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EXPERIMENTAL VERIFICATION OF SOFT SUBSOIL – LOADBEARING CUSHION SYSTEM

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Abstract: A loadbearing cushion is one of the ways of improving soil under foundations of buildings and engineering structures. In the paper published in [15], the author presented his own numerical model of the soft subsoil–loadbearing cushion system. It was 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 model was designated as the CM/CM. This concept was a result of a specific procedure, which comprised theoretical and empirical premises, FEM parametric studies and the author's own verifying experiments. They are making the content of this paper being the completion of [15].

Streszczenie: Poduszki wzmacniające to jeden z zabiegów poprawiających warunki posadowienia obiektów budowlanych i inżynierskich. W [15] autor przedstawił własną propozycję modelu obliczeniowego dla układu słabe podłoże–poduszka wzmacniająca. Jest ona oparta na założeniu, że do opisu właściwości poduszki i podłoża można wykorzystać ciała sprężyste-idealnie plastyczne o prawach płynięcia plastycznego stowarzyszonych z warunkiem Coulomba–Mohra. Taki dwustrefowy model oznaczono symbolem CM/CM. Propozycja ta była wynikiem określonej procedury postępowania, obejmującej: przesłanki teoretyczne i doświadczalne oraz studia parametryczne MES, a także własne badania weryfikacyjne autora. Zostały one opisane w tym artykule, stanowiąc uzupełnienie [15].

Резюме: Подкрепляющие подушки это одно из мероприятий, улучшающих условия основания строительных и инженерных объектов. В [15] автор представил собственное предложение расчетной модели для системы слабое основание–подкрепляющая подушка. Оно базирует на предпосылке, что для описания свойств подушки и основания можно использовать упругие, идеально пластические тела прав пластического течения, сопряженных с условием Кулона–Мора. Такая двухзонная модель обозначена символом ЦМ/ЦМ. Это предложение было результатом определенной процедуры, охватывающей: теоретические и экспериментальные предпосылки, параметрическое изучение МЭС, а также частные проверяющие исследования автора. Они описаны в этой статье и составляют дополнение [15].

1. INTRODUCTION

In [15], the author presented his own numerical model of the soft subsoil–loadbearing cushion system. It was 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 model was designated as the CM/CM. This concept was a result of a specific procedure, which comprised theoretical and empirical premises and FEM parametric studies.

The extensive parametric studies, presented in the monograph by SĘKOWSKI [14] and partially quoted in [15], have shown that with the parameters adequate to a given cushion geometry the model suggested is capable of a "loading–settlement" approximation, strictly convergent to a corresponding approximation using a much more sophisticated and appropriate base model: CM/MCC (MCC – the Modified Cam Clay Model describing a soft subsoil). As a realistic assessment of the improved soft subsoil settlement is concerned, in the case of a monotonic loading changing in a broad range, there is no necessity to refer to more complicated constitutive models than elastic-perfectly plastic CM/CM ones. The result of the parametric studies allows us to believe that the model chosen is a rational base for the dimensioning of the system, without constituting an irrefutable proof of adequacy. An experimental verification that would prove the conformity of the theoretical "loading–settlement" characteristics of the CM/CM model with the results of various experiments is essential.

The paper presents an attempt at such a verification, based on three of the test loads conducted by the author on the soft subsoil improved by the loadbearing cushion. The first experiment consisted in a model laboratory testing, the subject of which were "soft subsoil–cushion" systems differing with respect to the thickness of the cushion. The second experiment included in-situ model testing with two different cushion thicknesses as well as test loads of unimproved subsoil. Finally, the third one concerned a road embankment subsoil stiffened with a gravel layer reinforced with a geomesh. The experiment consisted of test loads before and after improvement.

An integral part of all the tests was an estimation of a corresponding theoretical "loading–settlement" profile of the CM/CM model as the solution of the appropriate boundary problem representing the test load process. The execution of this part required assessing the CM/CM model parameters and incremental FEM analysis, retaining the dimensions and boundary conditions occurring in the model tests. The ranges and procedures of parameters estimation were different in each test, flexibly adjusted to the database. Uniformity in FEM analysis was imposed by the software module CRISP'93 (BRITO and GUNN, [2]), widely used in the Department of Geotechnics (compare, SĘKOWSKI and STERNIK [10]).

2. LABORATORY TESTS

2.1. PROGRAMME, COURSE AND RESULTS OF MODEL TESTING

The model testing, described earlier by the author (SĘKOWSKI [7]–[9]), was conducted on a testing stand, which comprised: a medium-size testing box, materials simulating the subsoil, loading system and measuring device. A box of dimensions: 1 m×1 m×1 m, with a rigid steel framework and walls made of 20 mm thick plexiglass (figure 1) was the fundamental element of the testing stand. On the front wall of the box a 10 cm × 10 cm grid was plotted. Precisely behind the nodes of the grid, colourful point markers were placed in the material modelling the subsoil.

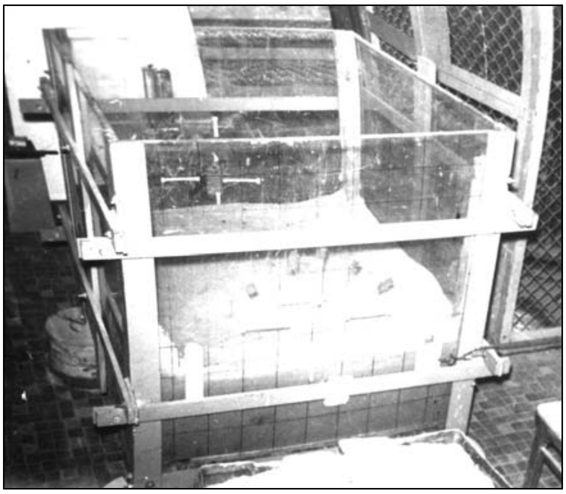

Fig. 1. General view of the medium size testing box

The soft subsoil was simulated by a power plant ash (Rybnik Plant Power); grain size distribution corresponded to that of uniform fine sand. The subsoil was then strengthened by uniform medium sand. The basic geotechnical parameters of the materials simulating the soft subsoil and the strengthening layer were determined in standard laboratory tests shown in table 1.

Table 1

Basic geotechnical parameters of the materials simulating the subsoil

Material	U [–]	w [%]	γ_s [kN/m^2]	γ_{ds} [kN/m^3]	w_{opt} [%]	I_S/I_D [–]
Fine sand	2.9	~ 0	26.0	17.5	11.0	0.92/0.41
Ash	3.85	25.0	22.7	8.2	54.0	0.92/0.78

A rigid continuous footing was simulated by a fragment of a wooden 14 cm× 23 cm cross-tie.

The loading system consisted of a manual screw elevator with the operation range up to 50 kN and a proving ring with the operation range up to 100 kN, exactly up to 0.8 kPa. The elevator was put on the foundation and abutted, through the proving ring, to a rigid steel beam, restrained by a slab of a great force. The measuring devices included dial gauges fixed independently of the loading system. They measured displacement of the foot's corners.

The loading and measuring system used in the research is shown in figure 2. The displacements of the model points chosen were measured visually and photographically with use of the colourful point markers described above.

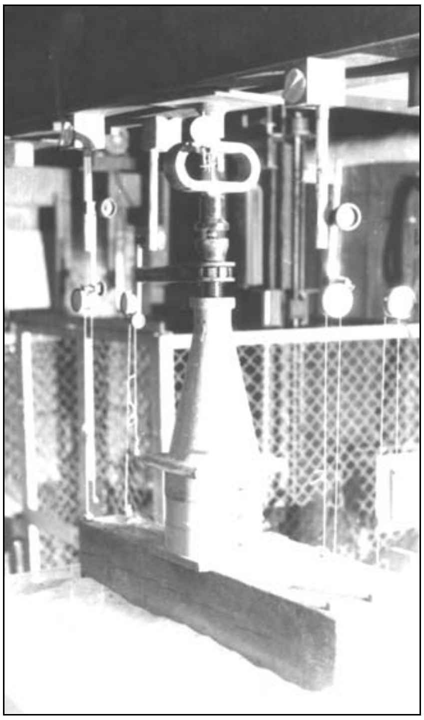

Fig. 2. Loading and measuring system

Each test was conducted following a precisely specified technological regime. The ash was placed in 10-cm thick layers. It should be mentioned that in this case the bottom part of the box up to the height of 30 cm was filled with compacted gravel. Each layer of the ash was manually compacted by means of a heavy Proctor tamper dropped from the height of 10 cm. The sand was compacted in a similar way, but the layers were 2-cm thick and the tamper was light. Both materials were compacted once "place by place". While forming the model, the point markers were placed at the intersection of grid lines close to the plexiglass wall. The foot was placed at a depth equal to half of its width ($D = B/2$). On the foot's corners four dial gauges were installed. They were fixed by means of magnetic holders to steel elements independent of the box and loading system. Scheme of the testing stand is shown in figure 3.

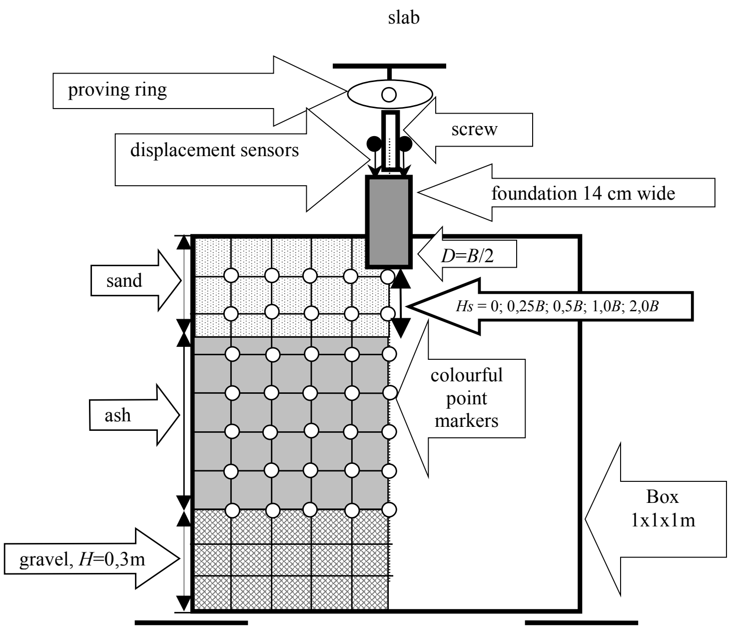

Fig. 3. Scheme of the testing stand

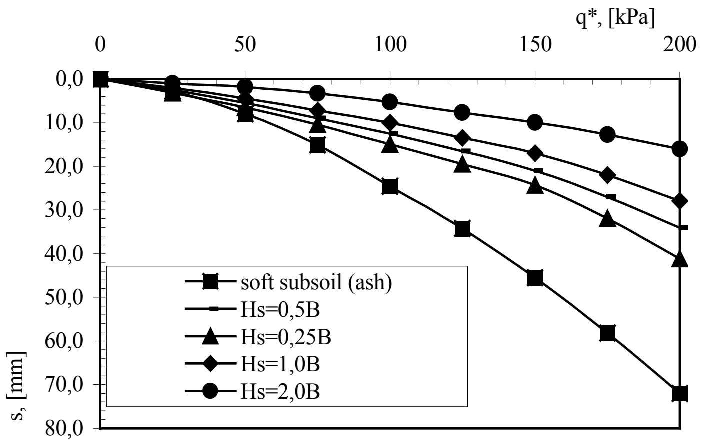

Fig. 4. Loading–settlement relationship for a homogeneous subsoil and the subsoil strengthened by sand layer of the thickness *Hs*

At the test stand the author conducted a series of model tests with a soft subsoil and the subsoil strengthened by the layer of sand. The thickness of the sand layer was different in all the tests ($Hs = 0.25\ B$; $Hs = 0.5\ B$; $Hs = 1.0\ B$; $Hs = 2.0\ B$, where $B = 14$ cm). In each test, the foundation was under the load of $q^* = 200$ kPa, increasing stepwise every 25 kPa. Succeeding loading steps were imposed after conventional settlement stabilization, i.e., when $\Delta s_i \leq 0.1$ mm/15 min.

The loading–settlement relationship for all subsoil models is shown in figure 4.

2.2. THEORETICAL INTERPRETATION OF RESULTS WITH USE OF CM/CM MODEL

In [15], the author concentrated mainly on capturing the effect of soft soil improving, expressed by the reduction of settlement or, indirectly, by an increase in its bearing capacity, without making any attempt to characterize the relations determined or to analyse them extensively. In the analysis presented below, the results of the specified tests determine an adequacy assessment base of the CM/CM model.

As was mentioned, the essence of verification represents a comparison of experimental $q^* - s$ relationships (figure 4) with corresponding theoretical relationships determined with use of the CM/CM model. The latter were determined in incremental FEM analysis using constitutive CM/CM formulas and discrete subsoil models, built of 285 eight-nodes rectangular elements, with slidable support along vertical walls and fixed support along the horizontal bottom.

The dimensions of the discrete model (H^*, B^*) corresponded to the dimensions of the box. Their relation to the width of the footing (H^*/B and B^*/B) was respectively 5.00 and 7.14.

The geometrical and discrete models of the system are shown in figure 5.

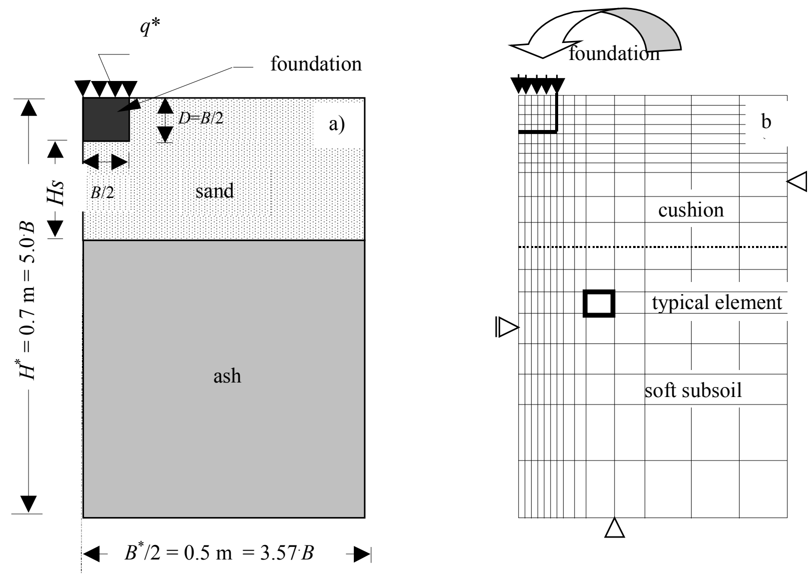

Fig. 5. Geometrical (a) and discrete (b) models of system

Special attention should be paid to the problem of CM/CM model calibration. The difficulty in the reliable determination of the soft subsoil parameters (power plant ash) in laboratory tests encouraged the author to their estimation in an inverse analysis. Regression of the test load and FEM results was used. Such a procedure usually has an essential shortcoming: using the results of test load for parametric model identification deprives us of database easily accessible for verification. Therefore, we can only make a less convincing inference about the model adequacy exclusively on the grounds of quality of matching, measured by the deviation of a modified coefficient of determination from unity (PIECZYRAK [4], [5]).

$$R^2 = 1 - \frac{\sum (y_i - \hat{y}_i)^2}{\sum y^2{}_i - \frac{\left(\sum y_i\right)^2}{n}}, \tag{1}$$

where:

y_i – the results,

$\hat{y}_i$ – the corresponding ordinate on the approximation curve,

n – the number of measurements

The experiment under consideration is free from the shortcoming mentioned. Out of four model tests carried out for strongly differentiated thickness of the cushion, only the results of the second one ($Hs = 0.5\ B$) were taken into account for parametric estimation. They were taken as reliable and independent of Hs/B relation. The other tests provide a database for verification. To the best of the author's knowledge, the idea has not yet been used in the testing of subsoil models. In the numerical analysis, it was assumed that the foundation was made of linear elastic material characterized by the following parameters: $E_f = 3200$ MPa, $\nu_f = 0.25$, $\gamma_f = 18$ kN/m^3. The ash and sand were taken in order to fulfill the postulates of Coulomb–Mohr's model, except for the evaluation of initial elastic modulus, which was done on the basis of linear elastic model.

The parameters we searched for were: the elasticity modulus E, an internal friction angle ϕ, the cohesion c. The value of the Poisson's coefficient ν of the sand and the ash was assumed to be 0.25.

Nonlinear regression analysis was carried out by trial-and-error method. Another more sophisticated method (e.g., simplex) would require a development of the FEM CRISP'93 software module.

The procedure comprised two stages. In the first stage, referring to homogeneous subsoil, the parameters of the ash were estimated. In the second stage, the sand parameters were estimated on the basis of q^*–s curve obtained out of the test, in which the strengthening layer thickness Hs was equal to $0.5B$. Having known the sand and ash parameters, we checked the remaining systems ($Hs = 0.25B$, $Hs = B$ and $Hs = 2B$). In the initial version, the elastic moduli of the sand and ash were estimated taking account of the "linear" part of the empirical loading–settlement relationship and linear elastic model. The remaining material parameters, i.e., the internal friction angle and cohesion, were assessed with the trial-and-error method, as mentioned above, with an adequate densification of the scanned range and the correction of the earlier assumed value of the elastic modulus.

The values of the material parameters are presented in table 2, whereas the mean values of the calculated and measured settlement at different loading levels – in table 3. The values of the modified coefficient of determination R^2 are presented in the last line of table 3. Both curves, experimental and numerical, representing the tests carried out, are shown in figures 6–10.

Table 2

Coulomb–Mohr's model parameters of sand and ash

Material parameters	ϕ [°]	c [kPa]	E [MPa]	γ [kN/m^3]
Ash	18	15.2	0.8	7.5
Sand	35	0.1	150	18

Table 3

Loading–settlement relationship obtained experimentally and **in numerical analysis**

Unit loading q^* [kPa]	Mean settlement s [mm] for different subsoil models				
	Unimproved (figure 6)	Strengthened by the sand layer of the thickness Hs			
		$Hs = 0.25B$ (figure 7)	$Hs = 0.5$B (figure 8)	$Hs = B$ (figure 9)	$Hs = 2B$ (figure 10)
0	0/**0**	0/**0**	0/**0**	0/**0**	0/**0**
25	2.8/**3.39**	3.2/**3.381**	2.5/**3.095**	2.0/**2.421**	1.0/**1.723**
50	8.0/**8.76**	6.5/**7.399**	5.5/**6.721**	4.5/**5.37**	1.85/**3.315**
75	15.1/**14.76**	10.5/**11.57**	9.0/**10.553**	7.25/**8.553**	3.35/**5.108**
100	24.6/**22.81**	15.0/**15.90**	12.5/**14.499**	10.0/**11.91**	5.25/**7.09**
125	34.2/**32.76**	19.5/**20.59**	16.5/**18.739**	13.5/**15.44**	7.7/**9.208**
150	45.5/**44.47**	24.3/**25.83**	21.0/**23.492**	17.0/**19.19**	9.95/**11.40**
175	58.2/**57.53**	32.0/**31.62**	27.0/**28.62**	22.0/**23.16**	12.75/**13.70**
200	72.0/**72.09**	41.2/**37.79**	34.1/**34.26**	27.9/**27.36**	16.05/**16.13**
$\boldsymbol{R^2}$	**0.999**	**0.988**	**0.979**	**0.977**	**0.942**

Both, the visual assessment of the goodness of fit of the relations compared and the high values of the modified coefficient of determination, prove that the CM/CM model is capable of the accurate characterizing of the soft subsoil–loadbearing cushion system behaviour in laboratory tests.

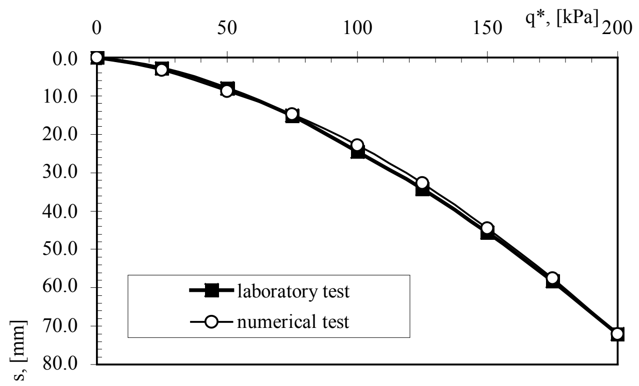

Fig. 6. Homogeneous subsoil. Loading–settlement relationship

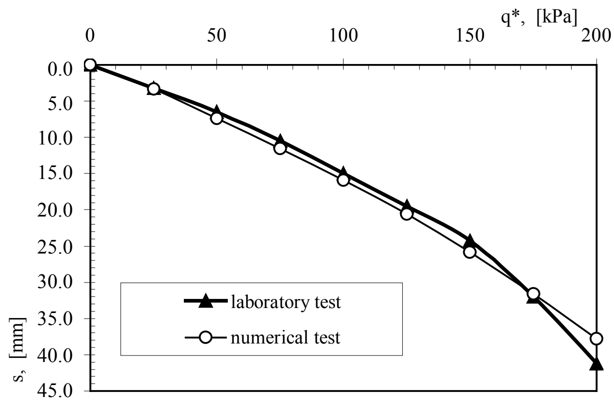

Fig. 7. Strengthening layer of the thickness $Hs = 0.25B$. Loading–settlement relationship

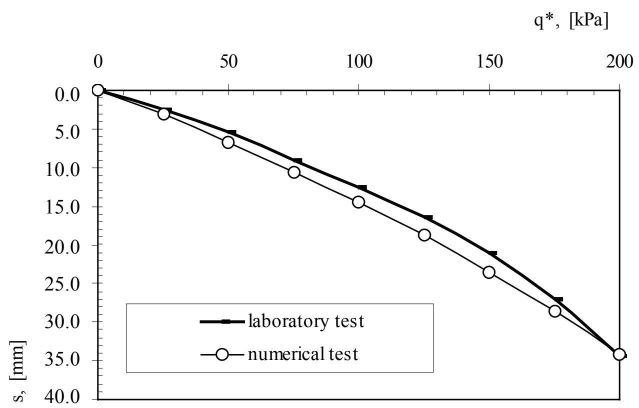

Fig. 8. Strengthening layer of the thickness $Hs = 0.5B$. Loading–settlement relationship

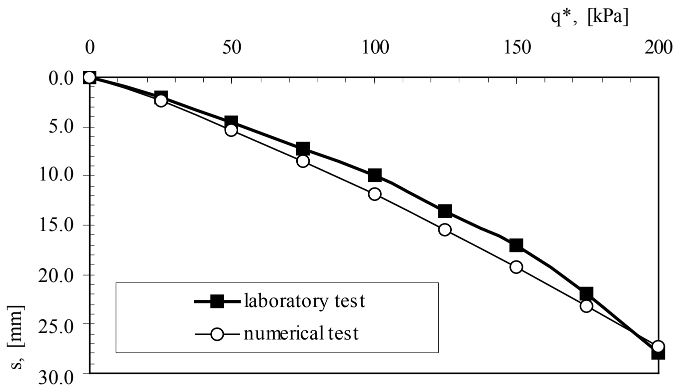

Fig. 9. Strengthening layer of the thickness *Hs* = *B*. Loading–settlement relationship

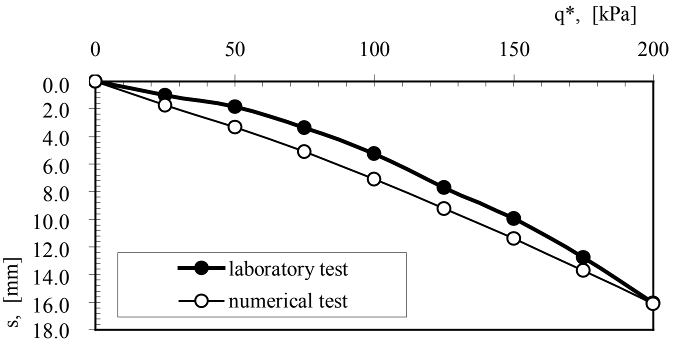

Fig. 10. Strengthening layer of the thickness *Hs* = 2*B*. Loading–settlement relationship

3. MODEL IN SITU TESTS OF THE FOUNDATION–LOADBEARING CUSHION–SOFT SUBSOIL SYSTEM

3.1. PROGRAMME, COURSE AND RESULTS OF IN SITU TESTING

The objective of the research and numerical analyses conducted by the author in the years 1997–1998 was, among other things, to assess the native subsoil improvement by means of a plain sand cushion and a sand cushion reinforced in the bottom part with a double geogrid. The background of the comparison was the result of non-strengthened subsoil (the native subsoil) testing. The examples presented below show only the native subsoil and the subsoil improved by a sand cushion and they refer to the author's earlier papers (SĘKOWSKI [11]–[13]).

The in situ tests were conducted in November 1997. The subsoil within the testing field limits of 8.0 m × 8.0 m was stratigraphically and lithologically homogenous. In a vertical section of soil, the following layers can be shown: 1. the topsoil layer of the thickness of 20 cm, 2. a hard-plastic clayey silt (I_L = 0.24, I_P = 11.2%) transformed into a plastic one (I_L = 0.30) at the depth of around 1.8 m, 3. a dense medium sand (I_D = 0.7) deposited to the depth of about 9.0 m. Ground water table occurred at the depth of about 1.4 m below the ground level. Figure 11 presents an exploratory bore-hole profile typical of the subsoil.

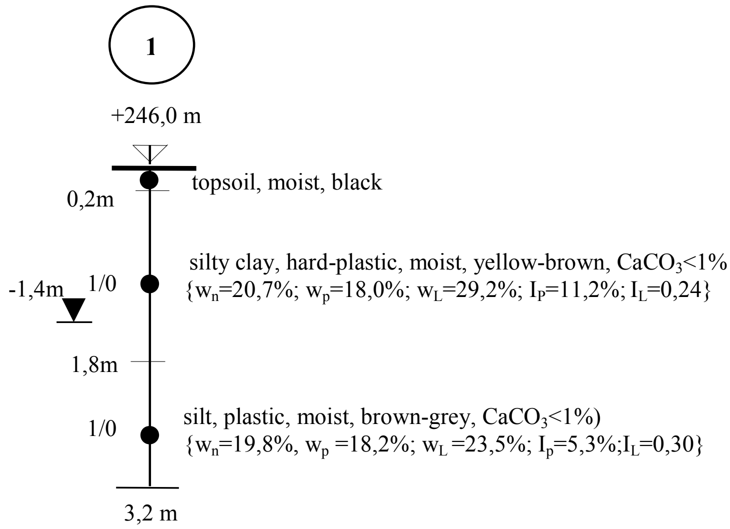

Fig. 11. Typical (simplified) geotechnical soil profile

Four test stands were prepared: the first one on the native subsoil, whereas the remaining ones on the subsoil strengthened by a sand cushion. The last loadbearing cushion was additionally reinforced with geogrids in the bottom part of the cushion. The first excavation was a square 0.5 m in width and 0.5 m in depth ($a_1 = b_1 = D_1$ = 0.5 m) (figure 12a). All the remaining excavations were squares as well, but their width was twice as great as that of the first one ($2a_1$, $2b_1$), and their depths differed (D_2 = 0.75 m, $D_3 = D_4$ =1.0 m). The loadbearing cushions in the excavations No. 2 and No. 3 were made of uniform, medium sand of adequate thickness: Hc = 0.25 m (0.5 B), Hc = 0.5 m (1.0 B) and the width Bc = 2B. The sand was laid in 5 cm thick layers, each similarly compacted (figure 12b). The density index (I_D = 0.88) was calculated according to PISARCZYK's [7] proposal on the basis of a mean value of the coefficient of compaction (I_S^{mean} = 1.0). The coefficients of compaction (two in each case) were estimated with the use of a volumeter and pit-and-water method.

a b c

Fig. 12. The stages of field tests: a) excavation, b) formed sand cushion, c) pad foundation

Each pad foundation, 0.5 m × 0.5 m in width, and 1.0 m in depth, was built of 12 concrete 8-cm thick flagstones on 1-cm thick cement mortar. On the penultimate flagstone, two parallel, 1.4 m long, thick reinforcement bars of the diameter of 12 mm were embedded in concrete. A free space around the foundations was carefully filled with the native soil. The test stands, prepared in such a way, with pad foundations being designed 0.5 m above the ground level (figure 12c) were left for two weeks.

Fig. 13. Testing field after experiments

Loading was applied in the following manner: first, one pavement slab was put on each pad foundation by means of a crane; in the next steps, two following slabs were put. Stock was taken of each pavement slab, the mean dimensions of the slabs were: 3 m × 1 m × 0.15 m. The condition of applying the next load was a conventional stabilization of the foundation settlement $\Delta s_i \leq 1$ mm/15 min. Finally, eight pavement slabs were put on the test stand No. 1 and nine slabs – on the remaining ones. Taking into account the way of imposing the load, its axiality was extremely important.

Settlement of the foundations was measured with a surveyor's level and a levelling staff fixed to the reinforcement bars. Before the main test loads, the settlement due to the deadweight was measured.

In figure 13, all the test stands after the completion of the experiments are shown.

The results for all the test stands are shown in tables 4–6 and in the form of loading–settlement relation in figure 14. It is worth mentioning here that the difference in the unit loading on particular stands arose from a different number of the loading pavement slabs. It should also be added that the experiments with the cushion reinforced with geogrids were eventually eliminated from a further procedure.

The critical state was not reached in any test, but all the graphs show a clear change of the curvature.

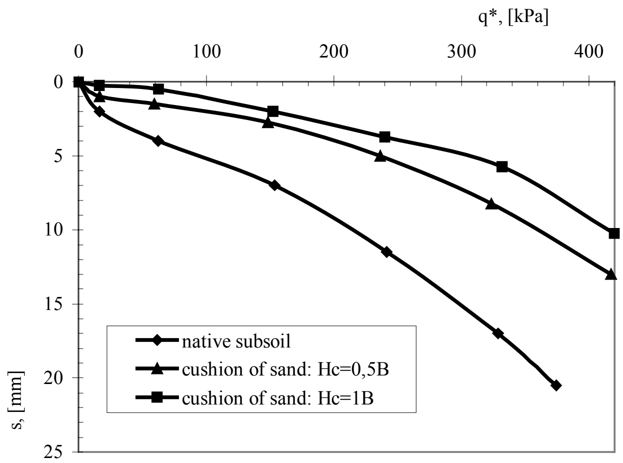

Fig. 14. Experimental loading–settlement relationship

Table 4

List of test results: loading–settlement of unimproved subsoil

Unit load q^* [kPa]	0	16.5	62.4	153.5	241.5	328.6	374.2
Mean settlement s [mm]	0	2.0	4.0	7.0	11.5	17.0	20.5

Table 5

List of test results: loading–settlement of subsoil strengthened by a sand cushion of the thickness $Hc = 0.5\ B$

Unit load q^* [kPa]	0	16.5	59.5	148.4	236.4	323.5	417.5
Mean settlement s, [mm]	0	1.0	2.5	2.75	5.0	8.25	13.0

Table 6

List of test results: loading–settlement of subsoil strengthened by a sand cushion of the thickness $Hc = 1.0\ B$

Unit load q^* [kPa]	0	16.5	62.6	152.4	240.0	332.1	420.0
Mean settlement s [mm]	0	0.25	0.5	2.0	3.75	5.75	10.25

3.2. THEORETICAL INTERPRETATION OF THE RESULTS WITH USE OF THE CM/CM MODEL

The basic parameters of the native soil and sand are presented in tables 7 and 8, respectively. They were determined in laboratory tests or selected from correlations and equations published in literature.

Table 7

Some geotechnical parameters of native soil

Material type	I_L [1]	I_P [%]	ϕ [°]	c [kPa]	E [MPa]
Silty clay/silt	0.24	11.2	32.3	20.5	3.8

Table 8

Basic geotechnical parameters of the material used for building a loadbearing cushion

Soil type	U [1]	w [%]	ρ_{ds} [kN/m^3]	w_{opt} [%]	I_S [1]	ϕ [°]	c [kPa]	M_0 [MPa]
Medium sand	2.75	≈ 0	17.44	12.4	1.00	35.0	0	170

The strength parameters (ϕ, c) of the cohesive native soils were determined in consolidated undrained triaxial tests after their initial isotropic consolidation under the pressure of 20 kPa. The samples, taken from the depth of about 1.0 m, were in the intact natural state. The modulus E was determined from the straight segment of the loading–settlement relation within unit load range of 0.0–62.4 kPa, using the formula given by WIŁUN [16]:

$$s_D = \alpha^* s = \alpha^* \Delta\sigma^* \omega^* (1 - \nu^2)^* B/E, \tag{2}$$

where: $B = 0.5$ m, $\omega = 0.88$, $\alpha = 0.63$, $\Delta\sigma = 62.4$ kPa, $s = 0.004$ m, $\nu = 0.3$.

It was assumed that the pad foundation was made of concrete of the class B15. Linear elastic model (LS) was used with the parameters: $E_c = 28500$ MPa, $\nu = 0.167$. Silt/silty clay representing the soft subsoil and medium sand of the loadbearing cushion was modelled with the elastic–ideal plastic Coulomb–Mohr's model (CM).

A discrete 3D geometrical model for the system was constructed of 296 twenty-noded hexagonal elements (441 nodes). Its "plane section" is shown in figure 15. The model was supported slidably along its vertical walls and fixed along the horizontal bottom. The dimensions of the model were the following: $B^* = L^* = 2.0$ m, $H^* = 1.5$ m, which are 4 B*4 B*3 B in relation to the foundation's width ($B = 0.5$ m).

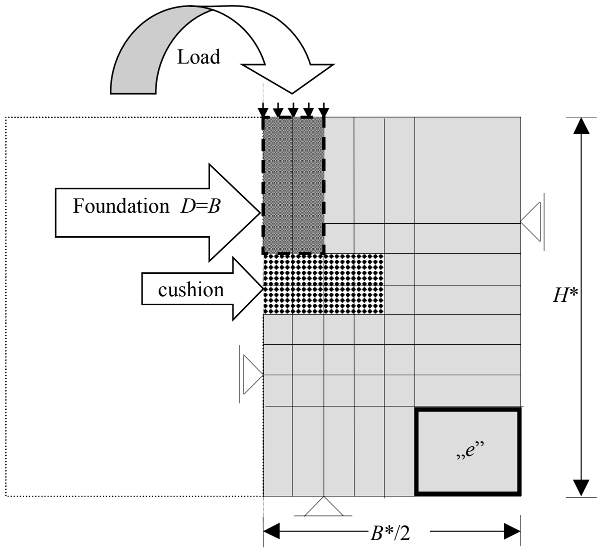

Fig. 15. Geometrical model for the system

The procedure in this case was different from that in the previous example. It was assumed here that the basis for the numerical analysis would be the subsoil and cushion parameters listed in tables 7 and 8.

In the first stage, the settlement of pad foundation on the native subsoil was calculated in the function of load. The magnitude of the load corresponded to the in situ loading.

Numerical results against the background of the empirical results are presented in table 9. Graphical interpretation is shown in a similar convention in figure 16.

Table 9

Loading–settlement relation for unimproved subsoil, in situ tests and numerical calculations

Unit load q^* [kPa]	0	16.5	62.4	153.5	241.5	328.6	374.2
Average settlement s, in situ/numerical ($R^2 = 0.931$)	$\frac{0}{0}$	$\frac{2.0}{0.6}$	$\frac{4.0}{2.3}$	$\frac{7.0}{5.8}$	$\frac{11.5}{9.8}$	$\frac{17.0}{14.7}$	$\frac{20.5}{18.1}$

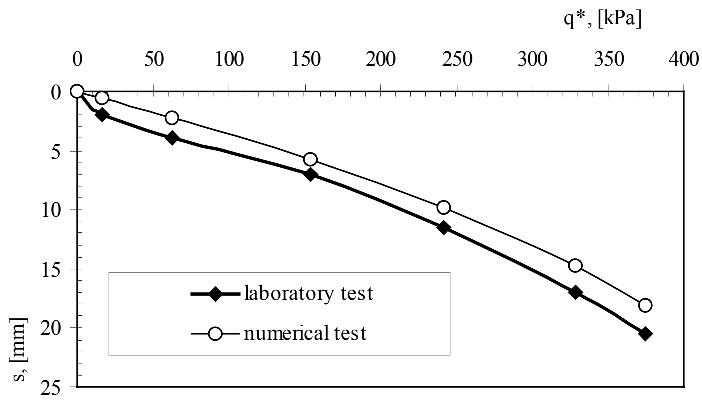

Fig. 16. Native subsoil. Loading–settlement relation

In the second stage, calculations were carried out for the subsoil strengthened by a sand cushion of the thickness Hc = 0.5 B and Hc = 1.0 B, assuming Coulomb–Mohr's model for the sand with material parameters given in table 9. The results are shown, again against the background of empirical ones, in tables 10 and 11. Graphical interpretation is given in figures 17 and 18.

Table 10

Loading–settlement relation for the subsoil strengthened by sand cushion of the thickness Hc = 0.5 B, in situ tests and numerical calculations

Unit load q^* [kPa]	0	16.5	59.5	148.4	236.4	323.5	417.5
Average settlement s, in situ/numerical (R^2 = 0.933)	0/0	1.0/0.4	1.5/1.4	2.75/3.8	5.0/6.4	8.25/9.7	13.0/14.3

Table 11

Loading–settlement relation for the subsoil strengthened by sand cushion of the height Hc = 1.0 B, in situ tests and numerical calculations

Unit load q^* [kPa]	0	16.5	62.6	152.4	240.0	332.1	420.0
Average settlement s, in situ/numerical (R^2 = 0.942)	0/0	0.25/0.2	0.5/1.0	2.0/2.9	3.75/4.9	5.75/7.2	10.25/9.7

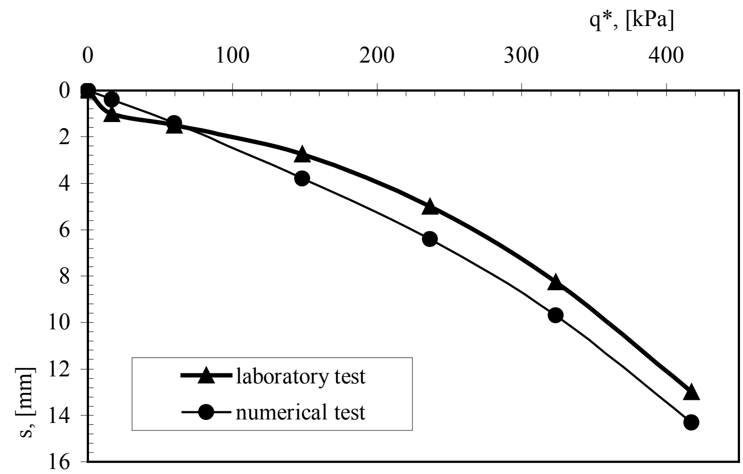

Fig. 17. Native subsoil strengthened by cushion of the height $Hc = 0.5\ B$ and the width $Bc = 2\ B$. Loading–settlement relation

The values of the material parameters of the native subsoil (E_{ns}, ϕ_{ns}, c_{ns}) are shown in table 12. They were determined following the procedure for the first experiment. The results and the curve representing the loading–settlement relation for these parameters are presented against the background of empirical tests in table 13 and figure 19.

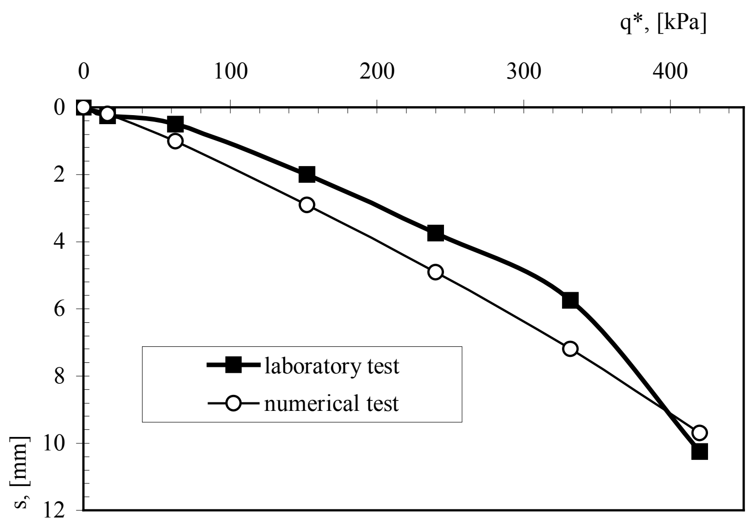

Fig. 18. Native subsoil strengthened by cushion of the height $Hc = B$ and the width $Bc = 2\ B$. Loading–settlement relation

Table 12

Coulomb–Mohr's model parameters for native subsoil from numerical analysis

Native subsoil	ϕ_{ns} [°]	c_{ns} [kPa]	E_{ns} [MPa]	R^2 [1]
Model parameters	24	18	3.8	0.982

Table 13

Loading–settlement relation for unimproved foundation subsoil with the parameters from table 12

Unit load q^* [kPa]	0	16.5	62.4	153.5	241.5	328.6	374.2
Average settlement s, in situ/numerical ($R^2 = 0.982$)	0/0	2.0/0.6	4.0/2.3	7.0/5.9	11.5/10.8	17.0/17.0	20.5/20.6

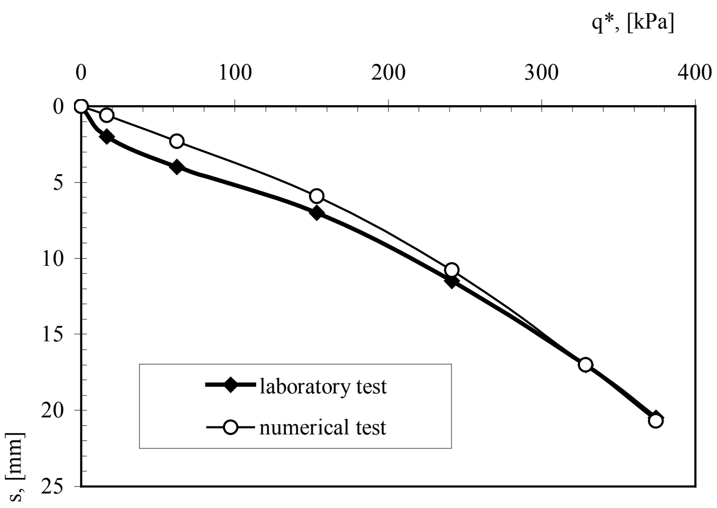

Fig. 19. Native subsoil. Loading–settlement relation

The differences in the values of the parameters ϕ and c, determined on the one hand in laboratory tests and on the other one – in numerical analysis, can be explained by the inaccuracy of the former as well as by the unfavourable weather conditions (it was raining and snowing just before the tests).

In the light of the model field tests, the convergence of the theoretical predictions with use of the CM/CM model with the results of the experiments can be considered as, at least, satisfactory. The initial deviation resulting from densification of the cushion and subsoil material lays out the range of the CM/CM model description.

4. IN SITU TESTS OF A ROAD EMBANKMENT SUBSOIL STREGTHENED BY A LOADBEARING CUSHION

4.1. IN SITU TESTING

The experiment under consideration (with numerical analysis broadened in comparison with that involving source material) was chosen from the research carried out by the scientists from the Department of Geotechnics of Silesian University of Technology (GRYCZMAŃSKI et al. [3]). The purpose of their research was to verify practically the effect of mining damage on the reinforcement of embankments and the Gliwice–Katowice A4 motorway section. The regions of the current and expected ductile mining deformation of the II–IV categories were protected, among others, by a strengthened loadbearing cushion made of coarse-grained material built below road embankment's body. In the first point, attention was focused on the stiffness and load capacity assessment of the reinforced cushions that strengthens the road embankment's subsoil. Thus field tests on unimproved subsoil and subsoil strengthened by geogrids produced by several companies were conducted. Their main purpose was to evaluate the elementary mechanical parameters of the reinforced loadbearing cushion such as E_{pz}, ν_{pz}, ϕ_{pz}, c_{pz} on the basis of test loads with the use of the FEM inverse analysis. In a comparative analysis, use was made of the parameters of an unimproved subsoil (E, ν, ϕ, c) and the subsoil strengthened only by a coarse grained material cushion (E_p, ν_p, ϕ_p, c_p) derived in the same way.

The testing field was located in Chorzów, nearby the Highway Through Route (DTŚ) in a shallow excavation, 7.0 m in width and 1.0 m in depth (figure 20). It was divided into 6 fields, 6 m × 6 m. The grit cushion in one field, in contrast to the others, was not strengthened by geogrids.

Fig. 20. Example 3. Testing field – shallow excavation in native subsoil

The subsoil down to the depth of 2.5 m consisted of uncontrolled embankment made of slag, stones, clays and silty clays. The cushion of the thickness of 35 cm was formed of sharp-edged uniform basalt grit of high strength.

The basic identification parameters of the material used for strengthening layer are given in table 14. They were obtained by standard testing methods: ϕ^* and c^* in direct shear apparatus (12 cm × 12 cm) and the modulus E^* in the test loads using a pile of pavement slabs.

Table 14

Some geotechnical parameters of grit

d_{50} [mm]	U [1]	C_c [1]	γ_S [kN/m^3]	ϕ^* [°]	c^* [kPa]	E^{**} [MPa]
13.2	1.17	1.006	26.0	41	0	20

The reinforcement of the embankment comprising a grit layer or a grit layer strengthened by two geogrids is shown in figure 21a, b.

The grit was laid on the native subsoil in three layers (5, 15 and 15 cm), each of them being carefully compacted by a static roller of the weight of 8 t. In the case of additional strengthening, a geogrid was put between the layers.

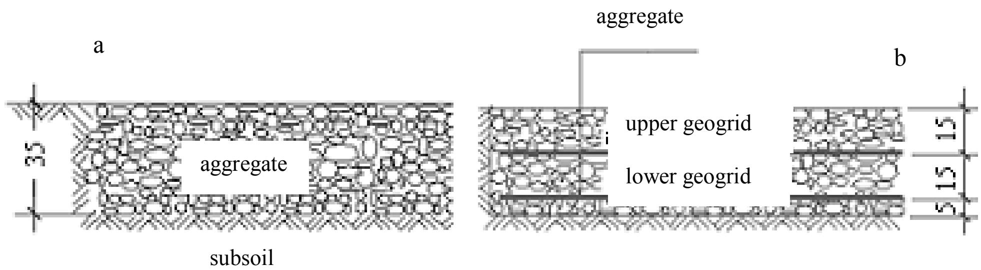

Fig. 21. Structure of the subsoil with strengthening layer: without reinforcement (a), with geogrids (b)

In the research programme, use was made of VSS plate test loads on the subsoil before and after its strengthening by a coarse material layer or by the same material but with additional geogrids. The load on the strengthening layer was imposed on the trace of the load used for unimproved subsoil. The layer and the subsoil were first loaded in steps, every step of 0.1 MPa (from 0.05 MPa up to 0.95 MPa), next they were unloaded in three steps, every of 0.05 MPa, and loaded again, every step of 0.2 MPa, up to the value of 0.95 MPa. The settlement was measured at three points, following the Polish Standard requirements for the time of a conventional settlement stabilization ($\Delta s_i \leq 0.05$ mm/2 min).

The results of the experiments in the range of primary loading are shown in figure 22 in the form of loading–settlement curve.

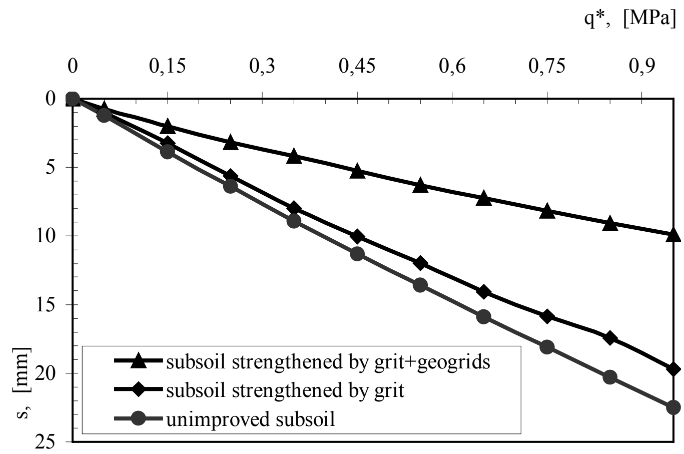

Fig. 22. Empirical loading–settlement relation

4.2. THEORETICAL INTERPRETATION OF THE RESULTS WITH THE USE OF THE CM/CM MODEL

The relations presented in figure 22 were subjected to numerical analysis. The strength and strain parameters of the unimproved subsoil and the subsoil strengthened by the coarse material layer with or without reinforcement were assessed by comparing the empirical and theoretical curves, obtained in incremental FEM analysis of the axisymmetric contact problem. Indispensable parameters of the subsoil and the strengthening layer were as follows: elastic modulus E, internal friction angle ϕ and cohesion c; in the case of reinforcement, we had to know an elastic modulus E. Poisson's coefficient ν was assumed to be 0.3 for the subsoil, 0.25 for the grit and 0.2 for the reinforcement. The fitting criterion for both curves was the condition of minimum deviation of the modified coefficient of determination R^2 from unity. The optimization process was performed by trial-and-error method. The values of elastic modulus for the subsoil and the grit were initially assessed with the use of linear elastic model. Hence, in the third experiment, the procedure was the same as in the first one.

In the numerical analysis, the subsoil and reinforcement were modelled by 276 eight-noded rectangular elements with slidable support along the vertical walls and fixed along the horizontal base. The dimensions of the discrete model, shown in figure 23 (B^*, H^*), were as follows: $H^* = 1.5$ m, $B^* = 1.8$ m. If $D = 0.3$ m we have $H^*/D = 5$ and $B^*/D = 6$. It was assumed that the foundation was the linear elastic material whose E_f = 9000 MPa and ν_f = 0.3, and the subsoil and grit fulfill the conditions of Coulomb–Mohr's model. The reinforcement was modelled using linear elastic model.

In the first stage of a three-stage analysis, the parameters for subsoil were chosen. In the second stage, a layered subsoil was analyzed, provided that the subsoil parameters from the previous stage were determined. And in the third stage, the parameters of the subsoil and the strengthening layer were known. In the first and the second stages, the sequence of searching was as follows:

- elastic modulus of the subsoil and the grit were assessed initially, assuming linear elastic model for the linear part of the loading–settlement relation,
- internal friction angle and cohesion were estimated assuming Coulomb–Mohr's model for the subsoil and for the coarse-grained material, appropriately reducing the searching range with simultaneous correction of the value of the elastic modulus assumed earlier.

In the third stage, the only variable was the elastic modulus of the reinforcement (with $\nu = 0.2$).

The values of the parameters of the subsoil, strengthening layer and reinforcement obtained in the experiments described above are shown in table 15. Loading–settlement relations obtained numerically and experimentally are presented in figures 24, 25 and 26.

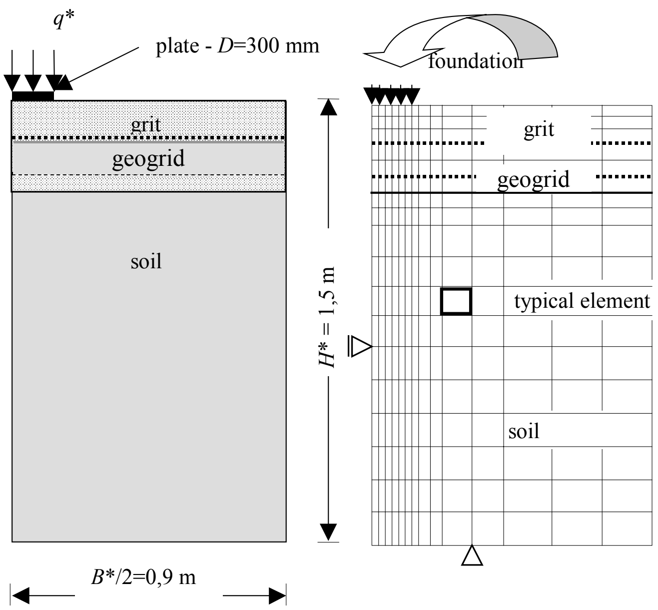

Fig. 23. Geometrical (a) and discrete (b) models of the system

Table 15

Numerically obtained model parameters for subsoil, strengthening layer and reinforcement

Model parameters	ϕ_p [°]	c_p [kPa]	E_p [MPa]	R^2 [1]
Soil	35	30	10	0.999
Grit	45	0.1	19	0.997
Geogrid	E = 450 MPa			0.990

Referring to the third experiment, it is worth indicating that the values of the strengthening layer parameters obtained numerically and analytically are well converged $\phi = 45°$ as compared to 41°, $c = 0.1$ kPa as compared to 0 kPa and $E = 19$ MPa as compared to $E = 20$ MPa.

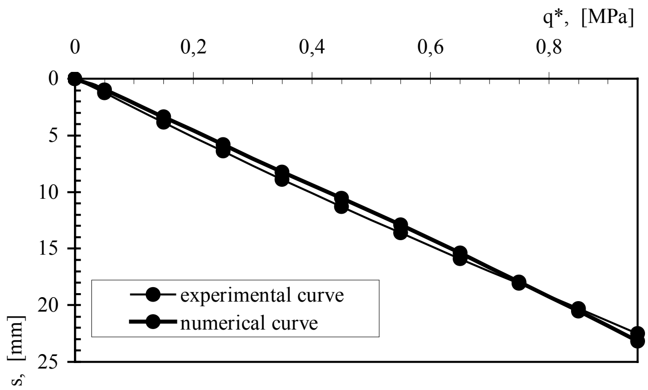

Fig. 24. Subsoil without strengthening. Loading–settlement relation

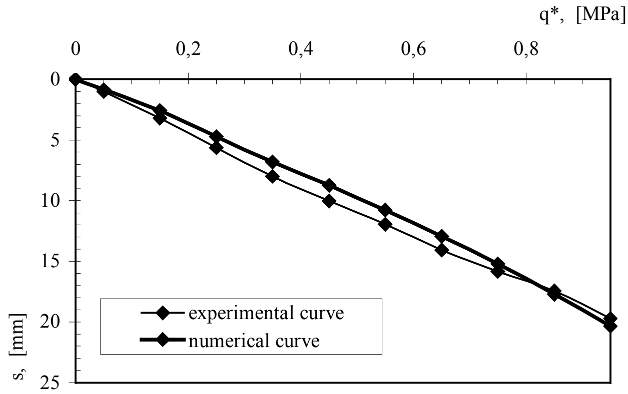

Fig. 25. Subsoil strengthened by the layer of grit. Loading–settlement relation

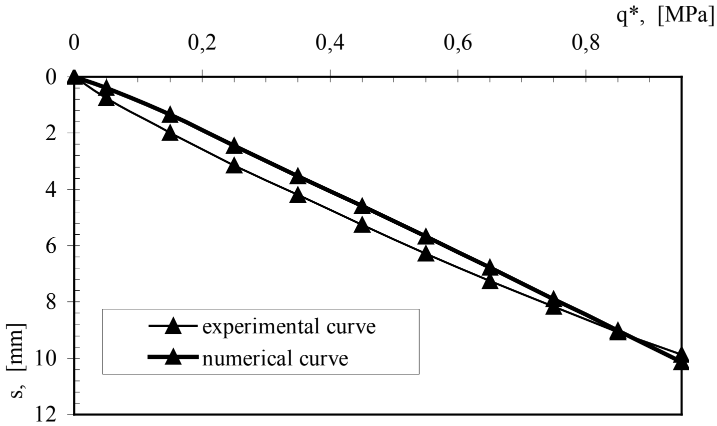

Fig. 26. Subsoil with reinforced layer. Loading–settlement relation

5. SUMMARY

The concept of using the elastic–ideal plastic Coulomb–Mohr's model for the description of soft subsoil–loadbearing cushion (layer) system was verified. To this end three different experiments were chosen. They differed in the testing scales (laboratory and field), system characteristics (2D and 3D problems), dimensions of the strengthening layer (limited or unlimited in respect of the loading dimensions) and purpose. The analysis was also extended to the case of the layer reinforced with geogrids.

The above concept was positively verified by the convergence of the loading–settlement curves obtained in empirical and numerical tests. The curves obtained numerically were determined by using known (from experiments or Polish Standards) or assumed (on the basis of a specified criterion) material parameters of the subsoil, strengthening cushion (layer) and reinforcement (geogrids).

Independently of all the differences mentioned above, the results may be acknowledged as very promising. This statement can be confirmed by the value of the coefficient R^2 which is never lower than 0.931. That means that the numerical curves representing the response of the subsoil strengthened by the sand or gravel cushion/layer correspond to the adequate experimental curves. It is clearly observed in the case where the plasticity effect becomes importand and the use of elastic models should be limited. To make the verification complete, material parameters determined in numerical analysis should also correspond to the parameters obtained in the experiments. The latter, as confirmed by the second experiment, may be estimated in the test maintaining the real work conditions of the sample in the subsoil.

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