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Load-bearing capacity of soil loaded with strip-shell foundations

Несущая способность основания, нагруженного ленточно-оболочечными фундаментами

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Ключевые слова: несущая способность; ленточные фундаменты; сжимаемые основания; оболочка

Abstract. The paper describes the special features of the calculation based on the load-bearing capacity of soil loaded with strip-shell foundations which offer high efficiency in construction of medium and high-rise buildings on strongly compressible soils. The necessity of this calculation is caused by the requirements of Building Regulations. Problem solution of load-bearing capacity of subsoil with a bent cylindric surface based on the known Prandtl Solution was considered. The new static solution to the Coulomb's wedge theory of load-bearing capacity of soil loaded with strip-shell foundations (SSF) indicating that taking into account a bent surface of the soil under the shell and the counterweight from strip foundations make it possible to reasonably increase the load-bearing capacity of soil.

Аннотация. В статье рассмотрены особенности расчета по несущей способности основания, нагруженного ленточно-оболочечными фундаментами, которые обладают высокой эффективностью при строительстве зданий средней и повышенной этажности на сильносжимаемых грунтовых основаниях. Необходимость разработки данного расчета обусловлена требованиями действующих норм. Рассмотрено решение задачи о несущей способности грунтового основания фундамента с криволинейной цилиндрической поверхностью на основе известного решения Прандтля. Получено новое статическое решение теории предельного равновесия грунтов для задачи о несущей способности грунтового основания ленточно-оболочечного фундаментама, показывающее, что учет криволинейной поверхности основания под оболочкой, с учетом пригруза от ленточных фундаментов, позволяют обоснованно увеличивать несущую способность основания.

Introduction

Strip-shell foundations (SSF) are widely used in construction of medium and high-rise buildings on strongly compressible soils; these are shallow foundations composed of cross sectional strip foundations joined by gently sloping shells formed on natural or artificial soils. These foundations are applicable due to their efficiency and lower calculations compared to traditional slab foundations. Earlier the authors described the analysis techniques for interaction of SSF and soil using the Winkler model [1]. The paper aims at setting forth the results on the soil-bed load-bearing capacity loaded across the curvilinear convex upwards surface and comparing them with load-bearing capacity of subsoil loaded with a flat footing – this is the main "core" solution presented in the Building Regulation 22.13330.2011 [2]. It is necessary to underline that as a special case, SSF composed of one shell may be used as a foundation for a single-aisle building.

Yet, it is necessary to complete the given techniques with analysis of the load-bearing capacity of soil loaded with SSF. This is due to the fact that the Building Regulation 22.13330.2011 [2] for soil analysis on both groups of ultimate limit states gives solutions for a single strip foundation. In case of other schematic views new solutions to the Coulomb's wedge theory (CWT) – the main theoretical basis for analysis of the load-bearing capacity should be found. For instance, many authors [3, 4, 5] including the foreign ones [6, 7, 8] formulated the problem on the load-bearing capacity of soil for foundations placed closely to each other taking into account their mutual influence on ultimate pressure magnitude.

In addition, in accordance with the Building Regulation 22.13330.2011 [2] the analysis based on the load-bearing capacity is required for strongly compressible soils, e.g. the soils composed of slowly compacting water-saturated clays and therefore, the new solution of CWT is a necessary requirement of the Building Regulations [9, 10].

This solution gives a certain ultimate pressure magnitude allowing the settlements to be analyzed using the finite element analysis (FEA) not only in the stage of linearly deformable soil, but also beyond it using the nonlinear soil models [11, 12]. Numerical solutions to this class of problems using FEA are given in [13, 14, 15].

There is also a considerable amount of foreign works dealing with the problems of the soil load-bearing capacity in terms of the static method of CWT [16, 17]. The papers of C.M. Martin and E.C.J. Hazell [18, 19] where the classical solutions of CWT obtained earlier are adapted to water-saturated soils should be highlighted.

Methods

Earlier the authors [20] considered the problem solution of the soil bed load-bearing capacity with a bent cylindric surface. Let us reproduce the basic calculation of the solution in terms of the strict static method of CWT:

$$\frac{\partial \sigma_x}{\partial x}+\frac{\partial \tau_{xz}}{\partial z}=0,\ \frac{\partial \tau_{xz}}{\partial x}+\frac{\partial \sigma_z}{\partial z}=\gamma;$$

$$\sqrt{(\sigma_x-\sigma_z)^2+4\tau_{xz}^2}=(\sigma_x+\sigma_z+2c\cdot\mathrm{ctg}\varphi)\sin\varphi, \tag{1}$$

where γ, φ and c – specific gravity, internal friction angle and specific cohesion.

The system of equations (1) was transformed to an accepted form – the equations drawn up by the characteristic lines being in agreement with the slide curves (Figure 1):

$$dx=dz\cdot\mathrm{tg}(\alpha\pm\mu),$$

$$d\sigma\pm 2\sigma\mathrm{tg}\varphi\cdot d\alpha=\gamma(dz\mp dx\cdot tg\varphi), \tag{2}$$

where $\sigma=\dfrac{\sigma_x+\sigma_z}{2}+c\cdot\mathrm{ctg}\varphi$ – average reduced stress; α – angle between the direction of the first main stress σ_1 and axis Oz; $\mu=\pi/4-\varphi/2$ – angle between the direction σ_1 and slide curves.

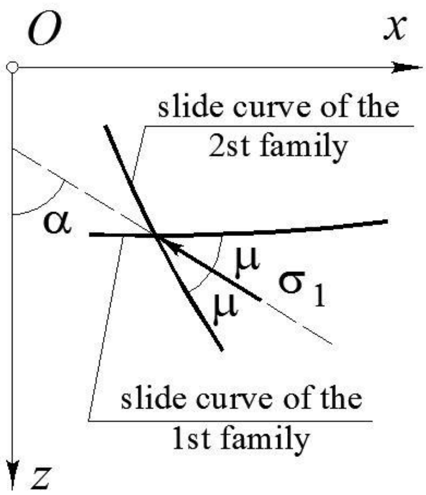


Figure 1. Relative orientation of the slide curves and direction σ_1

The solution for the specific schematic views was obtained by numerical integration of the accepted equations (2) by the method of finite differences using certain boundary value problems according to the algorithm given, e.g., in [21]. Components of the ultimate limit stresses were evaluated by the formulas:

$$\left.\begin{array}{l}\sigma_x\\ \sigma_z\end{array}\right\}=\sigma(1\mp\sin\varphi\cos 2\alpha)-c\cdot\operatorname{ctg}\varphi,\ \tau_{xz}=\sigma\sin\varphi\sin 2\alpha \qquad (3)$$

The schematic view of the given problem is shown in Figure 2.

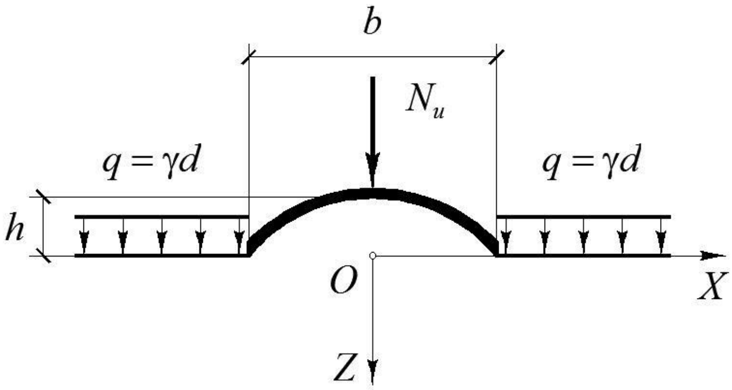


Figure 2. Schematic view (*d* – foundation depth)

The shell is of width *b* and height *h*. The equation of the bed is accepted as:

$$z=ax^2-h,$$

where *a* – geometric parameter, 4*h*/*b* – for the given diagram.

Taking the contact pressure equal to the value of natural pressure at the level of the foundation depth is some simplification which on the one hand takes into account the counterweight from strip foundations though ill-defined, on the other hand makes it possible to reveal the significance of the principal factors affecting the bearing capacity of soil bed loaded across the bent surface against the "basic" solution given in the Building Regulation under identical operating conditions.

Let us solve the problem using the strict static method of the Coulomb's wedge theory. The basic equations for the static method and the necessary explanations are given above. The order of boundary value problems is shown in Figure 3.

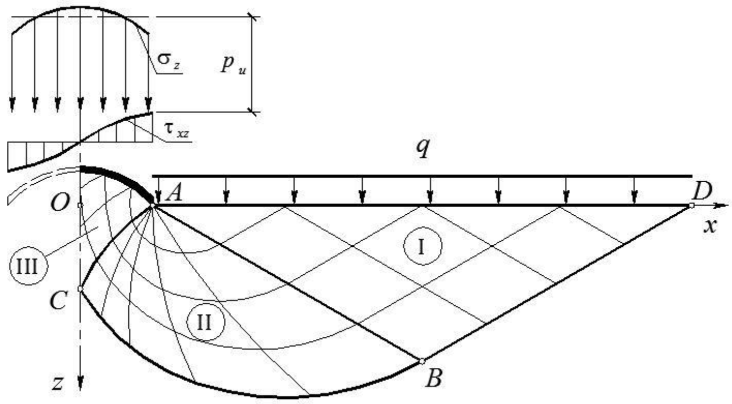


Figure 3. Grouping of boundary value problems (right half of the symmetrical schematic view)

In general, the solution agrees with the known Prandtl Solution [22, 23]. So let us describe the main details of the solution in short and the new features of it.

The ABD area is evaluated by the known integrals of the accepted equations of the static method (I boundary value problem):

$$\sigma = \frac{q+\gamma z + c\cdot \mathrm{ctg}\,\varphi}{1-\sin\varphi}, \ \alpha = \frac{\pi}{2}.$$

Let us remind that $\sigma = \dfrac{\sigma_x + \sigma_z}{2} + c\cdot\mathrm{ctg}\varphi$ – average reduced stress, α – angle between the direction σ_1 and axis *Oz*.

In the *ABC* radial sheaf one can solve the II boundary value problem with the known parameters of the accepted system of equations *x*, *z*, σ, α with *AB* characteristic line and conditions at the singular point *A*:

$$\sigma = \frac{q + c\cdot\mathrm{ctg}\,\varphi}{1-\sin\varphi} e^{(\pi-2\alpha)\mathrm{tg}\,\varphi}, \qquad \frac{\pi}{2} \ge \alpha \ge \alpha_F.$$

The boundary condition α_F and the length of the *AD* segment are sorted out on the assumption of meeting the conditions of symmetry at point *C*:

$$\alpha = 0, \quad x = 0,$$

i.e. the first main stress must be vertically oriented.

Here the value can’t be lower than

$$\alpha_{F,\min} = -\frac{\pi}{2} + \mu - \mathrm{arctg}\left(\frac{dz}{dx}\right)_{x=b/2}.$$

The value $\alpha_{F,\min}$ means that the leftmost characteristic line (from the side of the foundation) can appear at the singular point *A* at a tangent to the bent surface (here $dz/dx = 2ax = 4h/b$) or appear “inside” the surface, i.e. below the level of the bed.

When the *ABC* area is completed, two ways to analyze the ultimate load can be chosen. Firstly, knowing all the parameters of the accepted system of equations on the *AC* characteristic line, the area limited by the *AC* line below and the shell at the top can be seen; taking the balance of the area, the vertical component of the ultimate press force can be analyzed. The soil located in this area may be both in the compressed state from a theoretical point of view (the so-called rigid core) and in the ultimate state. Secondly, it is possible to build up the *ACC* area on the *AC* characteristic line and the axis of symmetry *Oz* by solving the III boundary value problem (Figure 4). In this case, the soil under the shell formally "becomes" ultimately stressed, but this fact doesn’t affect the value of the load-bearing capacity.

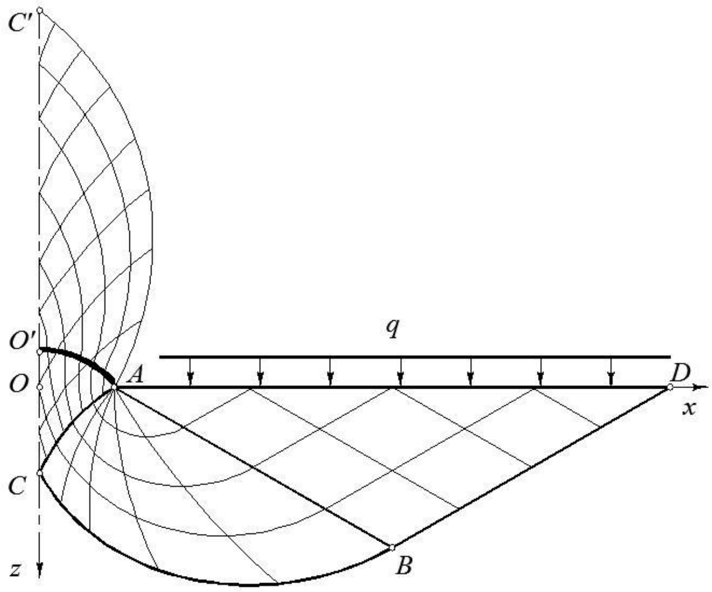


Figure 4. On building up the solution in the soil under the foundation bed

Soil conditions in the core under the foundation bed – conditionally “elastic” or ultimate – have been traditionally a matter of debate. When considering the strip-shell foundations there are more arguments for the "elastic" state. However, in this case it is much more difficult to obtain pressure diagram along the bed. In fact, this approach – ultimate limit state in the prism of a bulging area, radial sheaf and “elastic” state in the core under the foundation – was implemented by M.I. Gorbunov-Posadov. Due to considerable technical difficulties, this approach hasn’t widespread use, although in view of the availability of software systems that implement FEA, it seems possible to be implemented. The formal parameter of “elasticity” in the core can lead to the zones of destruction occurring in the stresses analyzed by the linear-deformable model, and hence it will lead to a new complication of the problem and new issues on compliance with the experimental data.

On the other part, ultimate limit stresses under the stamp make it possible to determine stress distribution diagrams along the bed; according to the numerous comparisons they agree with the experiments as to quality and quantity. Therefore, taking into account the conditionality of the ultimate limit state under the shell, let us integrate the accepted equations. Let us emphasize once again that the final result – the resultant ultimate limit pressure – is not affected, but allows determining the stresses along the bed.

Solution of the III boundary value problem, as discussed, will give the *ACC′* area which falls outside the limits of the soil surface (Figure 4). Let us do the following. The *AO′* line coinciding with the curvilinear foundation bed cuts off the “excess” part of *AO′C′*. The whole of the parameters of the accepted equations – x, z, σ, α are determined on this line, totally included in the *ACC ′* area. Thus, the contact vertical and horizontal stresses acting on the curvilinear shell can be analyzed using the known formulas:

$$\sigma_z = \sigma(1+\sin\varphi\cos 2\alpha) - c\cdot\mathrm{ctg}\,\varphi\,,\ \tau_{xz} = \sigma\sin\varphi\sin 2\alpha$$

This is the final problem solution.

Results

The results may be analyzed in relative variables: b – width unit, γb – stress unit. For added convenience, let us use the relative reduced stresses in the arguments. Thus, the relative counterweight and average ultimate limit pressure along the bed are as follows:

$$q' = \frac{q + c\cdot\mathrm{ctg}\,\varphi}{\gamma b}\,,\ p'_u = \frac{p_u + c\cdot\mathrm{ctg}\,\varphi}{\gamma b}$$

Figures 5 and 6 illustrate examples of the slide curves mesh (characteristic lines), vertical and horizontal stress distribution diagrams along the bed and the lateral counterweight. An interesting feature is needed to be pointed out: if the shell height increases, the diagram of vertical stresses (convex for a flat stamp) progressively flattens and in certain values it can even take a convex shape with low curvature.

Tables 1–4 show the values of the relative reduced forces N_u' ultimate limit pressure for internal friction angles φ = 10°, 20°, 30°, 40°, reduced values of lateral counterweights q' = 1, 2, 3, 5 and 10 and relative shell heights $\bar{h}$ = 0, …, 5 with spacing of 0.05. The main conclusion is obvious – the ultimate load significantly increases if the shell height $\bar{h}$ increases.

Attention is drawn to some reduction in N_u' values in low-level heights $\bar{h}$ and small values of the lateral counterweight q'. It is easily understandable, since in the absence of the counterweight the convex surface, of course, will carry loading less than the horizontal one until considerable horizontal forces perform, which, are formed due to the curvilinear surface of loading and in turn, further compress the soil; this is manifested in some values of $\bar{h}$.

It should be noted that stiffness of the shell itself and primarily tensile stiffness EA affects redistribution of the load transmitted to the soil bed through shells and strips and distribution of contact “reactive” pressures directly under the shell. Influence of the contact pressure distribution law under the bent surface of the shell can affect the values of horizontal forces and load-bearing capacity of the bed in a certain manner; however, this issue is the subject-matter of further research.

For greater clarity, let us illustrate the dependencies of the relative increase of the ultimate press force by the coefficient:

$$k = \frac{N'_u}{N'_{u,\text{СП}}},$$

where N_u' – relative reduced ultimate press force of SSF on soil, $N'_{u,\text{сп}}$ – relative reduced ultimate press force of a flat strip foundation which can be analyzed after the Building Regulation [2] (in Tables 1–4 the values are given in $\bar{h} = 0$):

$$N'_u = N_\gamma + q' N_q,$$

where N_γ, N_q – coefficients of the load-bearing capacity.

Dependency graphs $k(\bar{h}, q')$ for $\varphi = 10°, 20°, 30°, 40°$ are shown in Figures 7, 8.

a) b)

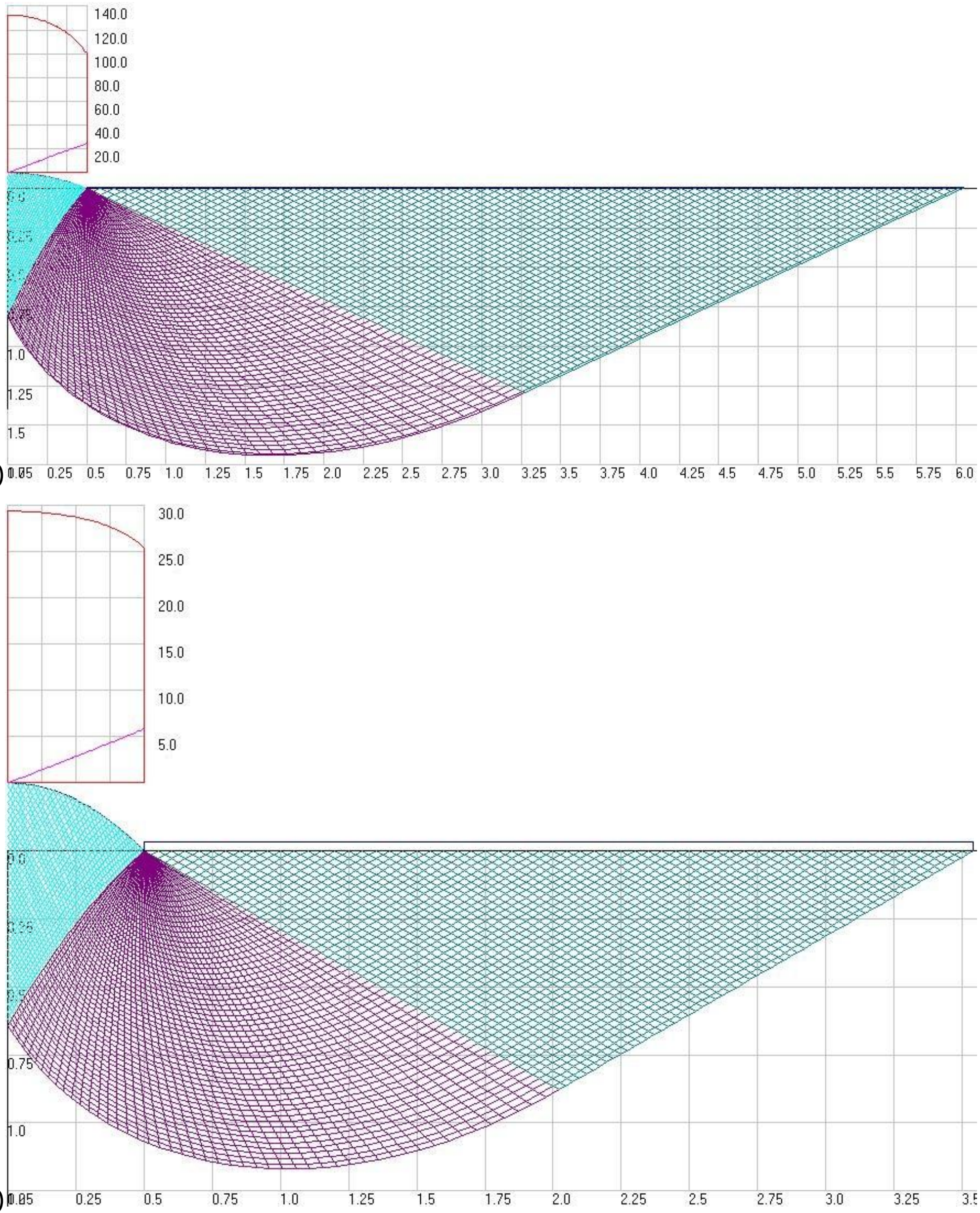
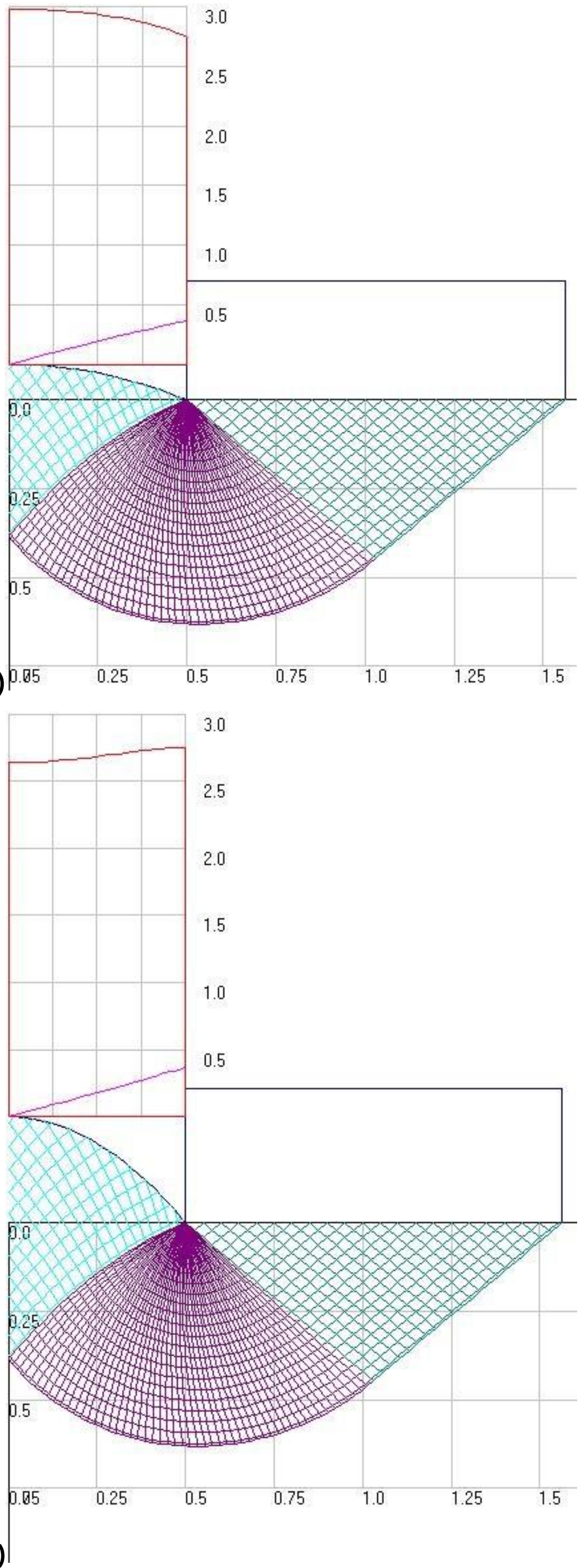


Figure 5. Examples of solution:
$\varphi = 40°$, $q' = 1$, $\bar{h} = 0.1$ (a); $\varphi = 30°$, $q' = 1$, $\bar{h} = 0.25$ (b)



a) b)

Figure 6. Examples of solution:

$\varphi = 10°$, $q' = 1$, $\bar{h} = 0.1$ (a); $\varphi = 10°$, $q' = 1$, $\bar{h} = 0.3$ (b)

The coefficients k do not considerably depend on the internal friction angle, but they significantly depend on the shell height and lateral counterweight.

Table 1. Relative reduced ultimate pressure forces for $\varphi = 10°$

$\bar{h}$	$q' = 1$	$q' = 2$	$q' = 3$	$q' = 5$	$q' = 10$
0	3.034	5.561	8.062	13.030	25.422
0.05	2.993	5.532	8.044	13.044	25.506
0.1	2.990	5.573	8.131	13.226	25.926
0.15	3.019	5.675	8.309	13.555	26.638
0.2	3.075	5.829	8.562	14.011	27.600
0.25	3.152	6.024	8.879	14.571	28.772
0.3	3.244	6.252	9.245	15.216	30.115
0.35	3.349	6.506	9.651	15.928	31.596
0.4	3.463	6.780	10.089	16.695	33.175
0.45	3.582	7.069	10.550	17.500	34.785
0.5	3.706	7.368	11.030	18.310	36.344

Table 2. Relative reduced ultimate pressure forces for $\varphi = 20°$

$\bar{h}$	$q' = 1$	$q' = 2$	$q' = 3$	$q' = 5$	$q' = 10$
0	9.159	15.812	22.329	35.282	67.502
0.05	9.049	15.714	22.271	35.275	67.599
0.1	9.075	15.852	22.527	35.775	68.716
0.15	9.197	16.163	23.033	36.675	70.606
0.2	9.400	16.620	23.750	37.915	73.160
0.25	9.668	17.197	24.641	39.439	76.269
0.3	9.987	17.869	25.672	41.192	79.830
0.35	10.346	18.616	26.814	43.129	83.760
0.4	10.734	19.421	28.044	45.214	87.990
0.45	11.144	20.270	29.342	47.417	92.454
0.5	11.570	21.153	30.693	49.711	97.036

Table 3. Relative reduced ultimate pressure forces for $\varphi = 30°$

$\bar{h}$	$q' = 1$	$q' = 2$	$q' = 3$	$q' = 5$	$q' = 10$
0	30.39	49.79	68.72	106.09	198.65
0.05	30.18	49.65	68.67	106.35	199.61
0.1	30.30	50.09	69.45	107.84	202.87
0.15	30.74	51.08	71.00	110.52	208.41
0.2	31.45	52.53	73.20	114.24	215.91
0.25	32.37	54.35	75.93	118.80	225.04
0.3	33.47	56.48	79.10	124.05	235.51
0.35	34.69	58.84	82.60	129.85	247.06
0.4	36.02	61.38	86.37	136.10	259.48
0.45	37.43	64.06	90.36	142.70	272.63
0.5	38.89	66.86	94.50	149.58	286.35

Table 4. Relative reduced ultimate pressure forces for $\varphi = 40°$

$\bar{h}$	q′ = 1	q′ = 2	q′ = 3	q′ = 5	q′ = 10
0	126.94	196.61	263.95	395.86	720.51
0.05	126.20	196.41	263.55	397.23	724.94
0.1	126.76	198.12	266.45	402.58	736.50
0.15	128.64	201.96	272.26	412.40	756.32
0.2	131.63	207.59	280.54	426.02	783.22
0.25	135.51	214.70	290.85	442.79	816.03
0.3	140.10	222.99	302.78	462.10	853.64
0.35	145.25	232.19	316.01	483.46	895.17
0.4	150.82	242.11	330.26	506.45	939.85
0.45	156.71	252.59	345.30	530.73	987.09
0.5	162.83	263.48	360.96	556.05	1036.41

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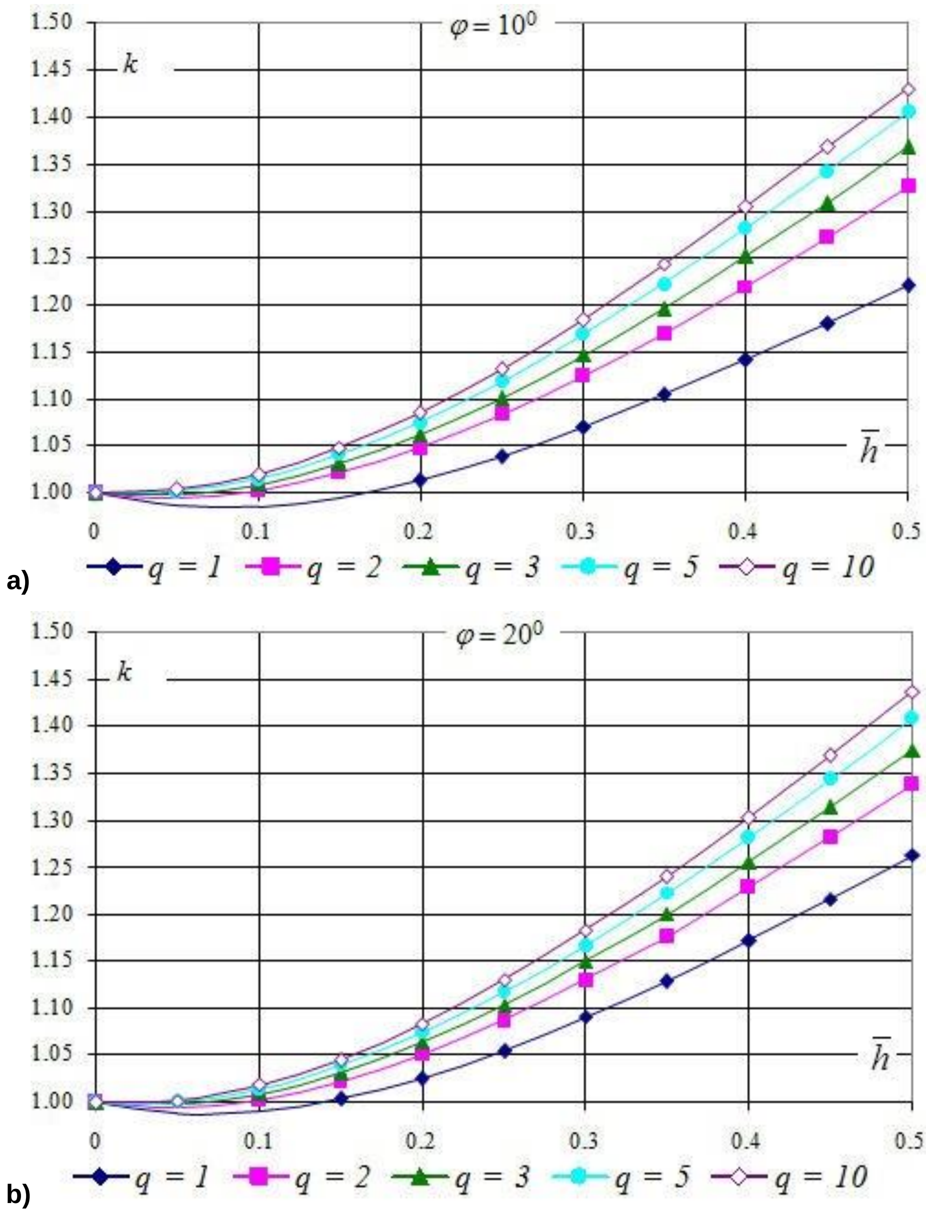

Figure 7. Dependencies of coefficients $k(\bar{h}, q')$ for $\varphi = 10^0, 20^0$

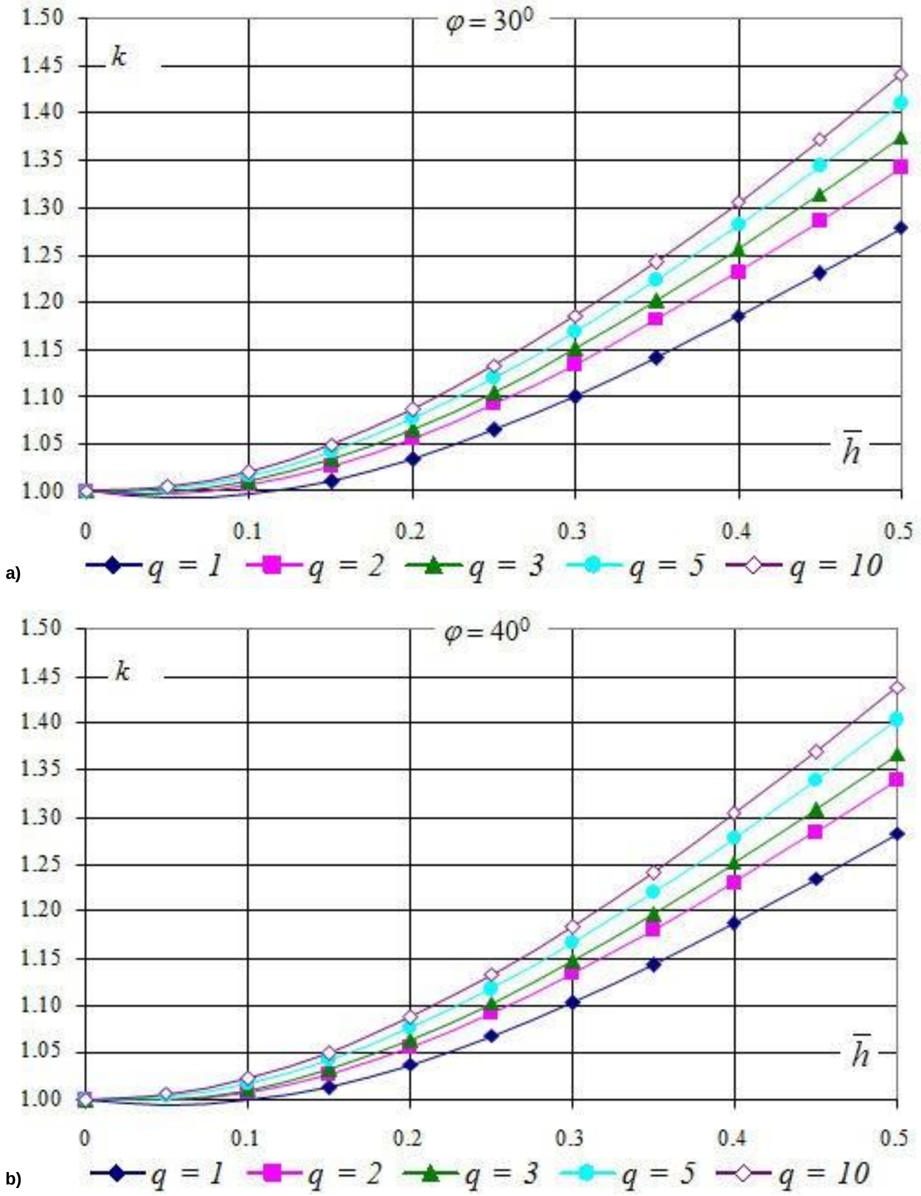

Figure 8. Dependencies of coefficients *k* ($\bar{h}$, *q′*) for φ = 30°, 40°

Thus, the ultimate pressure force on SSF may be evaluated by the following formulas:

– by coefficients *k* and values of the load-bearing capacity of flat foundation, *b* in width according to the Building Regulation [2]:

$$N_u = k \cdot b \cdot N_{u,\text{СП}} + (b - s) \cdot c \cdot \text{ctg}\,\varphi;$$

– by coefficients *k* and values given in tables 1 – 4:

$$N_u = \gamma b^2 N_u' - s \cdot c \cdot \text{ctg}\,\varphi$$

Here *s* – running meter area of SSF bed.

Conclusions

1. A new static solution to the Coulomb's wedge theory of load-bearing capacity of soil loaded with SSF indicating that taking into account a bent surface of the soil under the shell and the counterweight from strip foundations make it possible to reasonably increase the load-bearing capacity of soil.

2. It is stated that depending on the value of the bent surface height under the shell, within the gentleness criterion, the load-bearing capacity of soil can be increased up to 10 %. It is found out that the

load-bearing capacity of soil increases depending on the height of surface and counterweight pressure – q′.

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