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CALCULATION THE UPLIFT OF ISOLATED PILES FOUNDED IN SWELLING SOIL

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ABSTRACT

Swelling soils are found in many regions throughout the word. The state of the practice in this area has been changing over the past decades. Design of foundations for expansive soils is an important challenge faced by engineers. The methods and principles currently used for the design of foundations on swelling soils involve important problems due to non-uniform deformations of these soils when subjected to structural loads. In situ and laboratory tests were used to investigate the most fundamental aspects. This article analyses the behaviour of a pile in a swelling soil when it is moistened. The tendency that develops at the present time, for the design of a pile in a swelling soil, consists in verifying the calculation of the bearing capacity of piles, taking into account the reduction of the resistance induced by the swelling soil on the lateral surface of the piles. This situation leads to an upward displacement of the pile, and, in case of excessive humidity the characteristic of the rigidity as well as the bearing capacity charge, which in this case decreases. An analytical approach of introducing a contribution, proposed method consists in calculating the rise of the pile, based on the study of the influence of a swelling clay type and the length of the pile.

KEYWORDS: Swelling Soil, Piles, Uplift, Tension

INTRODUCTION

When developing projects of buildings or structures on expansive soils, the possibility of wetting the soil, either by rain or by water from the soil, including leaks in pipes or reservoirs, must always be analyzed. The uplift of foundation at the swelling of clays in the distribution depends on the soil volume values of the vertical stress and the swelling pressure [1-3]. One of the methods that ensure normal exploitation of buildings and structure built on expansive soils consists of supporting structures partially or completely on pile foundations through the expansive soils. In this case, it is managed in redacting or completely eliminating the uprising of the building [4-6]. However, this result depends on the adaptation of the structure of the pile foundation to real conditions of the swelling soil. The use of piles in construction for centuries has accumulated numerous experimental data on the determination of experimental values of the friction forces on the lateral surface of the pile (q_s) and normal forces of resistance of the soil under the pile tip (q_p). Complete key data were published elsewhere [7-9]. Subsequently, these data were refined several times and used. Authors, [10-13] have developed more detailed tables for the values of q_s and q_p for a wider range of soils, applicable to the case of short cast in situ piles, with length less than 10 m. Sorochan [10] presents results of four tests conducted in swelling clays (clays Sarmat type (I) clay Khvalin type (II), clay-type quaternary (III), and clays such Aral type (IV)) on isolated drilled piles and groups of drilled pile with lengths ranging between 1.0 m and 7.0 m, and diameters between 0.40 m and 1.0 m of cylindrical form or enlarged at the base. In all cases, the optimal solution of pile foundation depends on the reliability of the method of calculating the combined behaviour of the foundation and the swelling soil. In accordance with Sorochan [10], the length of the pile will be determined from the conditions of the bearing capacity and the necessary conditions, so that the lifting does not exceed the tolerated value for the structure. Based on the experimental data [10 -16], the method of calculating the pile uplift depends on the type of expansive soil, the shape of the pile, the geometrical dimensions of the section, and the type of the pile (drilled or hammered).

Apart from this, the method of calculating the pile uplift is based on conventional interactions. That is why the experimental work on the ground is limited, and it is not allowed to extrapolate these conditions on short piles, which are best suited in this field. It is for this reason that the authors propose a method of calculating the uplift pile in expansive soils which is based on the analogy of the process of swelling due to expansion of solids and the differential equations of the thermoplastic theory.

DETERMINATION OF PILE UPLIFT

For determining the pile uplift, the case where the piles go completely across the layer of expansive soil is studied (Figure 1). In the case when the pile does not entirely go through the layer of soil, the total (global) lifting is added to the swelling of the layer of the soil which lies below the pile tip. As for the uplifting friction forces on the side surfaces, the calculation remain unchanged.

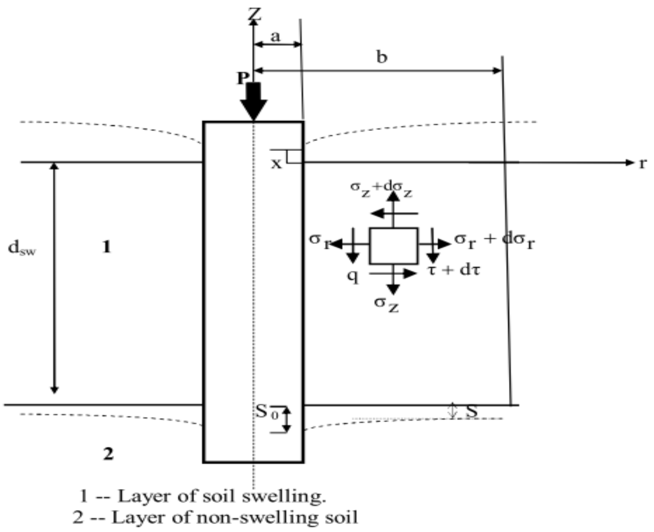

Figure 1: Representation the Pile Uplift

In general, the calculation is based on the analogy of the expansion process due to swelling of solids. In this function the intensity of the swelling is a function of the soil, f(r, z).

$$f(r,z) = \frac{dh_{sw}}{dz} = \alpha\, \theta(r,z) \quad (1)$$

Where:

hsw: Soil swelling height, (m)

α : Coefficient of linear expansion of the body, (degree -1).

θ : Change of body temperature, (degree).

r, z : Coordinates, (m).

The physical nature of the almost instantaneous, process of elastic bodies, that are heated layer by layer and after humidification of expansive soils, will be different. But the final results are the same; the expansion upon hearing of the solid body increases the volume of the soil after wetting. On the side surfaces of the pile, tangential tensions are distributed, in analogy with that the case of thermal stress.

DEFORMATION AND TENSION IN A SYSTEM OF PILES

The deformation and the tension in a soil-pile system are considered in equilibrium following the lifting of the swelled soil at a given time. The soil swelling is considered as a linear deformation of material having a modulus of deformation E and a Poisson ratio -ν-. The pile is considered as a cylinder of radius - a -, interacting with a layer of infinite thickness of swelling soil d_{sw}. The pile along its axis must resist to an applied force P, which is equal to the sum of the charge and the resistance force generated by soil layers. By analogy to the differential equations of the ax symmetric thermal state of infinite slab, the product α. θ, is replaced by the function f(r, z) given in (1).

The equilibrium equations of the displacement take the following form:

$$\left.\begin{aligned}&\Delta^2 u - \frac{u}{r^2} + \frac{1}{1-2\nu}\frac{\partial e}{\partial r} = \frac{2(1+\nu)}{1-2\nu}\frac{\partial f}{\partial r};\\&\Delta^2 \omega + \frac{1}{1-2\nu}\frac{\partial e}{\partial z} = \frac{2(1+\nu)}{1-2\nu}\frac{\partial f}{\partial z},\end{aligned}\right\} \qquad (2)$$

Where u, ω: correspond to the radial and vertical displacement (m) respectively.

e – Volumetric strain.

∇2 - Laplace operators in cylindrical coordinates.

In accordance with the general Hooke's law the stress state in a point (Figure 1) is:

$$\left.\begin{aligned}&\sigma_r = \frac{E}{1+\nu}\left[\frac{\partial u}{\partial r} + \frac{\nu}{1-2\nu}e - \frac{1+\nu}{1-2\nu}f\right];\\&\sigma_\theta = \frac{E}{1+\nu}\left[\frac{u}{r} + \frac{\nu}{1-2\nu}e - \frac{1+\nu}{1-2\nu}f\right];\\&\sigma_z = \frac{E}{1+\nu}\left[\frac{\partial \omega}{\partial z} + \frac{\nu}{1-2\nu}e - \frac{1+\nu}{1-2\nu}f\right];\\&\tau = \frac{E}{2(1+\nu)}\left[\frac{\partial u}{\partial z} + \frac{\partial \omega}{\partial r}\right].\end{aligned}\right\} \qquad (3)$$

In the absence of radial displacement of the pile, it is convenient to take $u = 0$. Therefore the system (3) takes the following form:

$$\left.\begin{aligned}
\sigma_r &= \sigma_0 = \frac{E}{(1+\nu)(1-2\nu)}\left[\nu\frac{\partial\omega}{\partial z} - (1+\nu)f\right];\\
\sigma_z &= \frac{E}{(1+\nu)(1-2\nu)}\left[(1-\nu)\frac{\partial\omega}{\partial z} - (1+\nu)f\right];\\
\tau &= \frac{E}{2(1+\nu)}\frac{\partial\omega}{\partial r}.
\end{aligned}\right\} \quad (4)$$

Substituting u = 0 in equation (2) yields:

$$\left.\begin{aligned}
&\frac{\partial^2\omega}{\partial z\partial r} = 2(1+\nu)\frac{\partial f}{\partial r};\\
&\nabla^2\omega + \frac{1}{1-2\nu}\frac{\partial^2\omega}{\partial z^2} = \frac{2(1+\nu)}{1-2\nu}\frac{\partial f}{\partial z}.
\end{aligned}\right\} \quad (5)$$

By integrating the first equation of (5) and substituting the second result into the second equation, we obtain:

$$\left.\begin{aligned}
&\omega = 2(1+\nu)F + C_1 z + C_2.\\
&\nabla^2 F = 0.
\end{aligned}\right\}; \quad (6)$$

Where $F = \int f(r,z)dz$.

Therefore, a harmonic function F is obtained which can be solved by using Bessel series of first and second levels multiplied by the trigonometric functions.

The function of the intensity of the swelling $f = \frac{\partial F}{\partial Z}$ should have a similar shape;

$$\left.\begin{aligned}
F &= \sum_{j=1}^{\infty}\left[A_j\sin(m_j z) + B_j\cos(m_j z)\right]\left[C_i I_0(im_j r) + D_j K_0(im_j r)\right] + f_0 z;\\
f &= \sum_{j=1}^{\infty} m_j\left[A_J\cos(m_j z) - B_j\sin(m_j z)\right]\left[C_j I_0(i m_j r) + D_j K_0(i m_j r)\right] + f_0.
\end{aligned}\right\} \quad (7)$$

Where: Aj, Bj, Cj, Dj, mj and f_0 : are constants.

I_0 (i mj r), K_0 (i mj r): BESSEL series respectively the first and second type of zero order of imaginary argument. j = 1, 2, 3...

Bessel function I_0 (i mj r) tends to infinite values for $r \to \infty$. Therefore, by studying the tension of the deformation of swelling layer, this must be excluded, that is to say Cj = 0. After this it follows taking Dj =1. Apart from that with $r \to \infty$, the intensity of swelling $f \neq 0$, which takes into account the introduction of the constant f_0. Its physical meaning is that apart from the effect on the function, the intensity of swelling is constant and equal to f_0 .

Substituting (7) and (6) into (4) we obtain the equations of motion and vertical stresses. For the case j =1, they have the following state.

$$\left.\begin{aligned} &\omega = 2(1+\nu)\left[A\sin(mz) + B\cos(mz)K_0(imr)\right] + z\left[2(1+\nu)f_0 + C_1\right] + C_2; \\ &\sigma_r = E\left\{\frac{\nu c_1}{(1+\nu)(1-2\nu)} - f_0 - m\left[A\cos(mz) - B\sin(mz)K_0(imr)\right]\right\}; \\ &\sigma_z = E\left\{\frac{(1-\nu)c_1}{(1+\nu)(1-2\nu)} + f_0 + m\left[A\cos(mz) - B\sin(mz)K_0(imr)\right]\right\}; \\ &\tau = -EmA\left[\sin(mz) + B\cos(mz)\right]. \end{aligned}\right\} \quad (8)$$

As the boundary conditions followings:

For $r \to \infty$ $\tau = 0$; $f = f_0$; $\omega\big|_{z=0} = -S$

For $z = d_{s\omega}$ $\sigma_z = 0$

For $r = a$ $\omega\big|_{z=d_{s\omega}} = x$; $\omega\big|_{z=0} = -S$,

Where S, S_0 are: soil compaction for $r \to \infty$ and $r = a$

If we take S =1mm, S_0 = 4 mm, clays type (I) S_0 =6 mm

x: Ground surface uplifting in the vicinity of the pile.

Substituting expression (8) into the boundary conditions, we obtain a system of equations, the resolution of which is that of case of a fixed pile. The constants are defined as follows.

$$\left.\begin{aligned} &C_1 = -f_0\frac{(1+\nu)(1-2\nu)}{1-\nu}; \\ &B = A\,ctg\,m\,d_{sw}; \\ &A = \left(S + S_0\right)tg(m)\frac{d_{sw}}{2}\left(1+\nu\right)K_0\left(ima\right). \end{aligned}\right\} \quad (9)$$

Taking into account the experimental data [15] the coefficient, $m = \dfrac{0,7\pi}{d_{sw}}$.

For these same data, the diagram of the distribution of vertical soil displacement ω (function of the depth z and wet thickness) has an exponential type relationship.

$$\omega = h_{sw}\, e^{(-\eta z)} \quad (10)$$

Where: η = 0.6 m-1 for type clays (I)

η = 0.4 m-1 for type clays (II)

η = 0.31 m-1 for type clays (III)

Assuming that the surfaces of diagrams drawn from experimental data and calculated ground displacement, the average intensity f_0 of the swelling of the soil is obtained.

$$f_0 = \frac{2(1-\nu)}{1+\nu}\left[h_{sw}\frac{1-e^{-\eta d_{sw}}}{\eta d_{sw}^2}+\frac{S_o}{d_{sw}}\right] \quad (11)$$

The uprising strength T acting on a motionless pile is:

$$T = -2\pi \; a E A K_1 (i m a). \quad (12)$$

Table 1: Comparison between T_c Calculated and Actual Values of T_f

Type of Clay	Length of Pile (m)	T_C; (kN)	T_f; (kN)	$\frac{T_C - T_f}{T_f} x100$ %
I	3	77.1	67	15
	4	91.8	83	11
	5	106.5	101	5
	6	120.2	124	-3
II	3	92.6	86	8
	5	127.8	142	-10
	7	160.7	183	-12
III	1	21.6	19	14

Table 1 shows the comparison between the forces of the uprising Tc calculated from the formula obtained and the actual values of Tf, taken from the work of [15]. In the calculations, the mechanical characteristics massive swelling clays are considered similar to those in a wet state:

Poisson's ratio ν = 0.3 ; the modulus of deformation E = 9 MPa for clays type (I) ; E = 5.4 MPa for type (II) clays end E = 3.6 MPa for type (III)clays.

During the determination of uprisings of piles, the known effect of braking "resistance movement" is used following the uprisings of uneven different layers of swelling soil. For this it is assumed that Z_0 is the coordinate of the soil layer where the displacements of the pile and of the soil coincide, that is to say ω = h.

Z_0 is defined from the equation of equilibrium of all the forces acting on the pile. After integration and transformation, equation (12) takes the following form:

$$T\left[\cos(m z_0) - \sin(m z_0) ctg(m d_{sw})\right] - 2\pi f_s z_0 = P \quad (13)$$

Where

f_s : the resistance of the soil layer below Z0 and is equal to 24 MPa for soil type II and III and equal to 58 MPa for soil type (I) according to [Sorochan, Trofimenkova] [15].

P: The load on the pile.

The expression (13) for Z0 = 0 becomes T = P. That is to say that the lifting force is equal to the load on the pile.

Replacing the value Z0 in the expression for the vertical non-mobile systems (8), a lift equal to h is obtained:

$$h=2(1+\nu)A\left[\sin(mz_0)+ctg(md_{sw})\cos(mz_0)\right]K_0(ima)+z_0\,f_0\frac{(1+\nu)}{(1-\nu)}-S_0 \tag{14}$$

In the case where the length "d" of the pile is less than the wet thickness dsw, lifting is determined as the sum of the displacements of the soil consequent to the action on the side surfaces h and the pile tip hT.

For this reason, in all formulas except the formula (11) -d- must be replaced by dsw.

Comparison of calculated displacements with those obtained experimentally.

The comparison of uplifting obtained by calculations to those obtained experimentally is given in Table 2.

Table 2: Results of Measured Uprisings of Pile According to the Method of Calculation and Experiments of Different Authors

Author of Experiment and Type of Soil Used	hsw (cm)	Length of the Pile. (m)	Depth Moisten (m)	Diameter of the Pile. (cm)	Load (kN)	Uplift of the PILE (cm)	
						Calculation	Experimental
E.A. Sorochan	14.5	2.5	3	20	50	2.4	2.1
	14.5	2.5	3	20	38	3.0	3.3
	14.5	2.5	3	20	30	5.1	4.0
Type (I)	14.5	2.5	3	20	22	5.2	4.2
	14.5	2.5	3	20	17	5.3	4.8
V.N. Boiem.	9	5	5	25 x 25	30	2,1	2.6
Type (II)	9	5	5	25 x 25	100	1,4	0.9
V.S. Sadjine Type (III)	19	3	8	25 x 25	50	14,9	13,5
	19	5	8	25 x 25	50	13,4	10,2
	19	5	8	25 x 25	100	10,8	9,8
E.A. Sorochan	19	5	8	25 x 25	150	8,9	8,8
	19	5	8	25 x 25	5	15,2	13,5
	8	2	3	20	47	6,6	3,8
	8	2	3	20	32	6,8	4,8
	8	2	3	20	18	7,0	5,4
TYPE (IV)	11	4	7	20	14	9,1	7,1
	11	4	8	20	25	7,6	6,1

It is to be noted from Table 2, that the results of calculations of uplift piles coincide to those obtained experimentally by the different authors.

CONCLUSIONS

In all cases, it is desirable that the piles cross completely the expansive soils or stop at a level at which the soil swells, when humidified, produce a permissible lifting of the structure. The method of calculating the pile uplift is based on swelling soil pile interactions. The authors propose a method of calculating the pile uplift in expansive soils based on the analogy of the process of swelling due to expansion of solids and the differential equations of the thermoplastic theory. From Tables 1 and 2, it appears that the results of calculations for pile lifting coincide with the experimental ones. It follows from the foregoing that one can always choose a vertical load P applied to the pile which must be greater than the active friction forces that appear on the lateral surface of the pile due to the lifting of the floor and therefore prevents the lifting of the pile to be occurred.

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