Journal of Engineering and Applied Sciences 2 (7): 1111-1119, 2007

Design Method of Flexible Continuous Footings on Swelling Clayey Soils

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Abstract: This study analyses the behavior of swelling soils when they are moistened under buildings and structures. The methods and principles used currently for the design of structure foundations on swelling soils involve important problems due to non uniform deformations of these soils when subjected to the structure loads. In order to avoid the negative effects of swelling soils and to reach the desired performance on one hand, and the economical results on the other hand, the special computations of the foundations stiffness and deformability must take into consideration the prevention of the swelling soils feature and in some cases preserve it. The current study was elaborated in order to design flexible continuous footings on swelling soils taking into account the water content change on one hand and the contact pressure distribution on the footing on the other hand.

Key words: Clay, swelling, swelling magnitude, swelling pressure, continuous foundation

INTRODUCTION

In any geotechnical study relative to a construction project, the inflation of a soil has a character as important as its settlement. The dimensional variations, which result from this phenomenon, constitute a permanent challenge for the design and geotechnical engineers. The durability of the structure constructed on the swelling soils depends on the good appreciation of the phenomenon.

The swelling of the clayey soils, containing smectites or illites in variable quantity, is at the origin of numerous distresses in buildings and large structures. These disturbances are frequent in the regions with dry climate as some Caucasian parts, in Kazakhstan, in Algeria, in Morocco, etc.

These soils, called "inflating", can provoke important material damages, or even partial to total rupture of the structure, when they are not taken into account in the design of projects. It is therefore, important to foresee correctly the possible distortions of inflating soils, in amplitude and speed of evolution and to analyze their influence on the serviceability or on the stability of the structure.

The inflating soils have been a major concern for the designers for many years. Some construction procedures have been developed to limit the effects of inflation on the constructions and can be found notably in the classic works, of Mouroux *et al.* (1988) in French, Sorochan (1989) in Russian and Chen (1988) in English.

Currently, an abundant documentation explains the mechanisms of the inflation of the clays, as much in the microscopic level as in test-tubes tested in laboratory or in-situ soil. Nevertheless, the survey of the behavior of the structures in contact with inflating soils constitutes a complex task and the existing methods contain some insufficiencies. Most of the research carried out, is limited to the amplitudes of inflation of the clay soils in nature, under loads of superficial foundations. Little attention has been paid to the propagation of the inflation phenomena in the mass of the inflating soils as a function of time.

MATERIALS AND METHODS

The studied inflating clay comes from the Urban District of Baku (Azerbaijan) where it has provoked many disturbances in the structures of a concrete channel. The survey has been achieved on test-tubes of undisturbed clayey soil samples, dated of the Pliocenes collected from the Shamour channel". Apcheron", (Baku, Azerbaijan) by the laboratory of soil mechanics of the Institute of civil engineering of Baku within a research program (Fig. 1).

Mineralogical analysis: A diffractometry analysis by X-rays has been achieved to determine the mineralogy of the studied soils.

The physical and mechanical properties are given in Table 1.

The chemical analysis of the samples of inflating clays gave the following composition:

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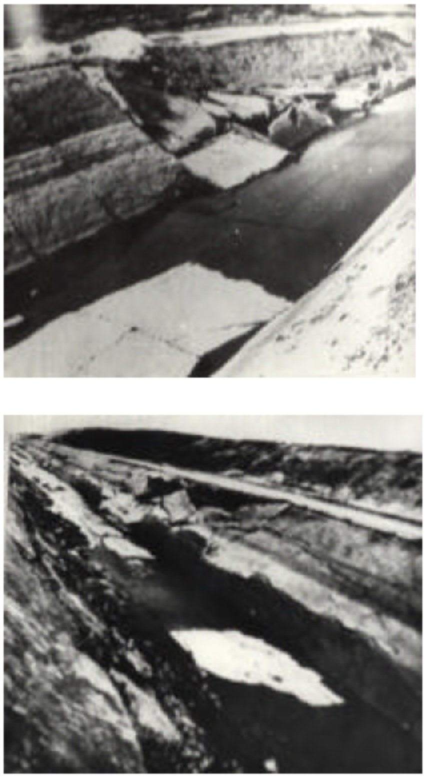

Fig. 1: Example of disorders in the construction of «Samour-Apchéron» concrete channel

Table 1: Physical and mechanical characteristics o of the studied clayey soils

The characteristics soil	Symbols	Unit	Values
Water content	W	%	10 -16
Degree of saturation	Sr	%	83-91
Wet unit weight	γ_h	kN m^{-3}	21.6-22.5
Dry unit weight	γ_d	kN m^{-3}	17.7-18.3
Specific unit weight	γ_s	kN m^{-3}	27.3-27.4
Voids ratio	e	--	0.496-0.542
Porosity	n	%	33-35
Liquid limit	W_L	%	46-51
Plastic limit	W_P	%	24-32
Plasticity index	I_P	--	19-22
Liquidity index	I_L	--	-0.41 to -0.74
Coefficient of compressibility	a_v	1/MPa	0.08
Modulus of distortion between 0.1-0.2MPa	E	Mpa	
At natural water content			7.0-7.8
After saturation			6.0-7.2
Cohesion	C	MPa	
At natural water content			0.2-0.58
After saturation			0.08-0.14
Internal friction angle	φ	Degree	
At natural water content			25-31
-After saturation			17-23
Grain size distribution			
0.5-0.25 mm	%		---
0.25-0.1 mm	%		---
0.1-0.05 mm	%		18.26
0.05-0.01 mm	%		23.58
0.01-0.005 mm	%		11.79
0,005-0.001 mm	%		46.37

Table 2: Chemical composition of the studied clays

No. of the sample	Units	Denomination Na+K	Ca^{++}	Mg^{++}	Cl	SO_4	HCO_3	CO_3	PH
1	%	13.88	1.49	0.33	7.3	6.59	1.19	0.63	7.5
2	%	12.18	1.69	0.33	7.30	5.42	0.89	0.59	7.8
3	%	13.88	1.29	0.25	7.30	6.84	0.89	0.39	7.8

SiO_2: 52.28 % **Al_2O_3: 15.27 %** **Na_2O: 2.73 %**
K_2O: 2.59 % **MgO: 2.45 %** **CaO: 6.70 %**
TiO2: 0.79 % **MnO_2: 0.10 %** **Fe_2O_3: 6.77 %**

The chemical analysis method by dosage of the elements existing in the buildings in clays has given the results of Table 2.

EXPERIMENTAL ANALYSIS AND INTERPRETATION OF THE RESULTS

The rate of inflation, corresponds to the relative variation of volume (in %) of a sample subjected to a non excitant overload or very low load (generally the weight of the piston in an oedometer) when it is put in contact with water with no pressure (Fig. 2). The pressure of inflation is constituted of an "osmotic" component due to the difference of concentration in salts of the interstitial water and a "matrix" component governed by the initial negative interstitial pressure of the sample that plays, in most cases a major role (Fig. 3).

Numerous methods have been proposed in the literature to evaluate the potential of a soil inflation from the measure of the parameters of plasticity and grading (Seed *et al*., 1962; Didier, 1972; Komornik and David, 1969; Vijayvergiya and Ghazzaly, 1973; Meisina, 1996, Chen, 1988; Sorochan, 1989).

For these authors, a very high inflation potential corresponds to a free inflation (expressed in percentage) superior to 25%, an high potential, to an inflation between 5 and 20%, a medium potential, to an inflation between 1.5 and 5% and a low potential, to an inflation lower to 1.5%. For Komornik and David (1969) the corresponding pressures of inflation are, respectively, superior to 300 kPa (potential very elevated), varying between 200 and 300 kPa (elevated), between 100 and 200 kPa (medium) and lower to 100 kPa (weak).

Several methods also exist to measure the pressure of inflation in oedometer, among these methods we quote:

- Method of Huder and Amberg (1970).
- Method of inflation with constant volume, according to the ASTM norm D 4546-90.
- Method of inflation or settlement under constant load, which requires several identical samples.
- Free inflation method followed by a reloading.

Fig. 2: Test of inflation in a free cell oedometer (without piston)

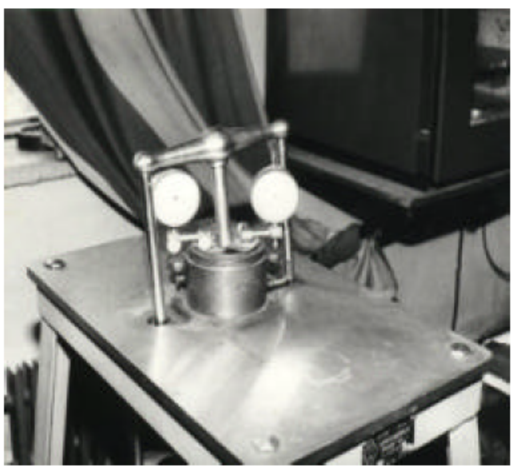

Fig. 3: Swelling test in an oedometer under loading

The experimental studies on the inflating soils (or expanding) show that the percentage of a soil inflation should increase proportionally with its density, its limit of liquidity, its contents clay, its indexes of plasticity and shrinkage, as well as its pressure of pre-consolidation (Seed *et al.*, 1962; Ranganatham and Satyanarayana, 1965; Vijayvergiya and Gazzaly, 1973; Sorochan, 1989). These same studies report that the pressure of inflation of an expanding soil should be inversely proportional to its natural water content.

The analysis of the experimental results (Beheddi and Mustafaev, 1990) allowed us to draw the curves giving the variation of the inflation potential in function of time for different values of the compression stresses. It also allowed us to establish the dependence of the water content after inflation in function of the different values of the compression stresses, the distortion of soils after their inflation in function of the different compression stresses as well as the variation of the inflation potential according to the water content in oedometer tests (Fig. 4).

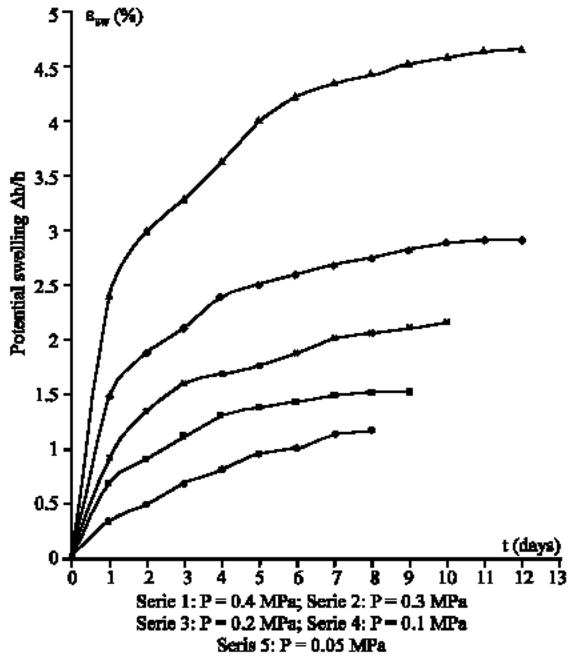

Fig. 4: Evaluation of the inflation potential versus time

The following study is relative to the mathematical description of the laws of behavior and inflation established from the oedometric testing of the samples.

The dependence of the water content inflating soil on the compression stresses after inflation has the shape of an exponential function, hence:

$$W_{SW} = W^{0}_{SW}\, e^{(-\gamma P)} \quad (1)$$

With:

W^0_{SW}: The content in water of a sample after swelling without load (P = 0)

and

$$\bar{\gamma} = \left(\frac{1}{P}\right)\ln\left(\frac{W^{0}_{SW}}{W_{SW}}\right) \quad (2)$$

For the interval of the stresses, that is generally from 50 kPa-400 kPa in civil and industrial constructions, the dependence between the water content of soil inflating and the compression stresses can be approximated by the relation.

$$W_{SW} = \bar{W}_{SW} - \aleph P \quad (3)$$

With:

$\bar{W}_{SW}$: Initial value of content in water taken from the graph $W_{SW} = f(p)$ (Fig. 5).

χ : Depends directly on the slope $W_{SW} = f(p)$ and for the study of soil by the oedometer, we obtain:

$$\bar{W}_{SW} = 0.24; \quad \chi = 0.2 MPa^{-1}$$

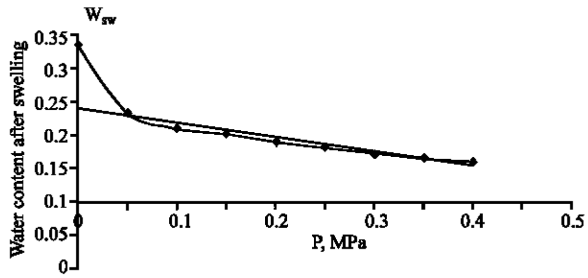

Fig. 5: Variation of the water content after inflation function of the different loads

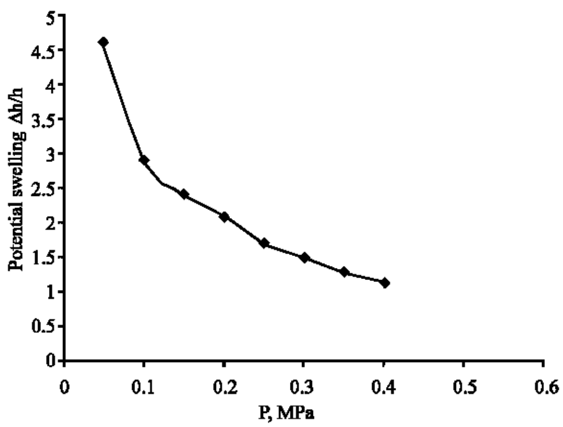

Fig. 6: Variation of the inflation potential function of the different loads

In this part, five models of inflation prediction according to the geotechnical properties of the inflating clays have been considered (Fig. 6). Their formulations are given in Table 3.

Further review of the models indicates that:

Some models do not have as parameter the content in natural water of soil: Models of Seed *et al.* (1962) first model of Vijayvergiya and Ghazzaly (1973).

The limitation of the domain of application of these formulas between a lower boundary-of the inflation amplitude from which the soil is qualified as inflating (5%) and an upper boundary mark is equal to the greatest percentage of inflation having been observed (60%). This allows us to determine, from these models, a minimum water content comparable to the shrinkage limit of and a content in maximal water comparable to liquid limit.

Mathematically, the models of prediction should constitute functional relations between a dependent variable, the percentage of inflation and some independent explicative variables (Fig. 7).

Proposition of a model for the swelling potential of the studied clays: The analytical results obtained show that, the mechanism of variation of the potential of soil inflation, dependent on the values of compression stresses and the variations of the contents in water in the process of its inflation, is governed by:

$$\varepsilon_{SW} = \varepsilon_{SW}^{0}\left(1 - \frac{P}{P_{SW}}\right)\left[(\bar{W}_{SW} - W_0) - \aleph P\right]. \qquad (4)$$

As it was showed by the analysis, this analytical expression leads to determine with a good precision the value of the inflation potential of the clayey soils on the basis of their features obtained from the tests on samples of soil inflating in oedometer molds. This formula contains the specific features of the inflating soils:

Table 3: Tested Models of prediction for the inflation of the clays

Model	Reference	Mathematical expression
Seed *et al.*-2	Seed *et al.* (1962)	$\varepsilon_{gonf} = 2{,}16.10^{-5} I_p^{2{,}44}$
Nayak and Christensen	Nayak and Christensen (1971)	$\lg(\varepsilon_{gonf}) = \frac{62.42\gamma_d + 0.65W_L - 130.5}{19.5}$
Vijayvergiya and Ghazaly-1	Vijayvergiya and Ghazzaly (1973)	$\varepsilon_{gonf} = 2.29.10^{-2} I_p^{1.45} \frac{C_2}{W_0} + 6.38$
Vijayvergiya and Ghazaly-2	Vijayvergiya and Ghazzaly (1973)	$\lg(\varepsilon_{gonf}) = \frac{0.4W_L - W_0 + 5.5}{12}$
Johnson-1	Johnson (1978)	Pour $I_p < 4$ $\varepsilon_{gonf} = -9.18 + 1.5546 I_P + 0.08424Z + 0.1W_0 - 0.0432 W_0 I_P - 0.01215 Z I_P$

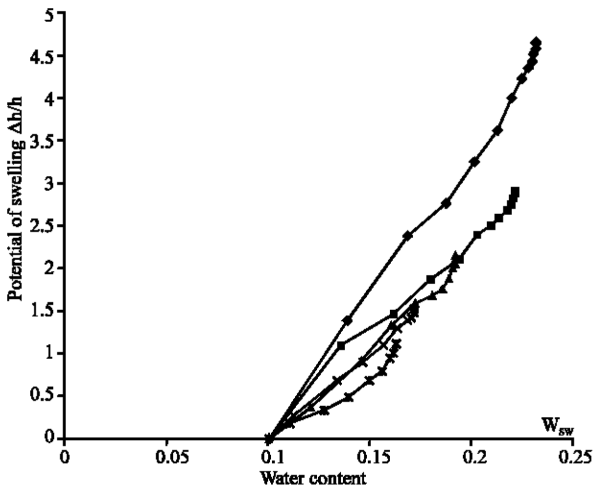

Fig. 7: Evolution of the inflation pontial function of the different loads

ε_{SW} : Potential of swelling.
ε^0_{SW} : Potential of swelling without loading.
P_{SW} : Pressure of swelling.
$\bar{W}_{SW}$: Initial value of water content given by the graph $W_{SW} = f(p)$ (Fig. 5).

The expression of the inflation potential is different from the one found in the literature by a good approximation of the non linearity of $\varepsilon_{SW} = f(p)$, so that the power of the P steers does not exceed 2.

CALCULATION OF THE FLEXIBLE STRIP FOUNDATIONS UNDER THE ACTION OF THE INFLATING SOILS

The superficial foundations on inflating soils must be designed by taking into account the capacity of these soils to inflate when their content in water increases. The design of the foundations on inflating soils must take into account the different possible shapes of soil distortion at the time of the wetting such as:

- Uplift of the foundation under the effect of the soil inflation;
- Drop of the foundation in the layer of soil that inflated because of the deterioration of these physical and mechanical properties caused by the wetting;
- Uplift of the foundation followed by its drop in the soil that inflated.

The inflating soils are characterized by an inflating pressure Psw, water content of inflation Wsw and a value of the inflation distortion (εsw under the imposed pressure P).

Given a flexible strip foundation of length L, with a constant bending rigidity on the whole length, the action of some outside loads q(x) and the layer under the footing composed of one clayey inflating soil, the interaction of the foundation base with the surface of inflating soil is determined according to the following proposed model (Mustafaev, 1989):

$$R(x) = K_S\, b\left[y(x) - \bar{S}_{sw}(x)\right] \tag{5}$$

Where:

R(x) : Reaction of the inflating soil pressure on the footing.
K_S : Coefficient of rigidity of the inflating soil.
b : The width of the foundation.
y(x) : The strain of the foundation.

The function $\bar{S}_{SW}(x)$ represents the variation in the vertical direction of the basis of the foundation to the upper surface of inflating soil, it is expressed as:

$$\bar{S}_{SW}(x) = S_{SW}^{max} - S_{SW}(x) \tag{6}$$

In this last expression S_{SW}^{max} is the final distortion after stabilization of the phenomenon of inflation that is concomitant with the minimal value of the contact pressure. $S_{SW}(x)$ is the non uniform distortion induced by the inflating soil on the basis of the foundation in function of the variation of the water content The action of the contact pressure on the base of the footing is in conformity with experimental research (Baheddi and Mustafaev, 1990).

$$S_{SW}(x) = S_{SW}^0\left\{\left[1 - \frac{P(x)}{P_{SW}}\right]\left[(\bar{W}_{SW} - W_0) - \aleph P(x)\right]\right\} \tag{7}$$

S^0_{SW} : Absolute amplitude of free swelling of soil, equal to: $S_{SW}^0 = \varepsilon_{SW}^0 . H_{SW}$ With
ε^0_{SW} : Free relative swelling,
H_{SW} : Thickness of the swelling soil layer.
P_{SW} : Pressure of swelling.
W_0 : Initial water content.
$\bar{W}_{SW}$: Initial value of the water content of the inflating soil, with P = 0

(Strait line that cuts the vertical axis taken from the diagram of the function $\varepsilon_{SW} = f(p)$ (Fig. 5).

$\aleph$: Slope of the straight line representing the diagram of the contents in water according to the compression stresses.

P(x): Distribution of the contact pressures under the base of the foundation, in correlation with the theoretical method of Simvoulidi (1987) concerning the calculation of a foundation on elastic soil.

$$P(x) = a_0 + \frac{2a_1}{L}(x - 0.5L) + \frac{4a_2}{L^2}(x - 0.5L) + \frac{8a_3}{L^3}(x - 0.5L)^3 \quad (8)$$

Where:

L : The length of the foundation

a_0, a_1, a_2, a_3: Known parameters depending on:

- Rigidity of the foundation.
- Length of the foundation.
- Module of soil distortion.
- Type and the location of the external load.

The indicative values of the parameters, in spite of their slight contribution facilitate the convenient calculations.

The differential equation of bending concerning the foundation will be written as follows:

$$EI\, y^{IV}(x) + K_S\, b\left[y(x) - \bar{S}_{sw}(x)\right] = q(x) \quad (9)$$

$$EI\, y^{IV}(x) + K_S\, b\, y(x) = q(x) + K_S\, b\, \bar{S}_{sw}(x) \quad (10)$$

Equation 10 can be used for bending of the footing on elastic soil, under the influence of an outside active load q (x) and complementary loads.

$q_{eq}(x) = K_S\, b\, \bar{S}_{sw}(x)$: The equivalent load on the footing of a non uniform distortion due to the contact of the surface of the footing with inflating soil after variation of its content in water.

Equation 10 can be written:

$$y^{IV}(x) + 4a^4 y(x) = \frac{1}{EI}\bar{q}(x) \quad (11)$$

Where:

$$a = \left(\frac{K_S\, b}{4E\, I}\right)^{\frac{1}{4}}; \qquad \bar{q}(x) = q(x) + q_{eq}(x)$$

For computational convenience, let us consider the non dimensional coordinates.

$\eta = ax$. In these conditions the derived functions (x) with respect to x and $\eta$are given by.

$$y'(x) = \frac{dy(x)}{dx} = \frac{dy(x)}{d\eta}\frac{d\eta}{dx} = a\frac{dy(\eta)}{d(\eta)}$$

$$y''(x) = \frac{d^2y(x)}{dx^2} = \frac{d^2y(x)}{d\eta^2}a\frac{d\eta}{dx} = a^2\frac{dy(\eta)}{d\eta^2}$$

$$y'''(x) = \frac{d^3y(x)}{dx^3} = \frac{d^3y(x)}{d\eta^3}a^2\frac{d\eta}{dx} = a^3\frac{d^3y(\eta)}{d\eta^3}$$

$$y^{IV}(x) = \frac{d^4y(x)}{dx^4} = \frac{d^4y(x)}{d\eta^4}a^3\frac{d\eta}{dx} = a^4\frac{d^4y(\eta)}{d\eta^4}$$

Equation 11 with the non dimensional coordinates takes the following form:

$$\frac{d^4y(\eta)}{d\eta^4} + 4y(\eta) = \frac{4}{K_S\, b}\bar{q}(\eta) \quad (12)$$

The boundary conditions of the problem are given by:

$$\left.\begin{matrix} y(0) = 0 & y\,(0) = \theta_0 \\ EI\, y\,(0) = -M_0 & EI\, y\;(0) = -Q_0 \end{matrix}\right\} \quad (13)$$

Where: y_0, θ_o, M_o, Q_o are initial parameters of the problem:

y_0: The strain of the footing.

θ_o: The angle of rotation.

M_o: Bending moment.

Q_o: The shear in the beginning of the section of the footing (with $\eta = 0$).

The mathematical formulation of the problem of the bending of strip foundation on a deformable basis due to inflation, leads to the resolution of the linear equation of the non uniform distortion of the fourth degree (11) with constant coefficients. In general the resolution of this equation is based on the global resolution of an uniform equation.

$$\frac{d^4y(\eta)}{d\eta^4} + 4a^4 y(\eta) = 0 \quad (14)$$

Equation 14 designates the particular solution of the heterogeneous Eq. 12.

The global resolution of the homogeneous Eq. 10, expressed with the help of the fundamental functions of Krylov (1930) $y_1(\eta)$; $y_2(\eta)$; $y_3(\eta)$ and $y_4(\eta)$ is:

$$y(\eta) = A\, y_1(\eta) + B\, y_2(\eta) + C\, y_3(\eta) + D\, y_4(\eta) \quad (15)$$

where: $y_1(\eta) = \cos\eta\, ch\eta$; $y_2(\eta) = \frac{1}{2}(\sin\eta\ ch\eta + \cos\eta\ sh\eta)$

$y_3(\eta) = \frac{1}{2}\sin\eta\ sh\eta$ $\quad y_4(\eta) = \frac{1}{4}(\sin\eta\ ch\eta - \cos\eta\ sh\eta)$

The function of Krylov satisfies the conditions of Cauchy and constitutes what is called the matrix unit:

$$\begin{array}{llll} y_1(0) = 1 & y_2(0) = 0 & y_3(0) = 0 & y_4(0) = 0 \\ y'_1(0) = 1 & y'_2(0) = 1 & y'_3(0) = 0 & y'_4(0) = 0 \\ y''_1(0) = 0 & y''_2(0) = 0 & y''_3(0) = 1 & y''_4(0) = 0 \\ y'''_1(0) = 1 & y'''_2(0) = 0 & y'''_3(0) = 0 & y'''_4(0) = 1 \end{array}$$

While using the matrix of reduction and boundary conditions (13), the constants of integration, A, B, C and D are then given by.

$$A = y_0 \quad B = \frac{1}{a}\theta_0;; \quad C = -\frac{1}{a^2EI}M_0; \quad D = -\frac{1}{a^3EI}Q_0$$

Considering the obtained expressions, the solution of the homogeneous Eq. 14 is written as:

$$y(\eta) = y_0y_1(\eta) + \frac{1}{a}\theta_0y_2(\eta) - \frac{M_0}{a^2EI}y_3(\eta) - \frac{Q_0}{a^3\ EI}y_4(\eta) \tag{16}$$

The fundamental functions $y_i(\eta)$ (i = 1; 2; 3 and 4) possess remarkable properties, they express themselves by successive products with precision until some constant coefficients repeat themselves, that is to say:

$$\begin{array}{ll} y'_1(\eta) = -4y_4(\eta); & y''_1(\eta) = -4y_3(\eta); \\ y'''_1(\eta) = -4y_2(\eta); & y_1^{IV}(\eta) = -4y_1(\eta). \\ y'_2(\eta) = y_1(\eta); & y''_2(\eta) = -4y_4(\eta); \\ y'''_2(\eta) = -4y_3(\eta); & y_2^{IV}(\eta) = -4y_2(\eta). \\ y'_3(\eta) = y_2(\eta); & y''_3(\eta) = y_1(\eta); \\ y'''_3(\eta) = -4y_4(\eta); & y_3^{IV}(\eta) = -4y_3(\eta). \\ y'_4(\eta) = y_3(\eta); & y''_4(\eta) = y_2(\eta); \\ y'''_1(\eta) = y_1(\eta); & y_4^{IV}(\eta) = -4y_4(\eta). \end{array}$$

The features of the Krylov (1930) function lead to simplifications in the solution of the problem, as for example the non necessity to express boundary conditions between separated sections of the foundation. This simplification leads to a unique solution of the shape Eq. 11. This solution permits to operate with any intermittent loads, with the exception of the one, where the unity matrix is reduced to zero with any type of anchorage of the foundations in the beginning of the study and when two out of the four unknown initial parameters are taken equal to zero.

The partial solution of the heterogeneous Eq. 12, considering the theory of Krylov (1930) takes the following shape:

$$\Phi(\eta) = \frac{4}{K_S\, b}\int_0^{\eta} y_4(\eta - \xi)\bar{q}(\xi)d\xi \tag{17}$$

The global solution of the problem of flexible foundations, free and supported by elastic compressible soil, ($M_0 = Q_0 = 0$), has the following expression:

$$y(\eta) = y_0y_1(\eta) + \frac{1}{a}\theta_0y_2(\eta) + \frac{4}{k_S\, b}\int_0^{\eta} y_4(\eta - \xi)\bar{q}(\xi)d\xi \tag{18}$$

The computation of the values of the reaction due to the pressure of soil, the bending moment and shear force, is carried out by means of expressions (19), (20) and (21):

$$P(\eta) = K_S\, b\left[y(\eta) - \bar{S}_{sw}(\eta)\right] \tag{19}$$

$$M(\eta) = -a^2EI\frac{d^2y(\eta)}{d\eta^2} \tag{20}$$

$$Q(\eta) = -a^3EI\frac{d^3y(\eta)}{d\eta^3} \tag{21}$$

APPLICATION

Given a flexible strip foundation of length L and section b×h, founded in a massif of inflating clay, subjected to a uniformly distributed load q_0= 15 kNm $^{-1}$ (Fig. 8), the coefficient of soil rigidity is $K_S = 1.104$ kN m^{-3}, the module of distortion of the footing is $E = 1.4\ 10^4$ Mpa.

A computer program for the determination of deformation of the footing (the deflection $y(\eta)$), the

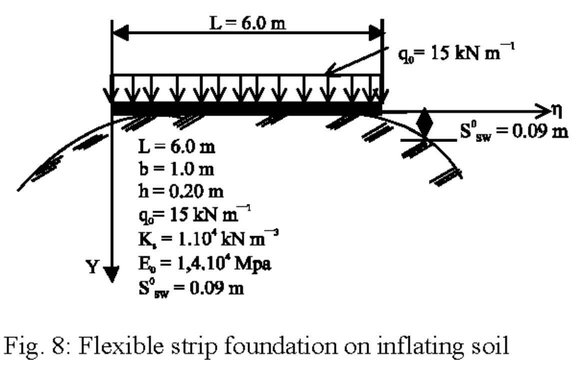

Fig. 8: Flexible strip foundation on inflating soil

Table 4: Summary of computation results

x, (m)	η, (m)	y(η), m)	P(η), kN/m	M(η), kN.m	Q(η), kN
0.0	0.0	0.071	---	0	0
0.6	0.243	0.053	----	27.09	76.67
1.2	0.486	0.035	26.35	79.50	90.49
1.8	0.729	0.020	60.28	129.33	72.17
2.4	0.972	0.011	73.54	163.50	40.50
3	1.215	0.008	76.61	177.09	0
3.6	1.450	0.011	73.54	163.50	-40.50
4.2	1.701	0.020	60.28	129.33	-72.17
4.8	1.944	0.035	26.35	79.50	-90.49
5.4	2.187	0.053	---	27.09	-76.67
6.0	2.430	0.071	---	0	0

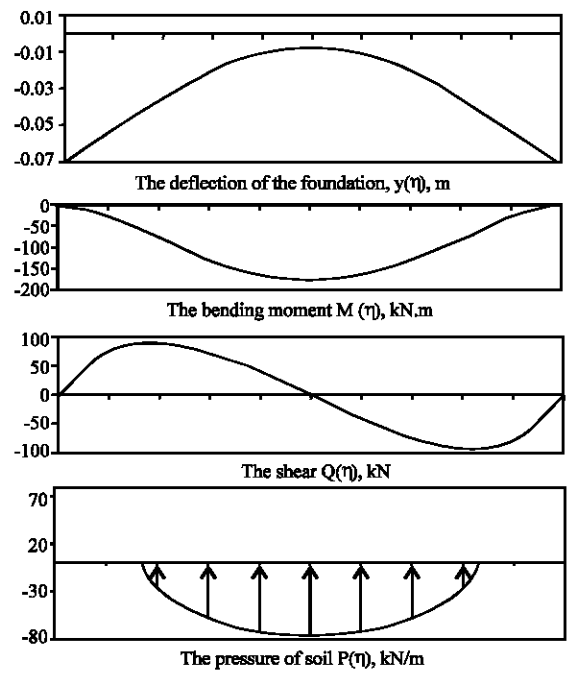

Fig. 9: Diagrams of the bending moment, shear force, soil pressure and deflection of the footing

bending moment M(η), the shear force Q(η) and the soil pressure P(η), was developed.

The results are regrouped in Table 4 and the diagrams on Fig. 9.

- Coefficient of rigidity

$$EI = 14.10^6 . \frac{1.0, 20^3}{12} = 9333 \text{kN m}^{-2}$$

$$a = \sqrt[4]{\frac{K_S . b}{4EI}} = \sqrt[4]{\frac{10^4 . 0,1}{4. 9333}} = 0.045$$

CONCLUSION

For the inflating soils, the designers are only interested in the measurable quantities that are generally the pressure and the amplitude of inflation. These values influence the choice of the foundation system. On the basis of the non linearity of the physical mechanism of distortion, the method developed in present work relates the phenomenon of inflation with the distribution of the contact pressures, the rigidity of the foundation and the soil of the work, as well as the value of the external load and its nature.

The proposed mathematical formulation has for objective identification of the different parameters influencing the behavior of a flexible strip foundation in contact with an inflating soil and then deriving its general solution.

The solution obtained in the studied real case corresponds to the method of Krylov expressed by (13).

The obtained results open new perspectives for the methods of conception of foundations on inflating clayey soils.

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