Computed Bearing Capacity of Piles in Residual Soil Compared with Laboratory and Load Tests

Estimation de la force portante de pieux enfoncés dans un sol résiduel comparée avec des essais en laboratoire et des essais de charge

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Summary

A comparison is made between computed bearing capacity of piles driven or cast in a peculiar type of residual soil occurring in Southern Brazil with bearing capacities observed in load-tests. The theory adopted was that of *Terzaghi* developed by *Meyerhof*.

An interpretation of the behaviour of the soil around and below a pile, as far as shear resistance is concerned, was made in order to adopt proper angles of shear resistance and cohesion coefficients for the computations.

A fairly good agreement between computed and observed values was found.

Sommaire

Les auteurs ont fait une comparaison entre la capacité théorique de charge de pieux enfoncés ou moulés sur place dans un terrain résiduel typique du Brésil méridional et les capacités vérifiées lors d'essais de charge. La théorie adoptée a été celle de *Terzaghi*, développée par *Meyerhof*.

Une interprétation du comportement du sol autour du pieu et sous celui-ci, en ce qui concerne la résistance au cisaillement, a été faite en vue de l'adoption d'angles de résistance au cisaillement et de coefficients de cohésion dans les calculs d'après les essais de laboratoire.

Les auteurs ont trouvé une bonne concordance entre les valeurs estimées et les valeurs obtenues au cours des essais.

Purpose of the Investigation

Theoretical analysis of bearing capacity of piles is far from being properly solved. Difficulties of theoretical as well as of experimental nature have arisen to prevent that accomplishment. From the theoretical point of view, we do not have enough knowledge of the influence of soil compressibility, around and below the pile, in its skin friction and point resistance. Compressibility prevents, partially or totally, the mobilization of shear resistance of soil affecting the bearing capacity of the pile. There is also a considerable lack of knowledge about the influence of the method of construction of the piles upon its resistance. The experimental difficulties are mainly due to the fact that relatively few data are available on the bearing capacity of piles, connected with proper knowledge of the soil characteristics, which might allow a correlation between observed and computed bearing capacity of piles.

Besides, only recently a sufficient knowledge of shear resistance of soil was attained. Therefore, an interpretation of results of load tests on piles in connection with shear resistance tests carried on undisturbed samples, was made possible only recently.

The purpose of this paper is to contribute to the investigation of the bearing capacity of piles, cast or driven, through a peculiar type of soil which occurs commonly in Southern Brazil. Such soil is formed by a superficial layer of porous clay or porous clayey sand (a non-saturated clay or clayey sand, with a high void-ratio) originated from decomposition of rock or evolution of an ancient clay layer, underlain by a hard clayey substratum. This clayey substratum may be a decomposed rock, with a clay character, or a clay layer hardened by a sort of precipitation of the lixiviated elements of the upper layer.

The intention of the authors was to compare the results of load tests on piles driven or cast in that type of soil with theoretical computation made according to *Terzaghi* (1943) and

Meyerhof's (1951) analyses, using coefficients of soil resistance obtained by a careful analysis of the shearing resistance of such soil.

Description of the Nature and Properties of the Soil

The peculiar type of soil—as described above—is a residual soil formed in the climatic conditions prevailing in Southern Brazil. Its nature and properties were studied with some detail by *M. Vargas* (1951).

In the interior of the region the soil is formed by decomposition of basalt or sandstone. In both cases there is a superficial porous layer of clayey or sandy-clayey character underlain by a base substratum of decomposed basalt or sandstone. Near the coast, residual soil is formed by decomposition of gneiss or granite or evolution of ancient beds of Tertiary clays.

The evolution of a Tertiary clay layer in the city of São Paulo results in a soil of characteristics similar to the residual soil originated from decomposition of sound rock, although their origins are quite different. The evolution of the superficial Tertiary beds of clay gives origin to a porous superficial clay layer. Precipitation of some elements from the superficial layer into a lower layer results in the hardening of the latter.

As far as the soil properties are concerned, it was concluded that the shearing resistance of a "porous layer", below a certain value of the minor principal stress is almost the same, regardless of the type of test. That fact is shown by Fig. 1*a* where, in the shear resistance diagram, we can observe that, left of line *X–X*, shear resistance envelopes at several depths can be substituted by one straight line *A–B*, whose equation is

$$\tau = c + \sigma \operatorname{tg} \varphi \qquad (1)$$

Values of c (apparent cohesion) and φ (angle of shearing resistance) are not intrinsic characteristics of the residual clay but empirical parameters without meaning as physical constants. For values of the principal stresses, right of line *X–X*, the shearing resistance of the porous clay cannot be presented by equation 1 and is different for each type of loading (slow, quick or consolidated-quick).

As to the shearing resistance of the hard layer, saturated or almost saturated, it is well known that the point resistance of

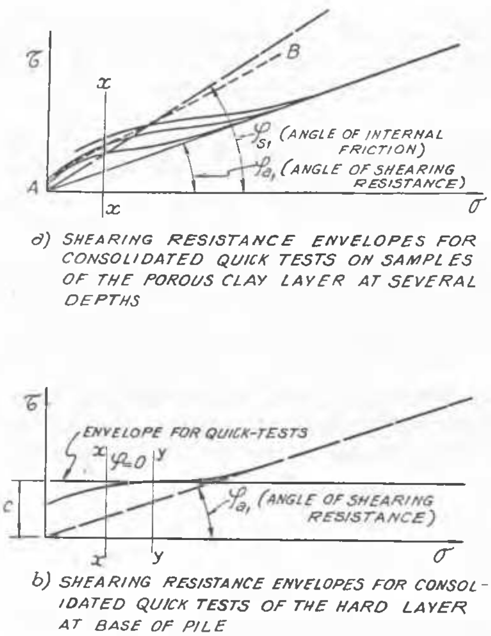


Fig. 1 Shear Resistance Characteristics of Residual Clays
Caractéristique de résistance au cisaillement des argiles résiduelles

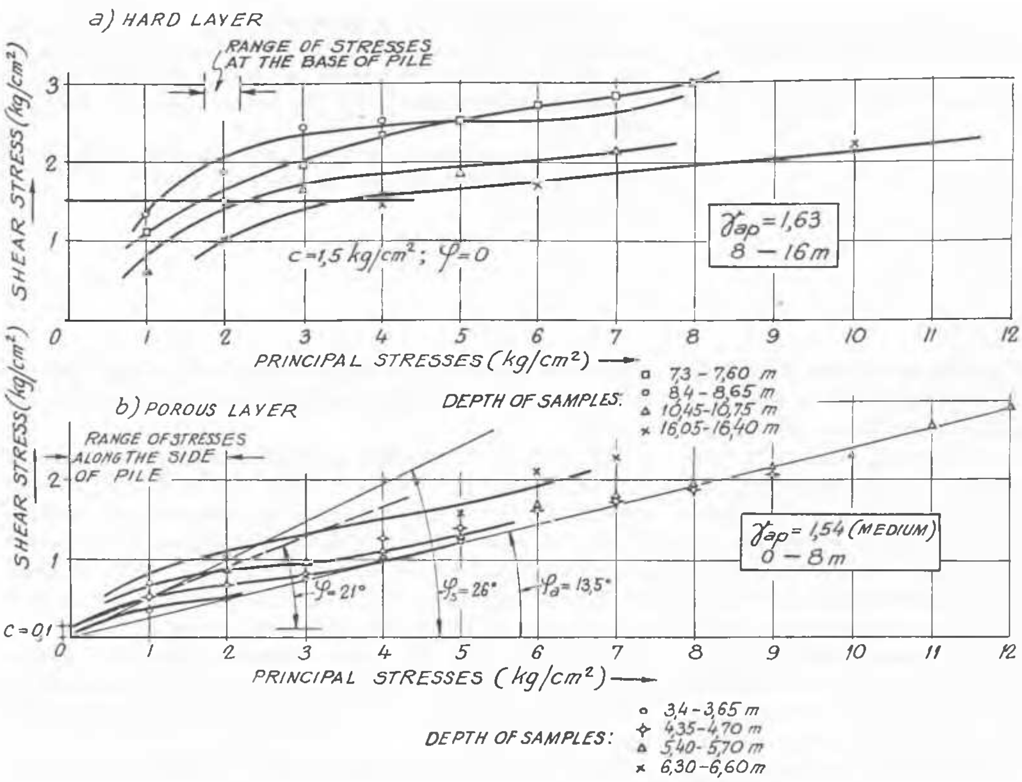


Fig. 2 Shear Resistance Characteristics of Soil at Site of Mentioned Franki Pile
Caractéristiques de résistance au cisaillement du sol au voisinage du pieu Franki mentionné dans le texte

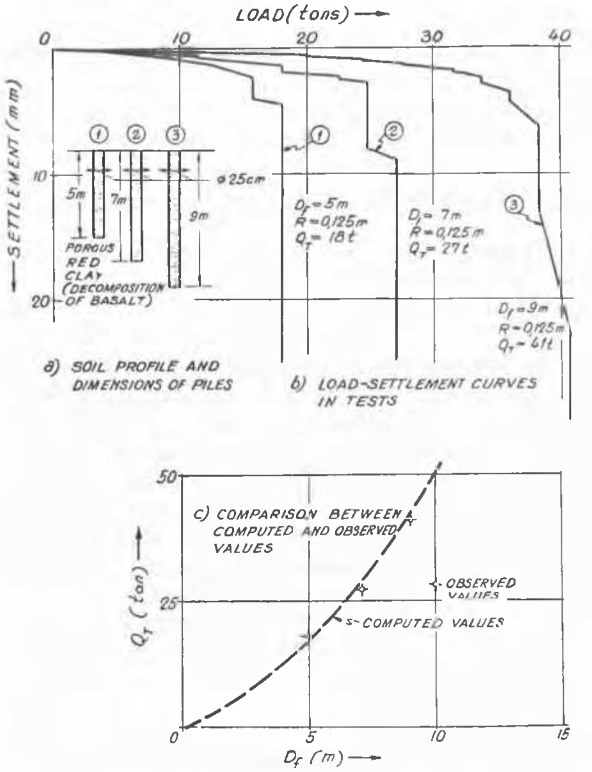
a pile resting on that layer can be computed on the basis of its cohesion observed in quick compression tests (see Fig. 1 *b*). In other words the loading is likely to be of the quick-consolidated type but, as for hard saturated clays, in the range of stresses prevailing at the point of the piles (between lines *X–X* and *Y–Y* in Fig. 1 *b*) the quick-consolidated and the quick envelopes are very similar. The error of taking the quick instead of the quick-consolidated envelope is small, and on the safe side.

Fig. 2 shows results of quick-consolidated shear tests made on undisturbed samples taken from the porous layer and from the hard layer at the site of a large Franki pile tested up to a 320 t load. This test confirms what was prompted above.

Table 1 presents results of average Atterberg limits and shear characteristics of soil from the places where load tests were made. The average shear characteristics were obtained according to what has been stated above.

Load Test Assembly and its Results

Load tests were made by means of hydraulic jacks reacting against counteracting loads formed by a balasted frame. Settlements were observed just after application of load and in successive intervals of time until stabilization was reached. No



← Fig. 3 Load Tests in Group of Piles *A* (Londrina)
Epreuves de charge du groupe de pieux *A* (Londrina)

Table 1 Average Characteristics of Soil from Sites of Load Tests

Group of Tests	Site	*Porous Layer* LL (%)	PI (%)	W (%)	γ (t/m³)	C (kg/cm²)	φ	*Hard Layer* LL (%)	PI (%)	W (%)	γ (t/m³)	C (kg/cm²)
A	Londrina	65	20	30	1,1	0,1	30	60	15	33	1,3	1,3
B	Garça	—	NP	30	1,2	0	30	—	—	—	—	—
	Araçatuba	—	NP	30	1,6	0,1	30	—	—	—	1,9	0,5–1,5
C	Congonhas	60	10	45	1,4	0,2	25	60	10	40	1,8	1
D	S. Paulo	70	15	42	1,5	0,1	25	65	10	40	1,6	0,5–1,0
—	Av. Paulista	75	25	40	1,5	0,1	21	70	12	38	1,6	1,5

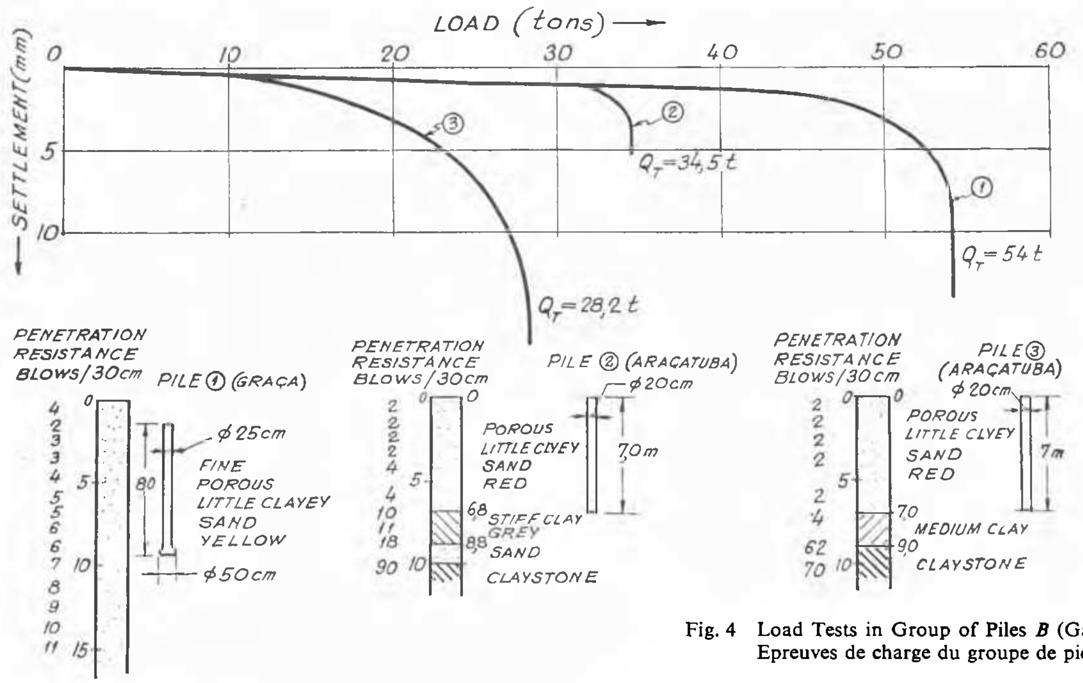


Fig. 4 Load Tests in Group of Piles *B* (Garça and Araçatuba)
Epreuves de charge du groupe de pieux *B* (Garça et Araçatuba)

load increment was applied to the piles before the settlements due to the preceding load had ceased.

The load tests were divided in groups. The first group, A, whose load test results are shown in Fig. 3, is composed of three concrete piles cast-in-place in a soil from decomposition of basalt, in the city of Londrina (State of Paraná). Group B (see Fig. 4) are three concrete piles cast-in-place in a soil from decomposition of sandstone, in the neighbour towns of Garça and Araçatuba (State of São Paulo). Group C (Fig. 5) are four concrete cast-in-place piles, in the city of São Paulo, through a porous clay originated from the evolution of an ancient Tertiary clay bed. Fig. 6 shows results of load tests on group of piles *D* which are two reinforced concrete pre-cast piles driven through the same porous clay of São Paulo. Finally Fig. 7 shows results of load test on a large Franki pile cast in this last type of soil.

In each one of these graphs there are sketches of the dimensions and position of the tested piles in relation to soil profiles.

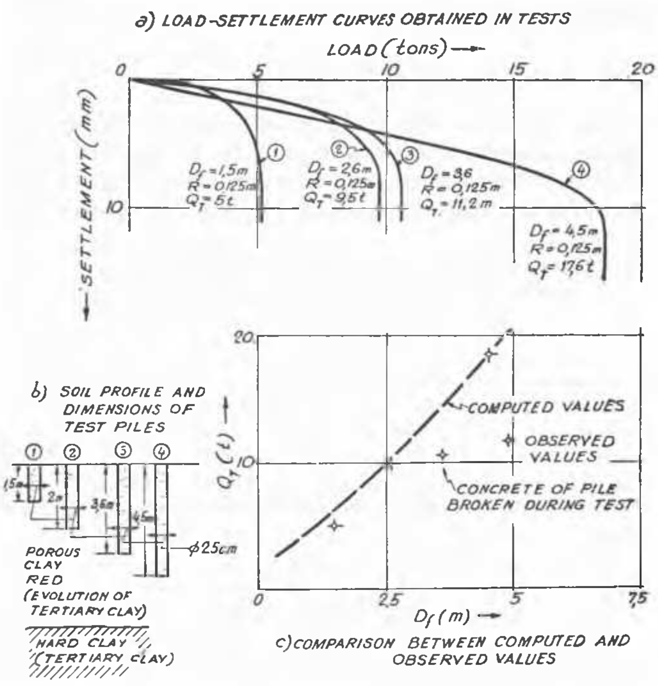


Fig. 5 Load Tests on Group of Piles *C* (Congonhas)
Epreuves de charge du groupe de pieux *C* (Congonhas)

Theoretical Consideration on Bearing Capacity Formulae

Point resistance of a pile according to *Terzaghi* (1943) and *Meyerhof* (1951) criterion is:

$$Q_B = \pi R^2 \,(1{,}3\, c N_c + \gamma D_f N_q + 0{,}6\, \gamma R N\gamma) \qquad (1)$$

where symbols R, C, γ and D_f are: radius of the pile point or pedestal, supposed circular, cohesion of the soil, specific gravity of the same soil and depth of the point or pedestal. N_c, N_q and N_γ are load-factors which, according to *Terzaghi*, are functions of the angle of shearing resistance of the soil, roughness of foundation and type of rupture (and, according to *Meyerhof*, are also functions of the foundation shape, its depth and of the compressibility of the soil. Actually those coefficients are difficult to determine in *Meyerhof*'s analysis, unless experimentally and in similar conditions.

Equation (1) can be expressed in the following form:

$$Q_B = B_1 + B_2 D_f \qquad (2)$$

where

$$B_1 = \pi R^2 \,(1{,}3\, c N_c + 0{,}6\, \gamma N_\gamma) \qquad (3)$$

$$B_2 = \pi R^2 \gamma N_q \qquad (4)$$

To determine the point resistance of piles resting on hard substratum we have to apply formula (1) with the following considerations: there is no possibility of a quick drainage of the soil at the base of the piles during application of load; so the angle of shearing resistance may be taken as zero, as stated above and the load-factors will be

$N_c = 5{,}7 \quad N_q = 1 \quad N_\gamma = 0$

Lateral resistance, according to *Meyerhof* will be expressed by:

$$Q_A = p \left(K_S \gamma \frac{D_f^2}{2 \cos \delta} + c D_f \right) \qquad (5)$$

where p is the perimeter of the pile, K_S the coefficient of earth pressure at rest (for the type of soil we are dealing with it will be approximately equal to 0,5) and δ is the angle of friction between pile and soil (which can be taken equal to φ).

(5) can be expressed by:

$$Q_A = A_2 D_f + A_3 D_f^2 \qquad (6)$$

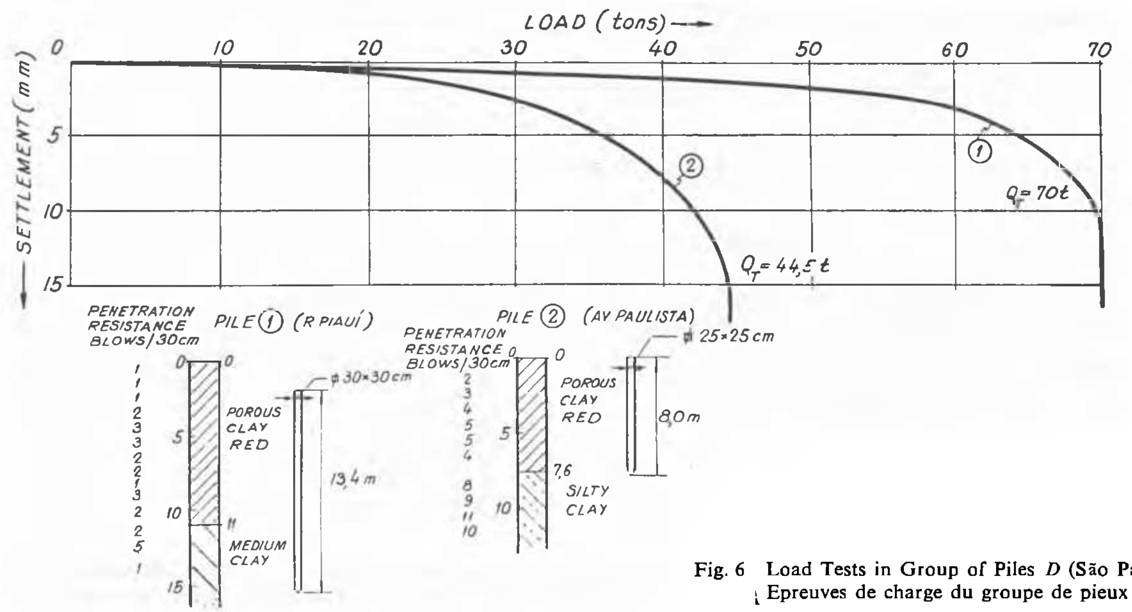


Fig. 6 Load Tests in Group of Piles *D* (São Paulo)
Epreuves de charge du groupe de pieux *D* (São Paulo)

Table 2 Results of Theoretical Computation and Observed Values in Load Tests

Group of Test	*Site*	*Computed Values*			*Observed Value in Load Test (t)*	*Observations*
		Point Resistance (t)	*Skin Friction (t)*	*Total Bearing Capacity (t)*		
A	Londrina	—	—	14	18	Total resistance computed by formula (9)
		—	—	28,5	27	
		—	—	42	41	
B	Garça	13	40	53	54	
	Araçatuba	23,7	5,5	29,2	34,5	
		23,7	—	26,0	28,2	
C	Congonhas	—	—	6	5	Total resistance computed by formula (9)
		—	—	10	9,5	
		—	—	14,4	11,2	
		—	—	18,4	17,6	
D	São Paulo	5,4	63,0	68,4	70,0	
		5,3	34,3	39,6	44,5	
—	Av. Paulista	203	245	448	> 310	

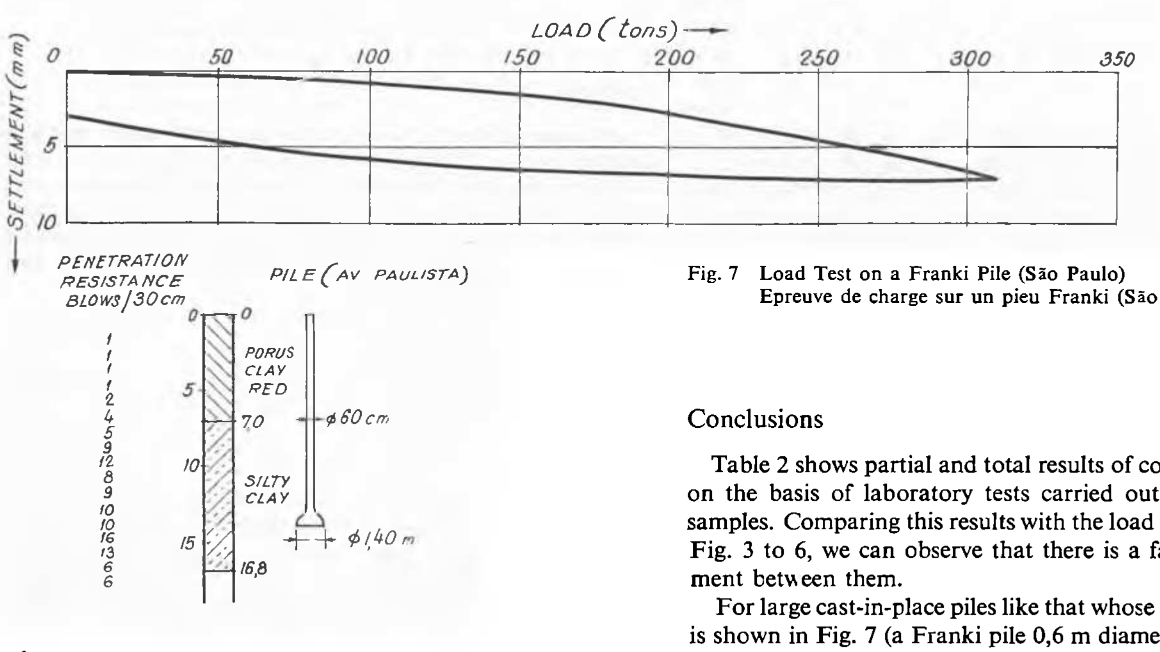


Fig. 7 Load Test on a Franki Pile (São Paulo)
Epreuve de charge sur un pieu Franki (São Paulo)

where

$$A_2 = pc \qquad A_3 = \frac{pK_S\gamma}{2\cos\delta} \tag{7}$$

The depth D_f considered as a term of friction (A_3) in the case of a deep foundation may not be the total depth of the soil. Eventually it is necessary to substitute D_f by $a D_f$, where a is an empirical coefficient which takes into consideration a reduction of the effective weight of the overlying soil.

Equation (6) becomes, then:

$$A_3 = \frac{pK_S\gamma}{2\cos\delta}\,a^2 \tag{8}$$

The total resistance is:

$$Q_T = Q_A + R_B = T_1 + T_2 D_f + T_3 D_f^{\,2} \tag{9}$$

where

$$T_1 = B_1 \qquad T_2 = B_2 + A_2 \qquad T_3 = A_3$$

which is a parabola of 2nd degree which can be adjusted statistically if we have three or more load tests on piles of different lengths in the same soil.

Conclusions

Table 2 shows partial and total results of computation made on the basis of laboratory tests carried out on undisturbed samples. Comparing this results with the load tests diagrams of Fig. 3 to 6, we can observe that there is a fairly good agreement between them.

For large cast-in-place piles like that whose load test diagram is shown in Fig. 7 (a Franki pile 0,6 m diameter of pipe) theoretical computation leads to very high values of bearing capacity, as compared with commonly allowable loads.

Load tests on such piles have been made up to loads of 500 t (*Ahrens*, 1948) and rupture was not attained due to difficulties in assembling such large loads in the test arrangements. They have shown however that large bearing capacities could likely be attained.

Further investigations and tests are in progress for the clarification of some doubtful points.

References

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