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Numerical Estimation of Settlement under a Shallow Foundation by the Pressuremeter Method

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Abstract

This work has two axes: The first one is theoretical (bibliographic analysis) on the theoretical estimation of the settlement under a Shallow foundation with the contribution of the characteristics of the results of the pressurometric tests and the second numerical axis by the numerical evaluation of the settlement generated by a superficial foundation that always happens by estimating the carrying capacity of these foundations by two methods the first is the classical method and the second the empirical method based on the direct interpretation of the in situ test such as the pressuremeter test by the determination Pressuremeter characteristics (Limit pressure (P_l) and the Pressuremeter module (E_M)), our contribution consists in using a calculation code based on the finite element method with the contribution of two laws of elastoplastic behavior namely Mohr-Coulomb and Cam- Clay we use geotechnical survey results project of the railway line project (Tissemesilt-Alger-Bughazoul) in Algeria.

Keywords: Settlement; Bearing Capacity; Shallow Foundation; Pressuremeter Test; Limit Pressure.

1. Introduction

The evaluation of settlements under shallow foundations and the estimation of carrying capacity are major and traditional problems in geotechnical medium and long term has therefore become more and more common, hence the need for reliable methods for the design and calculation of these foundations. So the estimate of these settlements generated by these foundations. Several methods have been developed to estimate the settlement that is directly related by the load capacity which can be evaluated by several methods namely the classical method (c and φ) [1], the methods from the results of the in-situ tests (pressuremeter test) and the numerical methods that the objective of this work are based in general on the finite element method. The main objective of this work is the contribution of the pressuremeter characteristics such as (the limit pressure and the pressuremeter module) in the numerical estimation of this settlement generated by these superficial foundations with two approaches for a railway station structure foundation.

2. Theoretical Synthesis

2.1. Calculation of Settlements by the Results of the Pressuremeter Test

The method of calculating settlements from the Ménard pressuremeter, proposed in issue 62, title V (1993) [10], is the calculation method originally proposed by Ménard. The pressuremeter module (E_M) is a deviatoric module, particularly suitable for calculating the settlement of foundations for which the deviatoric stress field is preponderant, namely the "narrow" foundations, such as the footings of buildings and structures [21] .the settlement formula is given

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by Equation 1.

$$s = S_c + S_d = \frac{\alpha}{9}\frac{q}{E_c}\lambda_c B + \frac{2}{9}\frac{q}{E_d}B_0\left(\frac{\lambda_d B}{B_0}\right)^{\alpha} \quad (1)$$

Where q is bearing capacity, λ_c et λ_d are form coefficients, given in Table 1, α is rheological coefficient, depending on the nature, the structure of the soil (or the rock) and the time, given in Table 2, B is width (or diameter) of the foundation, B_0 is reference dimension equal to 0.60 m and E_c et E_d are Equivalent pressuremeter modules in the volume zone and the deviator zone, respectively.

Table 1. Values λ_c *and* λ_d

L/B	Circle	Square	2	3	5	20
λ_c	1	1.10	1.20	1.30	1.40	1.50
λ_d	1	1.12	1.53	1.78	2.14	2.64

Table 2. Rheological coefficient α for clays, silts and sands

Nature of the soil	Clay		Silt		Sand	
Soil condition	E_M/P_{lim}	α	E_M/P_{lim}	α	E_M/P_{lim}	α
Overconsolidated or very tight	>16	1	>14	2/3	>12	1/2
Normally consolidated or tight	9-16	2/3	8-14	1/2	7-12	1/3
Underconsolidated and remolded or loose	7-9	2/3	5-8	1/2	5-7	1/3

Figure 1 shows the soil layers taken into account in the weighting of pressuremeter modules (E_M) for Equations 1 and 2. The E_c module is that of layer 1. The (E_d) module is calculated with the following weights:

$$\frac{1}{E_d} = \frac{0.25}{E_1} + \frac{0.3}{E_2} + \frac{0.25}{E_{3.5}} + \frac{0.1}{E_{6\,à\,8}} + \frac{0.1}{E_{9\,à16}} \quad (2)$$

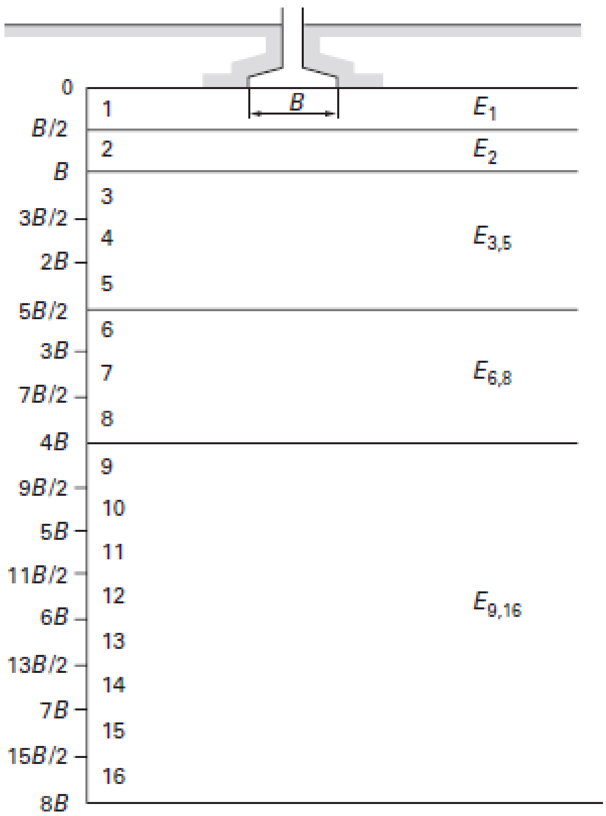


Figure 1. Weighting diagram of pressuremeter modules [19]

So the determination of the settlement is one function of the bearing capacity according to the Equation 1 for this la. There are different methods to estimate the carrying capacity of the superficial foundations namely the method (c and φ) is probably the method better known of soil mechanics, with the expression proposed by Terzaghi [1]:

$$q_u = \frac{\gamma B}{2} N_\gamma + cN_c + qN_q \tag{3}$$

N_γ, N_c, and N_q These are lift factors. Table 1 summarizes all these factors given by the different authors.

Table 3. Summarizes all the load factors

Authors	Year	Nγ	N$_c$
(Terzaghi)	1943	$N_\gamma = 0{,}5tan\varphi\left(\frac{K_{p\gamma}}{cos^2\varphi} - 1\right)$	$N_c = (N_q - 1)cot\varphi$
(Meyerhof)	1963	$N_\gamma = (N_q - 1)tan(1{,}4\varphi)$	$N_c = (N_q - 1)cot\varphi$
(Caquot et Kérisel)	1966	$N_\gamma = \frac{cos\left(\frac{\pi}{4} - \frac{\varphi}{2}\right)}{2sin^2\left(\frac{\pi}{4} + \frac{\varphi}{2}\right)}\left[k_p - sin\left(\frac{\pi}{4} - \frac{\varphi}{2}\right)\right]$	$N_c = (N_q - 1)cot\varphi$
(Brinch-Hansen)	1970	$N_\gamma = 1{,}5(N_q - 1)tan\varphi$	$N_c = (N_q - 1)cot\varphi$
(Brinch Hansen)	1961	$N_\gamma = 1{,}8(N_q - 1)tan\varphi$	$N_c = (N_q - 1)cot\varphi$
(Vesic)	1973	$N_\gamma = 2(N_q + 1)tan\varphi$	$N_c = (N_q - 1)cot\varphi$
(Giroud, and al)	1973	$N_\gamma = 0{,}5tan\varphi\left(\frac{K_{p\gamma}}{cos^2\varphi} - 1\right)$	$N_c = (N_q - 1)cot\varphi$
(Chen)	1975	$N_\gamma = 2(N_q + 1)tan\varphi tan\left(\frac{\pi}{4} + \frac{\varphi}{5}\right)$	$N_c = (N_q - 1)cot\varphi$
(Lebègue)	1981	$N_\gamma = \frac{cos\left(\frac{\pi}{4} - \frac{\varphi}{2}\right)}{2sin^2\left(\frac{\pi}{4} + \frac{\varphi}{2}\right)}\left[k_p - sin\left(\frac{\pi}{4} - \frac{\varphi}{2}\right)\right]$	$N_c = (N_q - 1)cot\varphi$
(Sanglerat and Costet)	1983	$N_\gamma = (N_q - 1)tan(1{,}4\varphi)$	$N_c = (N_q - 1)cot\varphi$

As well as the pressuremeter method of Ménard .This test is to realize the horizontal expansion of a cylindrical probe in a borehole at a given depth, under radial stresses until the rupture of the soil. It allows to obtain a relation between applied stresses and horizontal displacements of the borehole, which is a great advantage over other in-situ tests, as it allows the analysis of soil behavior in both small displacements and fracture. [8, 24]; thus the contribution of Louis Ménard (the miracle inventor) has mainly consisted in defining the pressuremeter characteristics of soils (limit pressure intervenes in the calculations of stability of the foundations (the bearing strength of the superficial foundations) and the pressuremeter module is used for the calculations of settlement) [4, 9, 13, 16, 20]. According to fascicle 62, the estimate of the breaking stress from the limit pressure [10, 22, 23], is in the following form:

$$q_l = q_0 + k_p p_{le}^* \tag{4}$$

3. Digital Study

For the purposes mentioned above, the following methodology is chosen in a scientific way, which is shown in Figure 2, our contribution consists in using a calculation code based on the finite element method. The problem is treated in plane strain in the plane (O, X, Y), the geometry is detailed in Figure 3, for the superficial foundation, and Figure 4 for the pressuremeter test. The construction of the model comprises several steps: the model construction, the mesh, the definition of the boundary conditions and the initial conditions are geostatic constraints representing the weight of the soil layers with a coefficient of resting earth pressure K_0, obtained by the formula of ($K_0 = 1 - \sin\varphi$) [18]. The soil of our study is considered as an infinite, homogeneous, isotropic mass. Its behavior follows two elastoplastic laws with Mohr-Coulomb criterion defined by (E, ν, c, φ, ψ) and Cam-Clay defined by (E, ν, λ, k, M, e_0, p_{co}) [11, 12, 15, 24]. As the geotechnical characteristics are grouped in table 4 we benefited from the results of the geotechnical survey in the region of Tissemesilt (Algeria), to realize a project of the railway line (Tissemesilt-Alger -Boughazoul).

For the pressuremeter the mode of deformation of the pressuremeter probe is plane deformation. In axisymmetric condition, in small deformations in drained conditions (in effective stresses) the soil is free on the vertical walls of the borehole and a vertical displacement is possible on the two vertical boundaries. For loading, a load applied by the probe on the ground. This type of loading is applied radially over a length equal to the length of the probe, downhole. [14, 24-29].

Table 4. Summarizes the geotechnical parameters adopted for the study

	Parameters	Unit	Silty clay	Shallow foundation
Behavior			Elasto-plastic	Elastic
Model	Distance	(m)	-	
Mohr-Coulomb	γ	(kN/m^3)	16	
	E	(MPa)	10	40000
	v	-	0.3	0.3
	φ	(°)	20°	
	C	(kPa)	15	
	ψ	(°)	0^0	
Cam-Clay	λ	-	0,155	
	k	-	0,052	
	M	-	0.70	
	e_0	-	0.680	
	p_{co}	(kPa)	0	

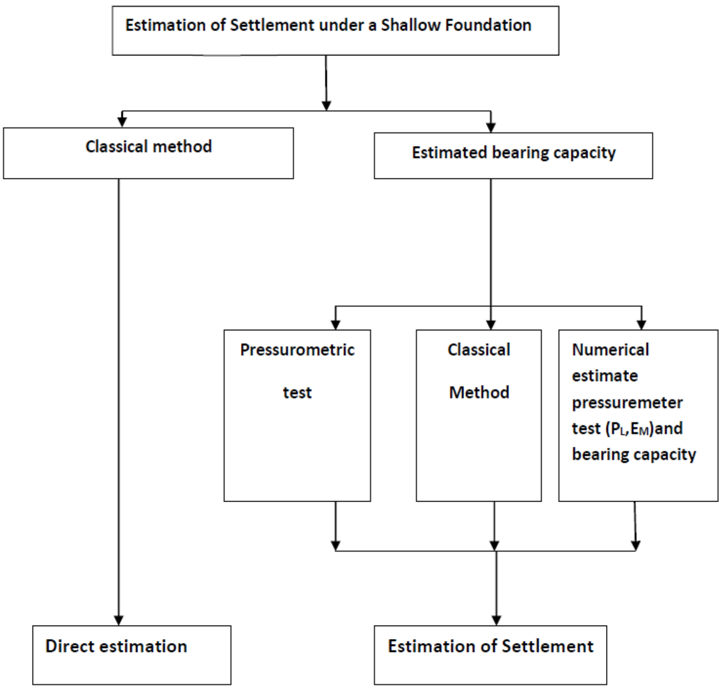
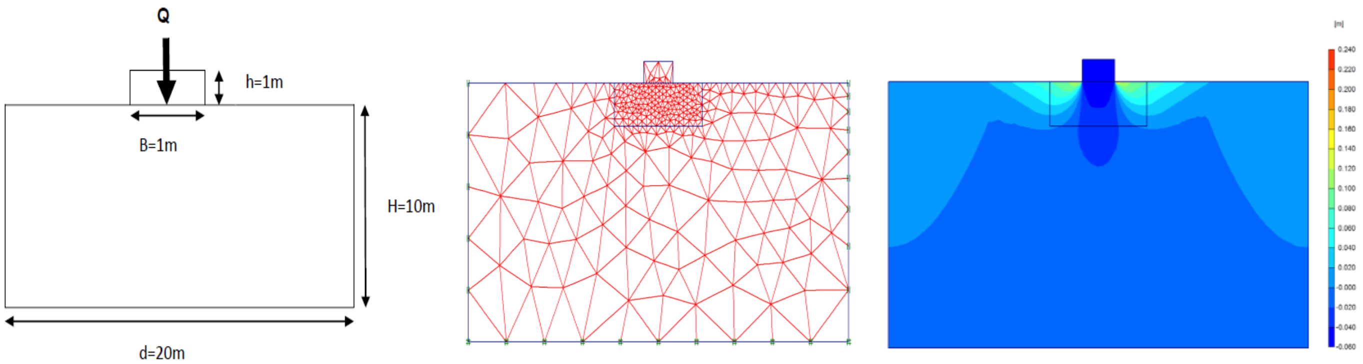
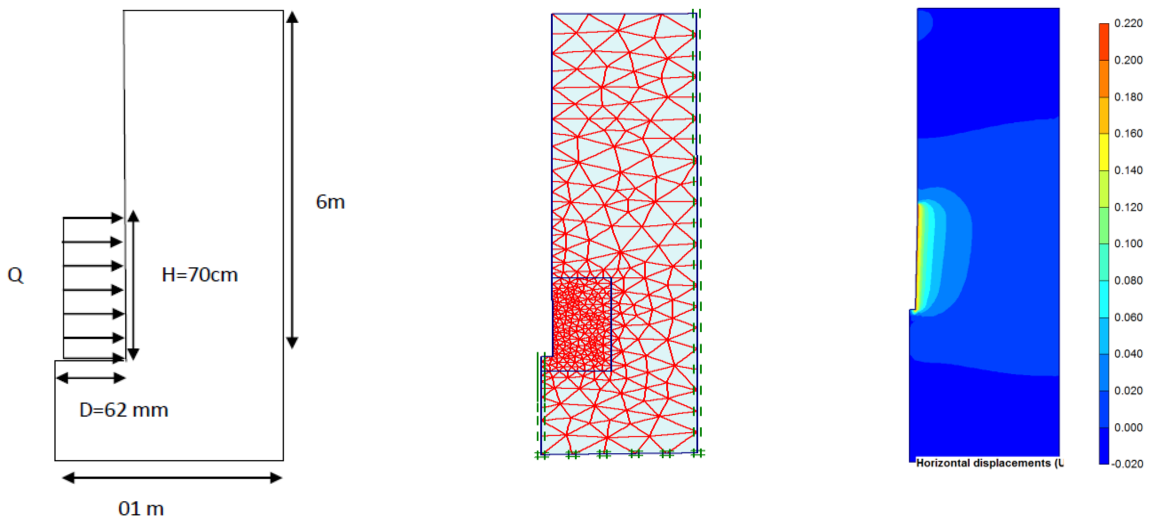


Figure 2. Methodology chosen to estimate settlement under a shallow foundation



a- Geometry of the problem **b- Optimal mesh** **c-Deformation of the massif**

Figure 3. Shallow foundation



a- Geometry of the problem **b- Optimal mesh** **c-Deformation of the massif**

Figure 4. Pressuremeter test

4. Results and Discussions

For the estimation of the bearing capacity the simulations are carried out in plane deformation condition. The foundation is, rough it has a width of 2 meters and a height of 1 meter, its behavior is linear elastic Figure 3.c, shows the mechanism of rupture of a superficial foundation under a vertical load .the two versons are close together to them which explains the increase of the bearing capacity for a horizontal mass After having plotted the various curves of the evolution of the bearing capacity, the results are presented in the form of curves giving the relative vertical settlement s / B in%) according to the average stress under the foundation q. Mohr-coulomb) is found to have a higher bearing capacity than the Cam-Clay model (fine soil). Qualitatively, however, for the shape of the curve (Figure 5).

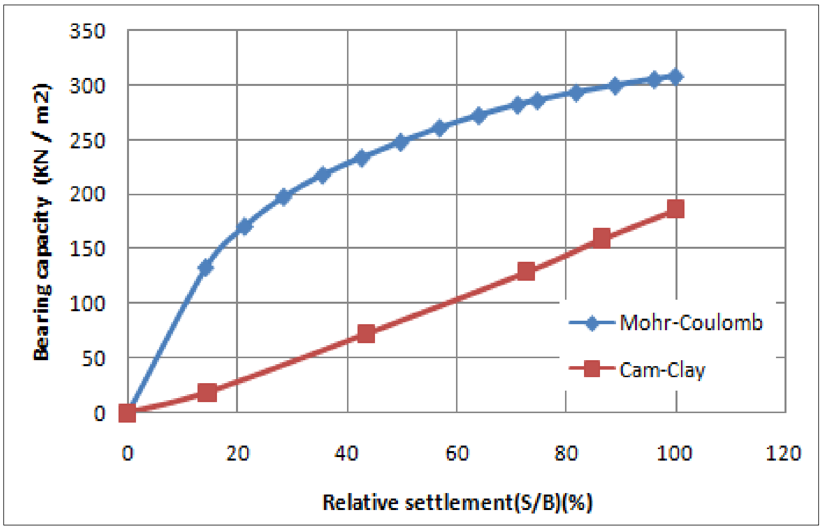
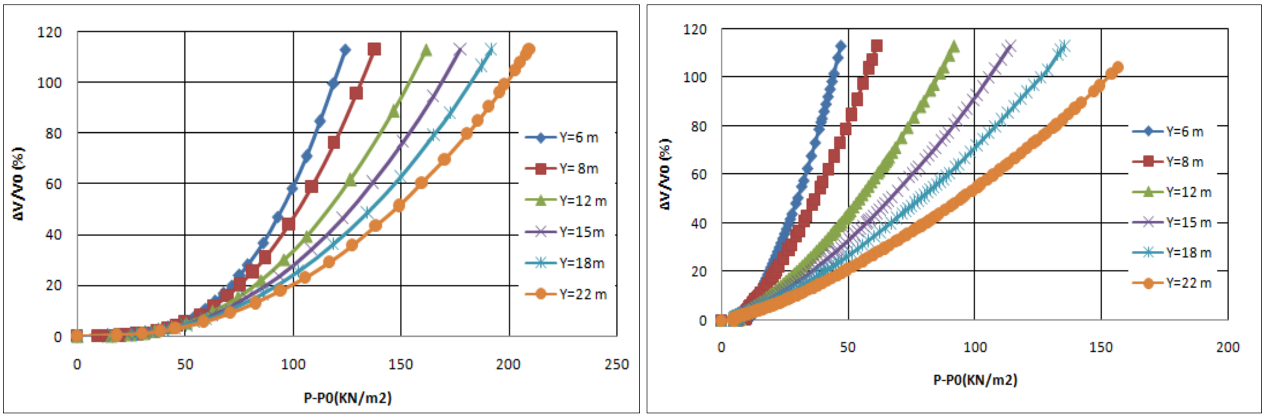


Figure 5. Evolution of the bearing capacity

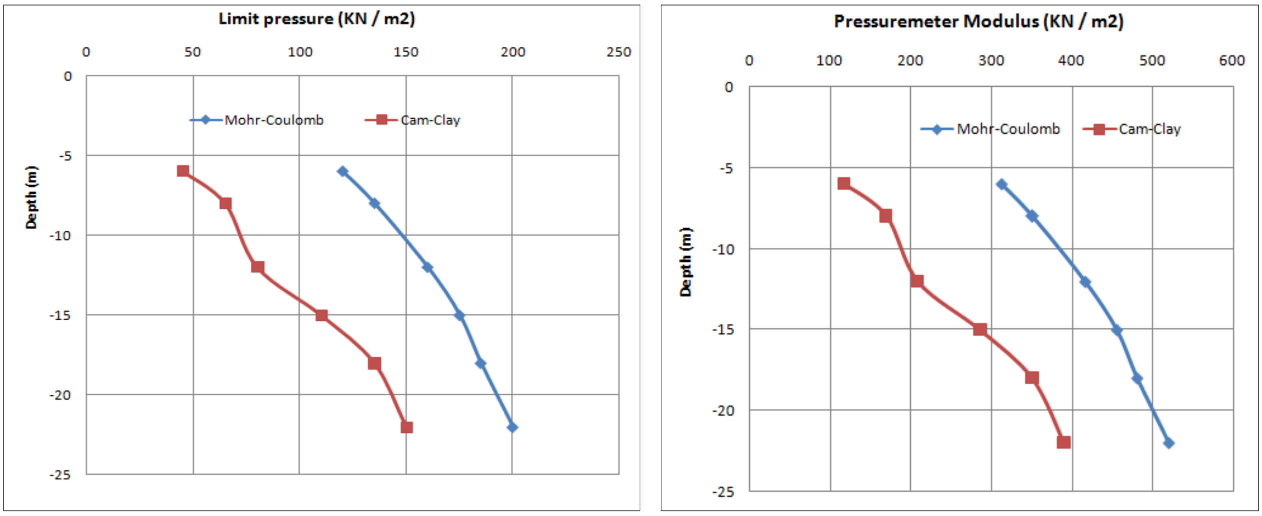
For the pressuremeter test, Figure 4c shows for the Mohr-Coulomb and Cam-Clay models, the results of horizontal displacements for different depths. it is found that the horizontal displacement has a proportional variation with the depth thus with the variations of the model of behavior; Figure 6 represents the set of pressuremeter curves which will be given in the form $\Delta V/V0 = f\ (p\text{-}p_0)$, with $(\Delta V\ /\ V_0)$ represents the relative deformation in the direction of application of horizontal loads from where V_0 is the initial volume of the probe, V the current volume, p_0 is the initial horizontal radial stress at the probe, and $(p\text{-}p_0)$ represents the pressure applied in the probe at the loading increment considered. These results are consistent with the conventional curves of the pressuremeter test [2, 3, 5, 6, 16], better to put classical references curves have a marked curvature. For high depths, they become quasi-linear.



a-Mohr-Coulomb **b-Cam-Clay**

Figure 6. Pressurometric curves

The profile of the limiting pressures and the associated pressuremeter module are shown in Figure 7 for both Mohr-Coulomb and Cam-Clay models). For all the results. It can be seen that the profile of the limit pressure depends only on the depth (Y) at which it is made. It is also noted that the limiting pressure has a proportional variation with the depth as well as the limit pressure for Mohr-Coulomb behavior model has a greater value than for Cam-Clay model. Same thing for the pressuremeter module [10, 16]. From these pressurometric characteristics, the bearing capacity can be estimated from the results of the in situ tests. Equation 4 is used.



a-Pressuremeter profile (P_{LIM}) **b- Pressuremeter module (E_M)**

Figure 7. Pressure Characteristics

So from the results we can now estimate the settlement using the Equation 1 the set of results are summarized in the table 5, and we can see that the settlement estimate for the Mohr-Coulomb model is greater than the settlement. with the Cam-Clay model and the settlement estimate by the direct method is less than the settlement found by the pressuremeter method.

Table 5. Recapitulated Settlement Estimate by Different

	Bearing capacity (KN/m^2)		Settlement	
	Direct method	**Pressuremeter method**	**Direct method**	**Pressuremeter method**
Mohr-Coloumb	300	320	3.5cm	4cm
Cam-Clay	201	222	1.8cm	2.1cm

5. Conclusions

This article is devoted to two main themes, the first of which is a theoretical synthesis on:

- Methods estimating settlement under a foundation;
- Estimated bearing capacity;

- Menard pressuremeter test.

And the second axis a numerical modeling. To estimate the settlement According to these results, we can see:

- We note that the horizontal displacement has a proportional variation with the depth and with the variation of the model of behavior the results are consistent with the conventional curves of the pressuremeter test from a qualitative point of view (curve shape). The curves have a marked curvature. For high depths, they become quasi-linear;
- As well as the limiting pressure and the pressuremeter module for the Mohr-Coulomb behavior model has a larger value than for Cam-Clay model; And a proportional variation with the depth;
- The bearing capacity has a Mohr-Coulomb behavior model variation has a greater value than for Cam-Clay model;
- The settlement estimate for the Mohr-Coulomb model is greater than the settlement found with the Cam-Clay model;
- The settlement estimated by the direct method is less than the settlement found by the pressuremeter method.

6. Acknowledgement

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7. Conflicts of Interest

The authors declare no conflict of interest.

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