows:

• The effect of plasticity on the stress and strain state, observed from the instant the load is applied, is greater for a plane strain problem. However, irrespective of the type of problem (plane or axisymmetric strain), the "load–settlement" relationship for the base model is best represented in the CM/CM scheme, and the accuracy of this representation increases with the size of the loadbearing cushion, which is also shown in figures 13 and 14.

a)



b)

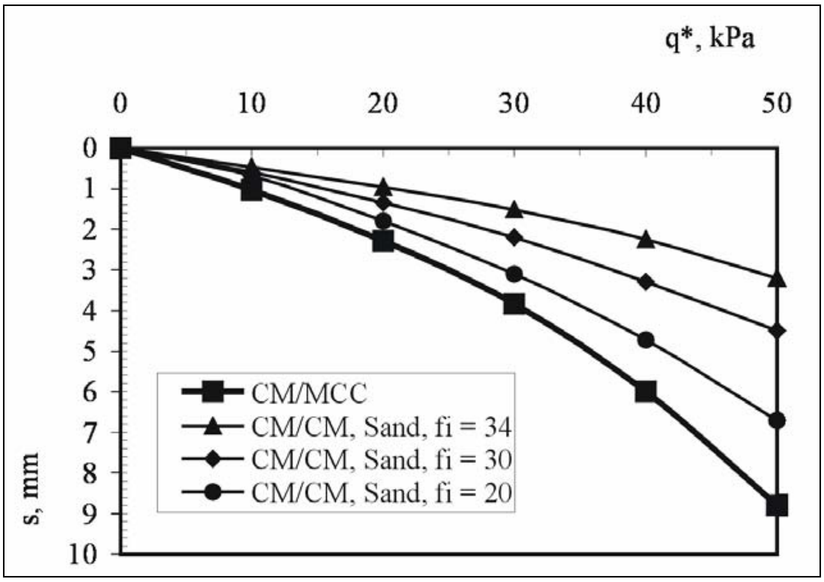
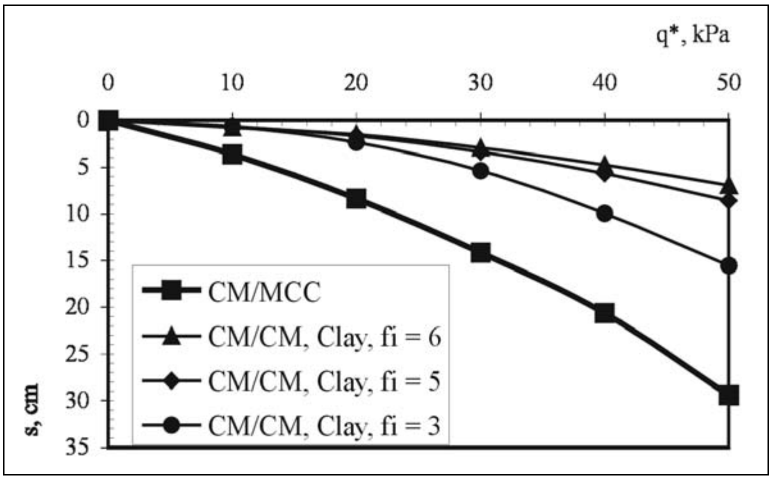


Fig. 15. The effect of the cushion's internal friction angle on the progress of the "load–settlement" characteristics in the CM/CM model for: a) *Hp* = 0.5*B*; *Bp* = *B*; b) *Hp* = 2*B*; *Bp* = 3*B*

• With the re-confirmed qualitative similarity of the "load–settlement" relationship in the base scheme (CM/MCC) and approximating scheme (CM/CM), the effect of the CM model parameters (*E*, *b* and *c*) on this characteristics for the subsoil and cushion depends on the dimensions of the latter. Namely, this effect decreases for the subsoil along with an increase in the cushion dimensions and vice versa, as shown in figures 15 and 16.

a)



b)

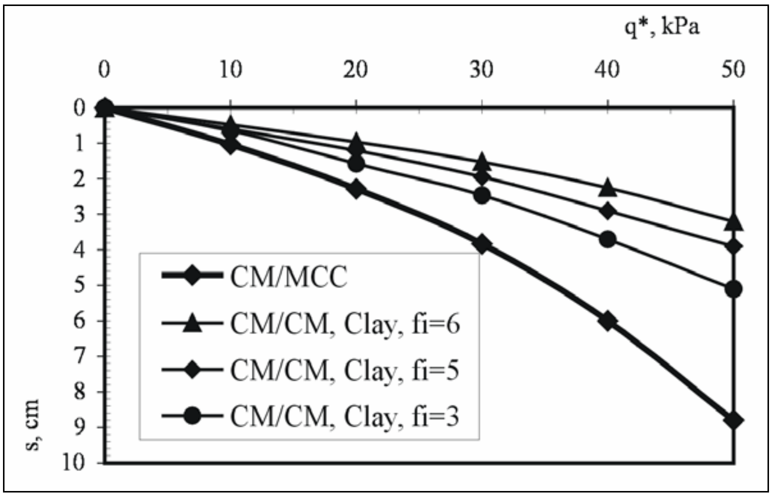


Fig. 16. The effect of the subsoil's internal friction angle on the progress of the "load–settlement" characteristics in the CM/CM model for: a) $Hp = 0.5B$; $Bp = B$; b) $Hp = 2B$; $Bp = 3B$

With reference to the angle of internal friction and cohesion, relevant figures refer to examples of extreme loadbearing cushion dimensions ($Hp = 0.5B$; $Bp = B$ and $Hp = 2B$; $Bp = 3B$).

Again, for both materials, the effect of Poisson's ratio on the final solution is minor.

3.4. SELECTION OF A NUMERICAL MODEL FOR THE SYSTEM

In accordance with the procedure proposed for developing the rational numerical model for the cushion–soft subsoil system, the pre-selection of the set of the models tested had been carried at its successive stages. Next, FEM parametric studies of their representative characteristics were performed. The "load–settlement" relationship was found to be a characteristics. For the relevant calculations, two numerical examples were chosen, one of which represented an axisymmetric problem, while the other – a plane strain problem.

The theoretical "load–settlement" curve representing the discrete CM/MCC base model (Coulomb–Mohr's model for the cushion and Modified Cam Clay for the subsoil) constituted a reference line for the calculations performed. In the numerical analyses, the "load–settlement" relationship was determined for simpler schemes and material parameters obtained for them, and the effect of such parameters on the characteristics in question was analysed.

The results of the calculations, partially illustrated in figures 8–11 and 13–19, and the analysis of the conclusions in point 3.3 indicate that:

1. In the scheme selected to represent the subsoil–loadbearing cushion system, the elastic-perfectly plastic Coulomb–Mohr's model (CM/CM) can be used to describe both the subsoil and the cushion. With the base solution relatively well approximated, this scheme is characterised by a limited number of generally accepted, intuitively sensed and easily determined material parameters (E, ϕ, c, v). However, a formally more correct CM/MCC scheme requires, at least from a designer's point of view, an access to parameters of the Modified Cam Clay model. At present, it is not yet possible. While such parameters can be obtained from laboratory studies (e.g. BZÓWKA et al. [6]) or derived from literature (e.g. ATKINSON [1]; SCHOFIELD and WROTH [21]; DESAI and SIRIWARDANE [8]), the first ones require a great deal of work and the second are few and they apply strictly to specific types of soil.

2. The results of the system numerical analyses that employ the Coulomb–Mohr's model are heavily dependent on the values of soil physical parameters. This refers, in particular, to the elastic modulus, the angle of internal friction and the cohesion (E, ϕ, c). Therefore, it is crucial that the parameters are determined realistically, taking into consideration the load history and conditions of the design structure–subsoil interaction.

More generally, soil parameters can be used in the analysis as independent variables (e.g. PIECZYRAK [20]).

To summarise, the numerical model CM/CM has been adopted for the description of a soft subsoil–cushion system.

The appropriateness of such a choice is additionally substantiated by the concept of the so-called experimental soil engineering (ESE) (DYER et al. [9]), currently popularised in geotechnics. ESE boils down to analysis of geotechnical problems by means of mechanical models that are as simple as possible, but whose parameters must be estimated taking into account the whole complexity of soil's behaviour under loading.

The choice of the scheme will need to be checked against the results of experimental studies, or even better, against the results from the monitoring of actual solutions.

The author has made such an attempt and its outcome will be presented in a separate paper.

4. FINAL CONCLUSIONS

1. Loadbearing cushions are effective in improving weak subsoil. They are easily implemented and applicable in a wide range of weak soils. There is also a good choice of improving soils that can be used. Moreover, the quality of the fill can be monitored while the work is in progress.

2. At present, an important limitation on a wider use of the above improvement is caused by the lack of theoretical and experimental basis for dimensioning loadbearing cushions. Hence, the numerical model of the soft subsoil–cushion system, CM/CM, has been proposed in the paper. In this scheme, elastic-perfectly plastic models with Coulomb–Mohr's yield surface have been used to describe both soils.

3. The "load–settlement" curve is sensitive to material parameters of both improved and improving soils. Thus, these parameters, obtained due to studies or adopted for calculations, need to be determined with the utmost caution. Conditions of the structure–subsoil system interaction should play a decisive role in the selection of parameters. Also, the loading history needs to be taken into account. Finally, precision in sampling, in preparing the samples and also in performing the tests is of importance.

REFERENCES

[1] ATKINSON J.H., *An Introduction to the Mechanics of Soils and Foundation. Through Critical State Soil Mechanics*, McGraw-Hill Book Company, 1993.

[2] BIERNATOWSKI K., *Fundamentowanie*, PWN, Warszawa, 1984.

[3] BRITTO A.M., GUNN M.J., *Critical State Soil Mechanics via Finite Elements*, Ellis Horwood, Chichester, 1987.

[4] BRYL St. et al., *Tablice inżynierskie*. Vol. 2. *Konstrukcje mostowe, fundamenty*, PWN, Poznań, 1957, s. 950.

[5] BRZĄKAŁA W., NGUYEN HUNG SON, *O poduszkach, poszewkach i materacach*, XII Krajowa Konferencja Mechaniki Gruntów i Fundamentowania, Szczecin–Międzyzdroje, 2000, Vol. 1a, 65–77.

[6] BZÓWKA J., GRYCZMAŃSKI M., SĘKOWSKI J., *Kalibrowanie modelu MCC na podstawie badań trójosiowych*, XLIV Konferencja Naukowa Krynica'98, Poznań–Krynica, 1998, Vol. 7, 113–120.

[7] CICHY W., ODROBIŃSKI W., *Badania nośności fundamentów posadowionych na podłożu słabym z częściową wymianą na podsypkę żwirowo-piaskową*, Archiwum Hydrotechniki, 1974, Vol. XXI, 3, 507–533.

[8] DESAI C.S., SIRIWARDANE H.J., *Constitutive Laws for Engineering Materials with Emphasis on Geologic Materials*, Prentice Halls, Englewood Cliffs, 1984.

[9] DYER M., JAMIOŁKOWSKI M., LANCELLOTTA R., *Experimental soil engineering and models for geomechanics*, 2nd Int. Symp. Num. Mod. Geomech., "NUMOG 2", Ghent, 1986, 873–906.

[10] FLORKIEWICZ A., *Nośność graniczna podłoża o cechach skokowo zmiennych*, rozprawa habilitacyjna.

Politechnika Poznańska, Rozprawy, 224, Poznań, 1990.
[11] GLINICKI S. P., *Fundamentowanie*, Skrypt Politechniki Białostockiej, Białystok, 1984.
[12] GRYCZMAŃSKI M., *Stresses and displacements in subsoils strengthened by loadbearing fills*, 6th Danube-European CSMFE, Varna, 1980, Vol. 2, 101–110.
[13] GRYCZMAŃSKI M., FEDYNYSZYN G., *Wpływ szerokości warstwy wzmacniającej na przemieszczenia i naprężenia w podłożu sprężystym*, Archiwum Hydrotechniki, 1976, Vol. XXIII, 4, 573–586.
[14] GRYCZMAŃSKI M., SKIBNIEWSKA A., *Osiadania fundamentów na podłożu wzmocnionym przez wymianę gruntów*, V Krajowa Konferencja Mechaniki Gruntów i Fundamentowania, Katowice, 1978, 21–27.
[15] GUSSMAN P., *Die Methode der kinematischen Elemente*, Ins. für Grundbau und Bodenmechanik, Stuttgart, 1986.
[16] KEZDI A., *Bodenmechanik*, B2. VEB Verlag, Ung. Akad. der Wisschaflen, Budapest, 1964.
[17] LECHOWICZ Z., RABARIJOELY S., *Ocena osiadań podłoża organicznego na podstawie badań dylatometrycznych*, XI Krajowa Konferencja Mechaniki Gruntów i Fundamentowania, Gdańsk, 1997, Vol. 2, 101–106.
[18] ŁĘCKI P., FLORKIEWICZ A., *Szacowanie nośności granicznej układu poduszka fundamentowa–podłoże gruntowe*, XXXIX Konferencja Naukowa, Krynica, 1993, 203–210.
[19] MITCHELL J.K., GARDNER W.S., *Analysis of loadbearing fills over soft subsoils*, J. Soil Mech. Found. Div. Proc. ASCE, 1971, 97, SM11, 1549–1570.
[20] PIECZYRAK J., *Ustalanie parametrów wybranych modeli gruntu na podstawie próbnych obciążeń*, rozprawa habilitacyjna, Zeszyt Naukowy Politechniki Śląskiej, s. Budownictwo, 91, Gliwice, 2001.
[21] SCHOFIELD A., WROTH P., *Critical State Soil Mechanics*, McGraw-Hill, London, 1968.
[22] СЕНКОВСКИ Е., *Исследоваия влияния размеров песчаной подушкн на осадку ленточного фундамента*, Строительтво и Архитектра, 1990a, 8, ИВУЗ, 130–132.
[23] SĘKOWSKI J., *Badania laboratoryjne nad efektywnością wzmacniania słabego podłoża gruntowego geosiatkami*, IX Krajowa Konferencja Mechaniki Gruntów i Fundamentowania, Kraków, 1990b, Vol. 2, 363–368.
[24] SĘKOWSKI J., *Analiza sprężysto-plastyczna podłoża wzmocnionego poduszką piaskową*, Zeszyty Naukowe Politechniki Śląskiej, s. Budownictwo, 80, 1995a, 77–92, Materiały Konferencji Środowiskowej Sekcji Mechaniki Gruntów i Skał oraz Fundamentowania Komitetu Inżynierii Lądowej i Wodnej PAN – Geotechnika w ośrodku gliwickim.
[25] SĘKOWSKI J., *Model układu „słabe podłoże–poduszka piaskowa" w świetle wyników badań terenowych*, I Problemowa Konferencja Geotechniki „Współpraca budowli z podłożem gruntowym", Białystok–Wigry, 1998, 139–148.
[26] SĘKOWSKI J., *Podstawy wymiarowania poduszek wzmacniających*, rozprawa habilitacyjna, Zeszyt Naukowy Politechniki Śląskiej, s. Budownictwo, 2002, 94, Gliwice.
[27] SZECHY C., *Teoretyczne i praktyczne określanie wymiarów poduszek piaskowych*, IV Ogólnopolska Konferencja Mechaniki Gruntów i Fundamentowania, Wrocław, 1967, 87–95.
[28] TOCHKOV E., *Determination de la hauteur des semiles de sable sur sols tendres*, Proc. 5-th ICSMFE, Paris, 1961, Vol. 1, 837–840.
[29] TSCHUSCHKE W., *Wykorzystanie metody statycznego sondowania do klasyfikacji gruntów* (a typescript), Katedra Geotechniki Akademii Rolniczej w Poznaniu, Poznań, 1993.
[30] ZADROGA B., *Ustalanie miarodajnych modułów odkształcenia gruntu na podstawie badań wzajemnego oddziaływania budowli i podłoża gruntowego*, Archiwum Hydrotechniki, 1982, z. 1–2, XXIX, 133–159.