INTERNATIONAL SOCIETY FOR
SOIL MECHANICS AND
GEOTECHNICAL ENGINEERING

This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:

https://www.issmge.org/publications/online-library

This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.

Static tests on friction piles in Mexico City clay

Essai statique sur des pieux flottants dans l'argile de la ville de Mexico

A.JAIME P., Research Professor, Instituto de Ingeniería, UNAM, Mexico
M.P.ROMO, Research Professor, Instituto de Ingeniería, UNAM, Mexico
J.A.PONCE, Solum, S.A. de C.V., Mexico
A.MITRE M., Fellow, Instituto de Ingeniería, UNAM, Mexico

SYNOPSIS: Results of penetration and pull out tests carried out on concrete piles driven in Mexico City clay show several significant characteristics of the friction pile behavior. First, the maximum penetration load capacity depends upon the rate of applied load, the higher the rate the higher the capacity. Second, the rate has also a similar influence on the mean slope of the load vs. displacement (P-δ) curve. Third, friction pull out capacity attains a maximum value equal to the friction penetration capacity; even though, the extraction load vs displacement curve is different from the corresponding one in penetration. Finally, the parameter α used on bearing capacity of friction piles is found to be 1.2 for Mexico City clays.

1 INTRODUCTION

As a consequence of the seismic events of September, 1985 in Mexico City several buildings founded on friction piles exhibited tilting and partial failure, and one structure completely collapsed. To better understand the behavior of friction piles driven in Mexico City clay a series of static and cyclic load tests were performed on concrete piles, driven for this purpose. The main objective of this paper is to present and analyze the static pile load test results.

It is recognized that the conclusions derived from the test results are, strictly speaking, valid only for Mexico City clay. However, these tests show some general trends applicable to other clay types. For instance, from laboratory results it is well known that the undrained strength of a clay is higher when the deviator stress is applied fast; likewise, the stiffness of the material is affected. These facts are qualitatively similar to the observed pile behavior: the higher the loading rate the higher the load capacity; similarly, the mean slope of the load vs. displacement curve (as it is seen later on). For these reasons, it is believed that the results and conclusions presented in this paper may be of interest to practitioners and researchers alike.

2 GENERAL INFORMATION

According to the subsoil type, Mexico City has been divided into three zones: Hilly zone, Transition and Lakes (Marsal and Mazari, 1959; Jaime, 1987). The Lakes Zone comprises Texcoco and Xochimilco-Chalco lakes. Texcoco Lake, in turn, has been subdivided into preloaded and virgin parts; the former corresponds to the ancient Texcoco lake bed on which Mexico City was first built (and severe water pumping of the aquifers has occurred); the latter, to the zone of recent urban development. In general, preloaded Texcoco Lake has soft clay deposits with a thickness of less than 50 m. The test site is localized in this part of the city at Centro Urbano Presidente Juárez, on Yucatán and A. M. Anza streets (fig 1).

This site was chosen because its general subsoil characteristics are representative of the most heavily damaged zone in the city during the September, 1985 seismic events, and because there was never a construction built on the site. It was used as a parking lot with a concrete pavement 10-cm thick.

2.1 Soil profile

To determine the stratigraphy of the site a continuous sampling boring and an electric cone penetration test were performed to depths of 15.5 m and 17 m, respectively.

From previous information it was known that the first 5 m of soil consisted of a very hard crust, the boring at the site was carried out from the surface down to 5.0 m depth drilling with a tricone bit 12.5 cm in diameter, and from 5 m depth on, Shelby tubes, 10 cm in diameter, were used to get good quality soil samples.

The site stratigraphy is shown in fig 2. There are three clay layers between 5 and 15.5 m depth with mean water contents, from the upper stratum to the lower one, of 375, 300 and 325 %, respectively. These soil strata can also be seen in the cone resistance profile, q_p. Between 12 and 13 m depth there is a stiffer stratum of sandy clay. The presence of this layer and previous experience in similar sites led to the decision of drilling a 15-cm diameter and 10-m long hole (starting at 5 m depth), before driving the precast concrete piles into the soil.

The undrained strength s_u of the clayey soil was determined in the laboratory by means of unconsolidated-undrained triaxial tests (UU tests). The results vs. depth are depicted in fig 2. Between 5 and 14.75 m depth, 38 determinations were made. The mean value of the undrained strength is s_{uu} = 34 kPa with a standard deviation σ= 9 kPa (variation coefficient CV= 27 %).

To estimate the point bearing capacity, the

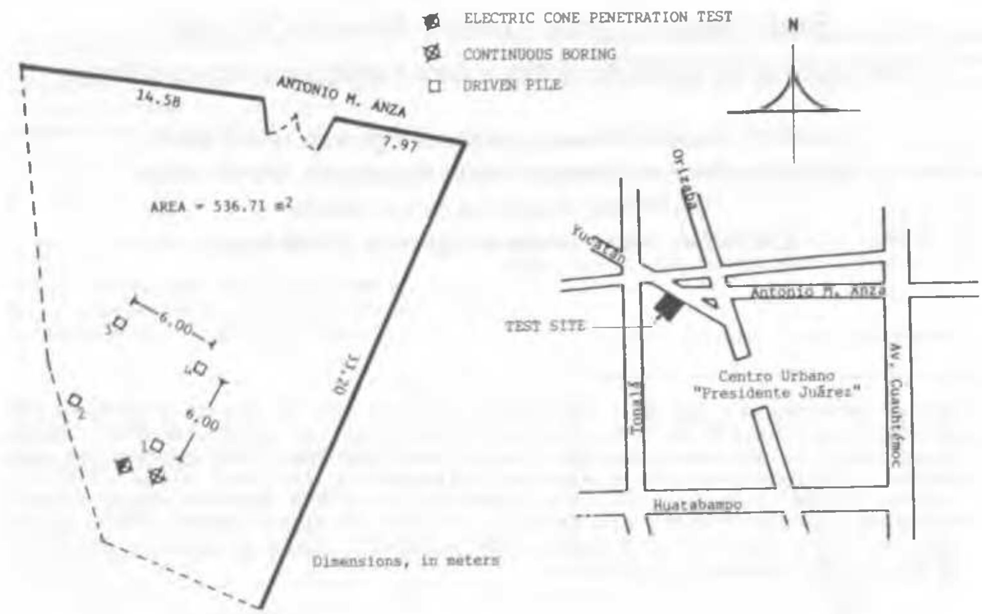

Fig 1. Site location, pile layout and borings

undrained strength of the soil from the tip of the pile elevation to an elevation three diameters deeper should be determined. Taking into account two UU test results and the cone resistance profile a mean value of s_u = 68 kPa was obtained.

2.2 Piles

Four precast concrete piles, with 30 x 30 cm square section and 15-m long, were driven at the corners of an imaginary square, 6 m on the side. This gave a spacing between piles of 20 times their lateral dimension. Laboratory and field experiences have shown that there is no pile to pile interaction if they are placed 12 pile diameters (or maximum lateral dimension) apart from each other, provided they do not exceed a length of 80 diameters (Tomlinson, 1977; Terzaghi and Peck, 1967; Sowers et al, 1961).

On the basis of the stratigraphic conditions at the site, it was decided, before driving the piles, to cross the superficial crust by drilling a hole 50 cm in diameter and 5 m long. The hole was encased with steel pipe of the same dimensions and 9 mm wall thickness. So the piles were left freestanding from the surface down to 5 m depth, and embedded in clay (fig 3) only from this elevation down to 15 m. As it was mentioned before, the piles were driven into a prebored hole 15 cm in diameter and 10 m long. On top of each element a one meter long pile section was added to facilitate the work during the tests, thus the final length of the pile was 16 m.

2.3 Loading system

The piles were subjected to penetration and pull out tests. Slow and quick penetration tests were carried out. The slow penetration test was performed applying load increments on the pile at two-hour periods; the piles were loaded by means of a hydraulic cylinder attached to a manual pump. The quick penetration test was made increasing monotonically the load on the pile. To do this the jack was connected to an electric pump; in this way, it was possible to reach a load of 600 to 700 kN in 30 to 40 seconds.

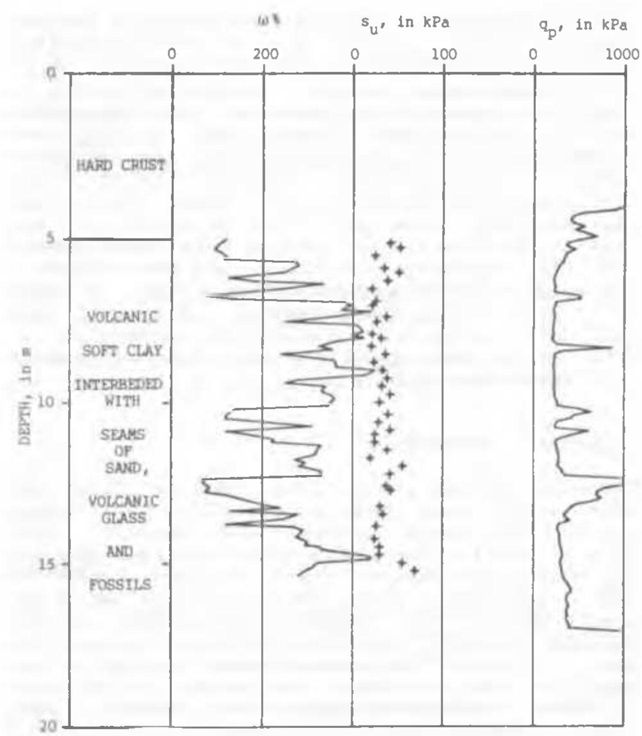

Fig 2. Soil profile

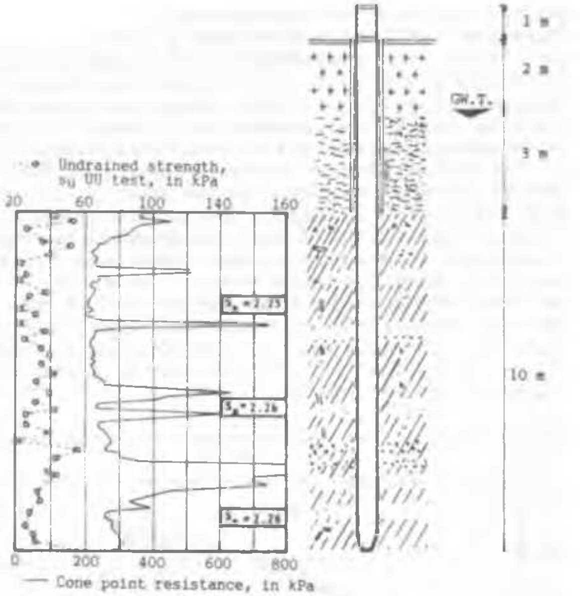

Fig 3. Cross-section of pile

A schematic view of the loading system for penetration tests is shown in fig 4. There is an articulation placed between the pile head and the hydraulic jack, an electric load cell placed on top of the jack and several square steel plates used for adjustment. The joint allowed small vertical deviations of the axial load, the pile top and the reaction frame. The applied load was measured with the load cell and two manometers (Bourdon type). One of the manometers was attached to the jack and the other to the pump. The displacements were measured with four dial gages (0.01 mm accuracy). The dial gage bodies were firmly supported by an independent bridge-like structure, and their stems reacted against glass plates located on the sides of the pile (fig 4).

The pull out test was performed in a manner similar to the slow penetration test, but instead of pushing down the pile it was forced out of the soil, fig 5.

3 PILE LOAD TESTS

3.1 Penetration tests

The driven piles were left undisturbed for five months before performing one slow and two quick penetration tests.

The slow penetration test (pile 2) was carried out increasing the load on the pile in increments of 50 kN applied at two-hour periods. Displacement (δ) and load (P) readings were taken at 1, 2, 4, 8, 15, 30, 60, 90 and 120 min. During the first five increments, no more appreciable displacements occurred after 30 min. The load was kept constant by manipulating the hydraulic pump, when needed.

The results of the slow penetration test are shown in fig 6. The squares correspond to instantaneous readings of displacement and the crosses to readings taken after two hours. It can be seen that there is an almost constant load-displacement relation up to a 350 kN load;

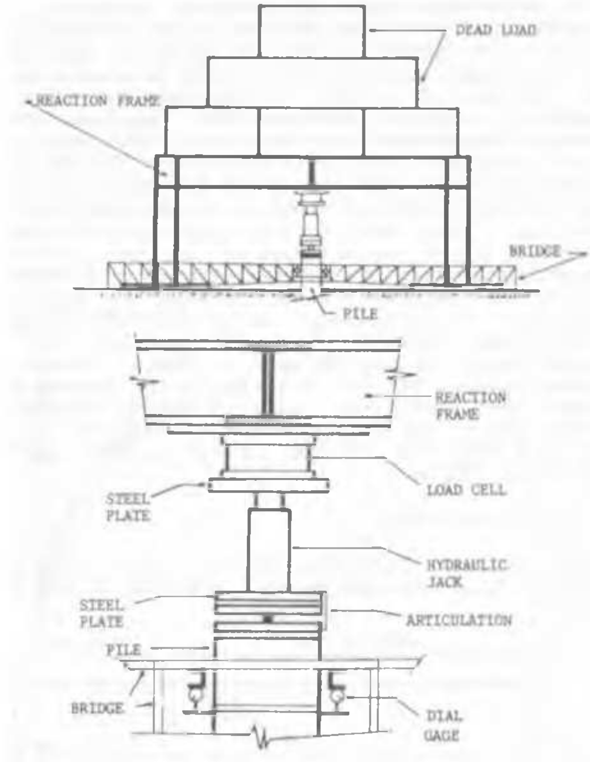

Fig 4. Loading system. Penetration test

beyond this point the relation becomes highly nonlinear. It is also appreciated that delayed displacements are very small at the beginning and become increasingly larger after the 350 kN load. A peak load of 540 kN is reached at a displacement of 22 mm. After reaching the maximum load the pile exhibited a very fast displacement of 11 mm (in 4 min) going down from the peak load to a residual one of 480 kN. The shape of the curve linking the maximum and the residual loads is unknown, because beyond the peak load the test changed from load controlled to displacement controlled test. The displacement depends on how fast the (hand-operated) pump can be actuated and on its flow capacity.

The pile was unloaded in six decrements, spaced at 10 minute periods. The pile went from a maximum displacement of 33 mm to 24 mm; that is to say, it had a 9 mm rebound and accumulated 24 mm. It may be observed a small delayed rebound in the unload branch of the P vs δ curve.

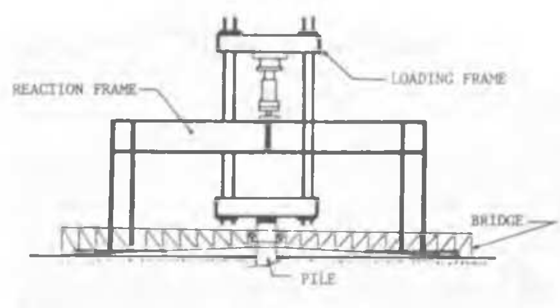

Fig 5. Pull-out loading system

Piles 1 and 3 were subjected to quick penetration tests. The load on the piles was monotonically increased. In the first quick test (pile 3) a maximum applied load of 640 kN was reached in 32 sec (fig 7). After that the test was abruptly stopped because the applied load almost surpassed the dead load. Then the reaction frame was moved to repeat the test on pile 1 and the dead load increased 100 kN in order to have enough reaction force to bring the pile to failure. This test was also interrupted after a load of 730 kN was applied in 55 sec. without reaching the failure of the pile as shown in fig 7. It was not possible to increase the dead load because of structural limitations of the reaction frame. However, from the results of the quick tests, it may be seen an almost linear P-δ relationship up to 400 kN load. Beyond this load some nonlinear effects are exhibited. Based upon the shape of both P-δ curves it can be said that the maximum load in quick test is at least 750 kN

3.2 Pull out test

The pull out test was carried out on pile 4. The tensile load increments were applied in the same way as in the slow penetration test. From 0 to 200 kN the load increments had a 50 kN magnitude, then two increments of 4 kN each were applied, and the last six increments had a 30 kN magnitude (fig 8). After reaching a peak load of 460 kN, at a displacement of 34 mm, a large deformation occurred with a residual load of 385 kN. As in the slow penetration test, beyond the maximum load the test changes from a load controlled type to a displacement controlled one. The pile was unloaded in five equal decrements. The maximum displacement was 42 mm, and at zero load the displacement had a value of 26 mm; thus, the recoverable deformation was 16 mm.

The P-δ curve, of fig 8, is nonlinear from the beginning. The delayed deformation is negligible up to a load level of 300 kN, and is increasingly larger beyond this point.

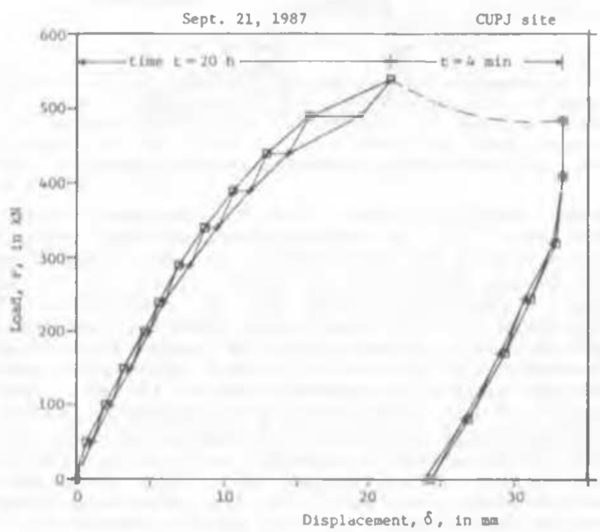

Fig 6. Slow penetration test

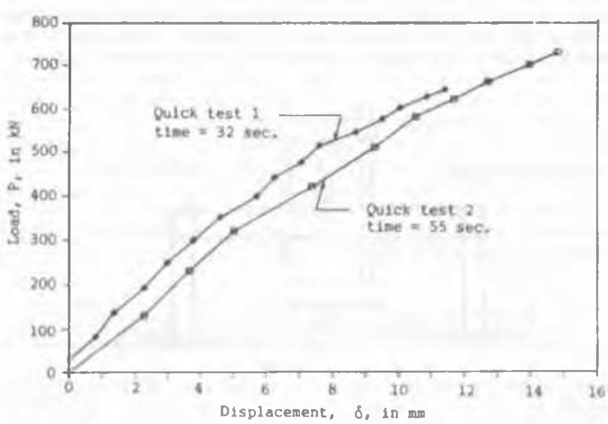

Fig 7. Quick penetration tests

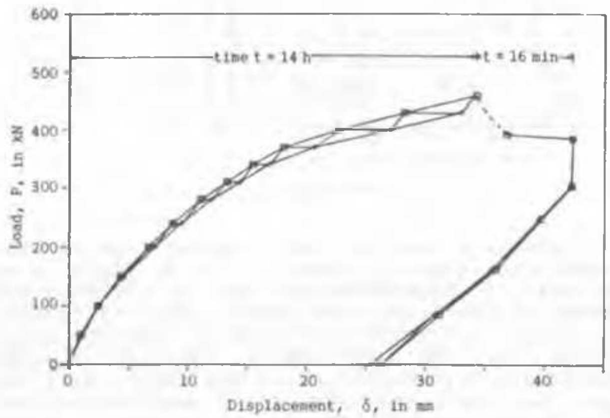

Fig 8. Pull-out test

4 INTERPRETATION OF RESULTS

For the sake of comparison, the P-δ curves obtained from the slow and quick tests are shown together in fig 9. It can be seen that the two quick tests (performed in two different piles) are quite similar. The average slope of these curves is 1.6 times larger than the corresponding mean slope of the slow test. It can be added

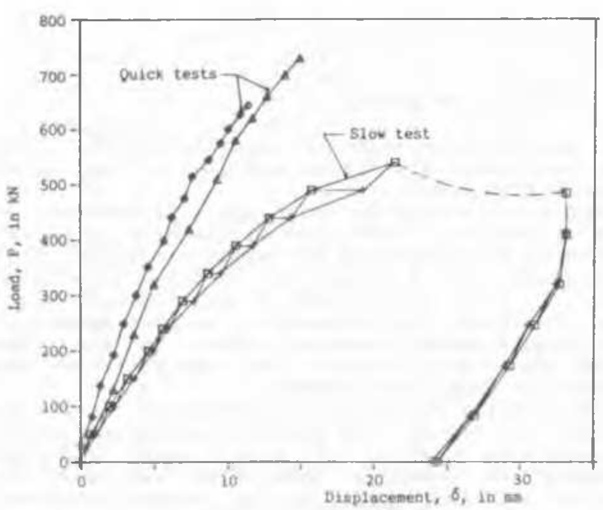

Fig 9. Slow and quick penetration tests

that the maximum load in quick test conditions is at least 1.5 times the peak load value obtained in slow penetration tests.

On the other hand, in fig 10 the P-δ curves of the slow penetration and the pull out tests are compared. It is appreciated that the maximum loads are attained at vertical displacements of 22 and 34 mm, respectively. Also, the slope of the pull out curve is always below the one corresponding to the slow test. Based on this comparison it appears that: 1) the friction pile behavior depends on the applied load direction; that is to say, if the element is subjected to a penetrating or extracting load; and 2) the response differences are caused by point resistance effects.

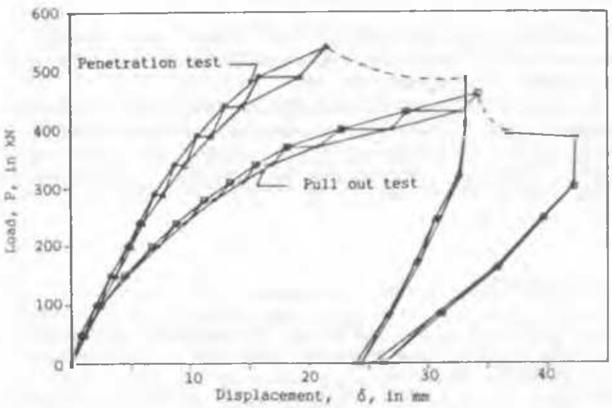

Fig 10. Slow penetration and pull-out tests

To further explore the pile behavior in penetration and pull out conditions, the P-δ curves of both tests were corrected by substracting the structural shortening and elongation of the pile, respectively. In tension, only the reinforcing bars were taken into account, neglecting the concrete contribution. The pile (as a whole) and the steel bars were assumed to be linearly elastic elements. The shear stress distribution along the pile sides was taken as constant, as well as the shear stress along the reinforcing bars. On this basis the deformations were determined with the equation:

$$\delta = \frac{PL}{2AE} \qquad (1)$$

in which:

- P compressive or tensile load
- L total length of the pile (16 m)
- A cross-sectional area. A = 900 cm^2, in compression; A = 20 cm^2, in tension.
- E elastic modulus of the element.

It can be seen, in fig 11, that below a load of 250 kN the corrected pull out curve is stiffer than the slow penetration one; beyond 250 kN the penetration curve becomes stiffer than the pull out one. The displacements at which the peak loads occurred are 18 and 25 mm for penetration and extraction, respectively.

From field and laboratory experiences (i. e., Tomlinson, 1977; Sowers et al, 1961; Vesic, 1975), it is known that friction resistance of piles in penetration tests is fully mobilized at a displacement smaller than the one needed to reach the maximum point load resistance. Therefore, the results in fig 11 should exhibit a stiffer pull out curve all the way to failure. Since this is not the case, this result seems to confirm that the friction pile behavior in tension is different from the response in penetration. The difference seems to be only in the shape of the P-δ curve and not in the maximum load as it is shown in the next paragraphs.

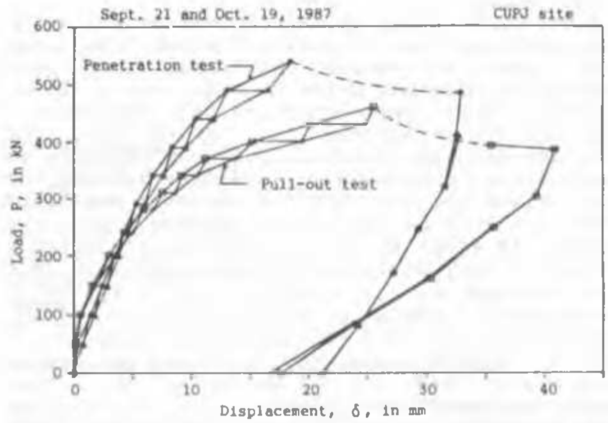

Fig 11. Corrected slow penetration and pull-out test curves

The effective weight of the pile has to be added to the maximum load measured in the slow penetration test and substracted from the corresponding pull out value. Doing this it is obtained P_p = 563 kN and P_{ue} = 437 kN, for penetration and extraction tests, respectively.

The ultimate friction pile capacity is calculated with the well known equation:

$$Q_f = \alpha(\Sigma_i\, c_i\, d_i)\; 4B \qquad (2)$$

in which:

- α Tomlinson's correction factor (for c_{uu} = 30 kPa α =1, Tomlinson, 1977)
- c_i undrained strength of ith layer of soil
- d_i thickness of ith layer
- B pile width

The ultimate point bearing capacity is calculated with the expression:

$$Q_p = A_p\,(cN_c + p_v) \qquad (3)$$

where:

- A_p area of pile tip
- c undrained strength of soil below pile tip
- p_v total overburden pressure at pile tip
- N_c bearing capacity parameter. For deep foundations it is accepted a value of 9

Using equations 2 and 3 the following values are obtained: Q_f = 401.4 kN and Q_p = 73.5 kN. If Q_f is compared with P_{ue} it is observed that the latter has a value 9 % higher than the former which is a reasonable approximation.

However, P_p is 19 % greater than Q_u = 475 kN (Q_u=Q_p+Q_f). This difference can be due to point resistance effects; i.e. the N_c parameter has a value higher than 9. De Mello (1969), citing other authors, points out that from laboratory and field measurements the value of N_c varies from 7 to 10; even though, some researchers report values as high as 20. Vesic (1975) suggests that the N_c range of values is from 7 to 12, depending upon a rigidity soil coefficient, which, in turn, is a function of the elastic modulus, Poisson's ratio and strength of soil. On these bases, it can be concluded that on the average, $N_c = 9$; furthermore, it is the best estimate in absence of a direct measurement. Thus, the difference between P_p and Q_u is caused by other factor.

When pile 4 was driven, almost at the end of the operation, the pile side touched the steel pipe (used to encase the upper 5 m of soil) pushing it down one meter. From this, it is assumed that the effective pile embedment in clay was 9 m instead of 10 m. Taking this fact into account, the friction pull out capacity was recalculated obtaining Q_f = 362 kN; thus P_{ue} is 1.21 times higher than this value. Based upon this result it can be concluded that the α parameter is equal to 1.2, instead of the value of 1.0 previously considered. Recalculating Q_f with L = 10 m and α = 1.2, a value of Q_f = 482 kN is determined. Adding Q_p to Q_f gives a total load capacity of Q_u = 555 kN, which is only 1.4 % smaller than P_p (measured peak load in penetration test); thus, α = 1.2 seems to be a good value for Mexico City clays.

When friction piles driven in Mexico City clay are withdrawn, a hard clay crust is observed to be surrounding the pile. So the failure surface is developed between the clay crust and the surrounding soil, and not along the pile-clay interface. The crust has a mean width of 5 to 10 % of the pile radio (or half the pile width). To take into account this field evidence, Zeevaert (1975) suggests to increase the nominal pile diameter (or width) by a factor of 1.1. In the previous analysis this effect was not explicitly taken into account. It was considered to be involved in the α parameter, as well as other factors such as driving effects, clay reconsolidation, etc.

5 CONCLUSIONS

From the results of quick and slow penetration tests, and pull out test performed on piles driven in Mexico City clay, the following conclusions can be drawn:

1) The maximum load capacity, in penetration test, of a pile fully embedded in Mexico City clay depends upon the applied load rate; the higher the rate, the higher the capacity. (This agrees with test results reported by Marsal and Mazari, 1959). The peak value of load capacity is at least 1.5 times the one obtained in a slow penetration test.

2) The average slope of the P-δ curve increases with the rate of applied load. The slope increment measured in the tests reported herein is about 1.6 times, when comparing the test performed in a few seconds and other lasting several hours.

3) The P-δ curves obtained in slow penetration and pull out tests exhibit a peak load and a residual one. This residual load reaches a value 15 % lower than the maximum load. Marsal and Mazari (1959) reported, from tests in wood piles, residual loads as low as 50 % of the peak value.

4) The clay was considered to behave as cohesive material ($c \neq 0$, $\phi = 0^o$); on the basis of this assumption and using eqs. 2 and 3, it was shown that the ultimate friction and point bearing capacities can be calculated with reasonable accuracy, using a total stress approach.

5) From back calculations the α parameter for Mexico City clay was found to be equal to 1.2.

6) The peak friction capacities determined in penetration and pull out tests are similar, even though the P-δ curves are different.

6 ACKNOWLEDGEMENTS

This project was sponsored by Departamento del Distrito Federal and Consejo Nacional de Ciencia y Tecnología. The authors gratefully acknowledge this support which made the investigation possible.

Grateful appreciation is also expressed to Grupo ICA, and its branch Solum, S.A. de C.V. for their cooperation.

R.J. Marsal and D. Reséndiz made very valuable suggestions; R. López Roldán, Vice-President, Grupo ICA, enthusiastically supported the project from the beginning to the end. Thanks are also given to A. Bello M., General Manager, Solum, S.A. de C.V.

7 REFERENCES

De Mello, V.F.B. (1969), "Foundations of buildings in clay". State of the art volume, 7th ICOSOMEF, Mexico.

Marsal, R.J. and Mazari, M. (1959), "El subsuelo de la ciudad de México". Edited by UNAM. Mexico.

Reséndiz, D. (1964), "Estudio de campo sobre pilotes de concreto reforzado". Revista INGENIERIA, vol. XXXIV, January, Mexico.

Sowers, G., Martín, C.B., Wilson, L. and Fausold, M. (1961), "The bearing capacity of friction pile groups in homogeneous clay from model methods". Proceedings 5th ICOSOMEF, vol. 2, pp 155-159, París.

Terzaghi, K. and Peck, R. (1967), "Soil Mechanic in Engineering Practice". John Wiley and Sons, 2nd ed., New York.

Tomlinson, M.J. (1977), "Pile design and construction practice". A Viewpoint Publication. Great Britain.

Vesíc, A.S. (1975), "Principles of pile foundation design". School of Eng., Duke University. Soil Mechanics Series No 38, Durham, North Carolina.

Zeevaert, L. (1973), "Foundation engineering for difficult subsoil conditions". Van Nostrand Reinhold, Co., New York.