Compensated Friction-pile Foundation to Reduce the Settlement of Buildings on the Highly Compressible Volcanic Clay of Mexico City

Fondations Compensées avec Pieux à Frottement pour Diminuer le Tassement des Bâtiments Construits sur l'Argile Très Compressible de Mexico City

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Summary

The use of friction piles in compensated foundations appears to open a new horizon for design in places like Mexico City where very highly compressible clay deposits are encountered.

The author describes the design and performance of such a foundation at a site where the clay deposits assume a very high compressibility compared with other places in the city.

Settlement observations and subsoil investigations revealed the behaviour of the friction piles used in the building. The settlement was much smaller compared with settlement estimates for the same building but without piles. A great economy was achieved because the friction piles carried only a fraction of the total load of the building. Furthermore, since the piles were designed without support from deep strata, the undesirable feature of the sidewalk settling away from the building, on account of the well known ground surface subsidence of Mexico City, was eliminated.

Sommaire

L'emploi des pieux à frottement dans les fondations compensées ouvre un nouvel horizon dans le dessin des fondations pour des régions comme la Cité de Mexico, où l'on trouve une argile très compressible. L'auteur rend compte des résultats obtenus avec une fondation compensée, avec des pieux à frottement, en un point de la Cité de Mexico, où les depôts d'argile assurent au sol une plus grande compressibilité qu'en d'autres emplacements dans la Cité. L'observation des tassements et les recherches sur le sous-sol ont permis de connaître le comportement des pieux à frottement sous l'édifice et ont montré que les tassements obtenus étaient considérablement réduits comparés aux estimations pour l'édifice sans pieux à frottement.

Une grande économie fût obtenue grâce à ces pieux qui ont supporté seulement une fraction de la charge de l'édifice.

De plus, les pieux n'étant pas appuyés sur les couches les plus profondes, on élimine le problème difficile du tassement relatif entre l'édifice el la surface du sous-sol dû à l'affaissement superficiel bien connu dans la Cité de Mexico.

The building 'La Azteca' with a total weight of 1·2 kg/cm^2 was constructed in a part of Mexico City where the clay deposits are very highly compressible.

Subsoil investigations showed that at this particular site the first hard stratum at a depth of 32 m, used in the heart of the city to support point-bearing piles, was absent. It was found that the only possible level to support piles was on a firm sand stratum encountered at 42 m depth. However, this idea was abandoned because of the high cost of the piles, which had to be designed not only to carry the total weight of the building but also the large negative friction induced by the ground surface subsidence.

The piles to reach 42 m depth had to penetrate all the highly compressible clay deposits which are strongly affected by the reduction in the piezometric levels. The ground surface subsidence caused by the compression of these deposits is very large; therefore difficult problems could be created in the public utilities entering the building (ZEEVAERT, 1952a).

A totally compensated foundation, on the other hand, was also undesirable because costly special precautions would need to be taken to make a deep excavation in the soft clay. A large heave would be obtained (ZEEVAERT, 1955) and the re-compression would be very large because of the very high compressibility of the clay.

Finally, the author decided to recommend a compensated friction pile foundation which, according to soil mechanics computations, gave an acceptable settlement. The weight of the building was balanced partly by an excavation 6 m deep, and partly by 83 concrete friction piles 40 cm diameter driven a depth of 24 m.

The total weight of the building is 7950 ton. The compensation load available at 6 m depth is 5790 ton, and therefore the piles were assumed to take a load of 2160 ton or the equivalent of 26·0 ton per pile. The foundation area and the pile layout are shown in Fig. 7.

Computed and observed settlements are compared in Fig. 9. The observed settlement resulted in about one-half of the estimated settlement that would have taken place if the same foundation had been designed without friction piles. The computed settlement for the compensated friction pile foundation is of the same order of magnitude and trend as the observed settlement.

This paper discusses also the probable behaviour of the friction piles under the foundation structure in order to justify the working hypothesis that has been used in the design.

Subsoil Characteristics

The site is located in the lacustrine area occupied by the western central part of Mexico City. This location is known particularly by the very high compressibility of its subsoil, since many buildings up to six storeys high on fully-compensated foundations have suffered large settlements. The subsoil profile is shown in Fig. 1. The typical lacustrine series of deposits of volcanic clay with a high water content start at 6 m and extend to 40·5 m depth, and are underlain by a series of alluvial deposits of fine andesitic gravel, sand and clayey silt that follow to greater depth, Tarango Sand II (ZEEVAERT, 1953 a, b, c).

The lacustrine volcanic clay deposits are interbedded with thin fine sand and silt stratifications. Specially significant for settlement analysis (ZEEVAERT, 1951) are those marked clearly in the soil profile (Fig. 1). The first thick soft clay deposit, Tacubaya I to V, exhibits a very high water content that in

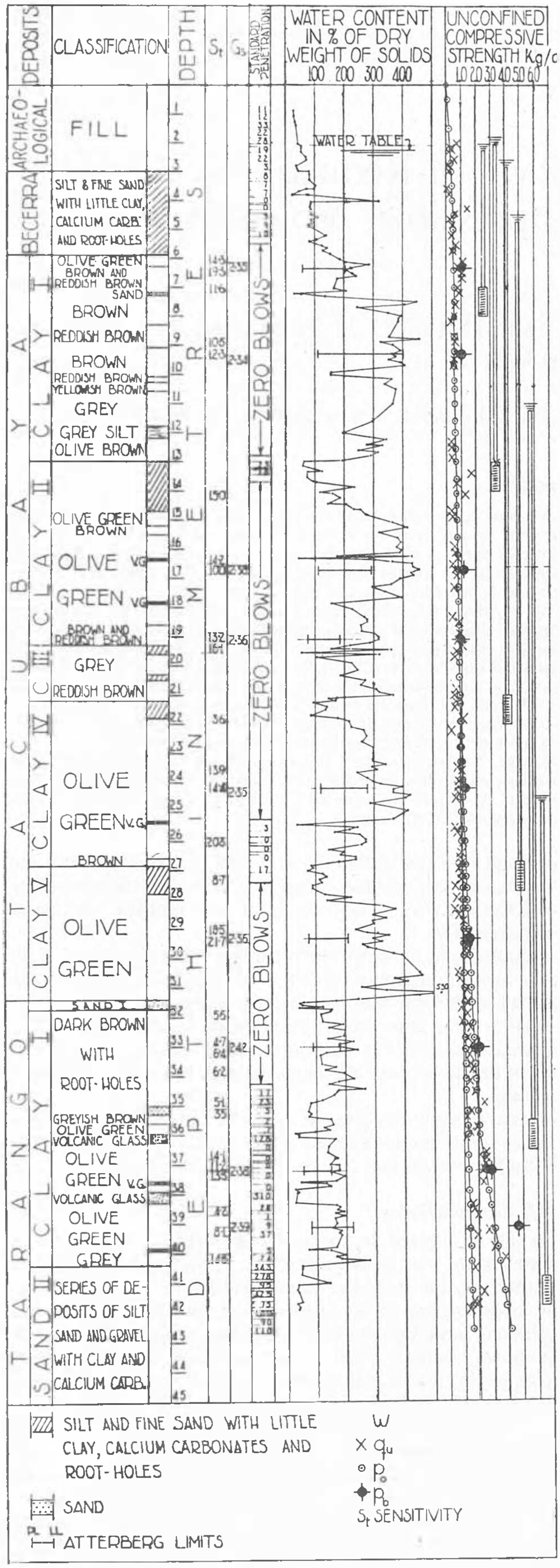


Fig. 1 Subsoil profile—Caractéristiques du sous-sol

some places is well above 400 per cent. The second semi-rigid clay deposit, Tarango Clay I, shows an average water content of about 175 per cent and is correspondingly less compressible.

It should be noticed that at the contact horizon at 31·5 m depth, between first and second clay deposits, the typical Tarango Sand I alluvial deposit known as the first hard formation is absent. Most of the point-bearing piles for buildings in the heart of the city rest on this formation.

To find out the source of ground surface subsidence in this area piezometers were installed in the pervious strata at depths 42, 37, 28, 22, 14 and 8 m respectively. It was found (Fig. 1) that there is practically no drop in piezometric level from the ground surface to a depth of 28 m, where a water-bearing stratum of silt and fine sand of microscopic shells is encountered. The vertical effective pressure in the subsoil has remained practically unchanged up to this depth. However, from 28 to

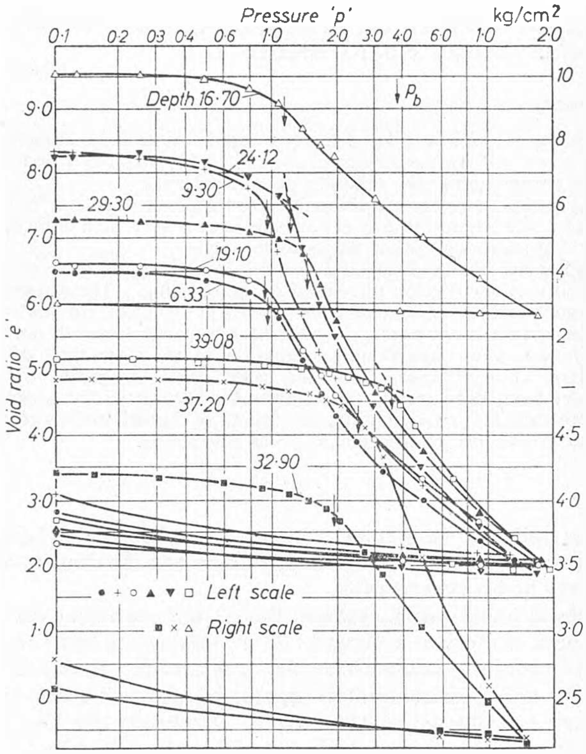


Fig. 2 Compressibility curves
Courbes de compressibilité

42 m depth an important reduction in the piezometric pressures has taken place and the effective pressures have increased; at 36 m depth to 0·9 kg/cm² and at 42 m to 2·0 kg/cm². This situation demonstrates that the ground surface subsidence at this place is primarily due to the compression of layer V of Tacubaya clays and of the Tarango clay I deposits.

The compressibility curves obtained from consolidation tests performed on the volcanic clay at different depths are shown in Fig. 2. From these curves the very high compressibility of this material may be noticed, specially for pressures greater than the break or critical compressive pressure p_b. The value of p_b determined from these curves (Fig. 2) is shown plotted on soil profile (Fig. 1). The comparison of the critical compressive pressure with the overburden effective pressure is highly significant in the settlement analysis (Zeevaert, 1952b).

Investigations on Shearing Strength

The ultimate shearing strength along the shaft of the piles had to be investigated to understand better the behaviour of the friction piles to be used in the foundation design. Undisturbed clay samples close to the shafts of piles driven in other parts of the city were recovered in open-pit excavations, the piles investigated having been driven for several months. From the results of tests (Fig. 3) made on the clay samples and field observations of the clay close to the pile shaft, the following conclusions were drawn:

(a) A thin skin of perfectly remoulded and consolidated clay was observed close to the pile shaft that did not exceed 5 per cent of the diameter of the pile, thus giving rise to shearing strengths (Fig. 4) according to the equation:

$$S_{\phi} = 0{\cdot}53 . K_0 p_v \qquad \ldots . \ (1)$$

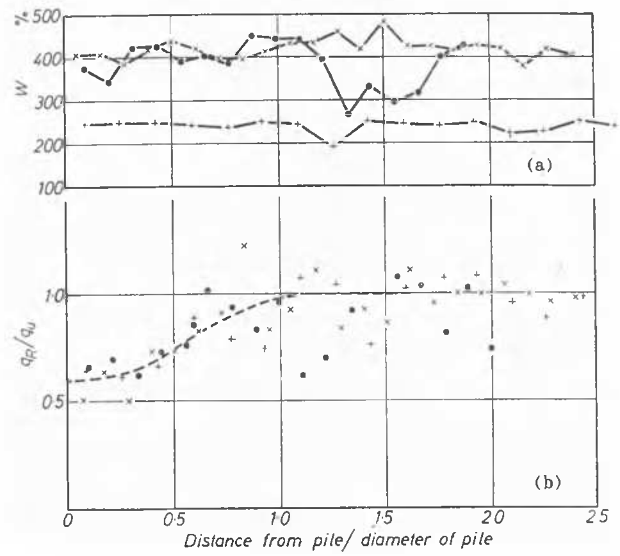


Fig. 3 Shearing strength of disturbed clay close to pile shaft
Résistance au cisaillement de l'argile remaniée près du fût du pieu

where $K_0 = 0{\cdot}75$ is the ratio of horizontal to vertical effective pressure (ZEEVAERT, 1953d).

(b) The water content (Fig. 3) close to the shaft of the pile remains unchanged showing that there is practically no consolidation of the clay during pile driving or afterwards. This fact may be explained if it is considered that the excess pore water pressure induced during pile driving produces momentarily a reduction in the effective pressures and a corresponding tendency for expansion of the clay mass in the vertical direction. Upon dissipation of the excess pore water pressure, the vertical effective pressure assumes its initial value. Therefore, no appreciable consolidation of the clay takes place under these circumstances, except very close to the pile shaft where a thin layer of remoulded clay consolidates because of complete structural breakdown.

(c) The natural shearing strength of the clay, approximately $0{\cdot}5q_u$, is affected by pile driving to about a distance of one diameter away from the pile shaft. Close to the pile shaft the shearing strength of the clay is reduced to about 60 per cent (Fig. 3b). Hence shearing will develop in the semi-disturbed clay at a distance of about $1{\cdot}1r$ from the centre of the pile with a shear strength of approximately:

$$S_c = 0{\cdot}3q_u \qquad \ldots . \ (2)$$

To investigate the total shearing resistance at which a friction pile starts to yield it is necessary to investigate the minimum values of the shearing strength by means of both formulae 1 and 2.

An analysis of the shearing strength per unit length of a concrete pile 40 cm in diameter, using both concepts 1 and 2, was made and is shown in Fig. 5. The average values of q_u for each clay layer to a depth of 27 m are also shown in Fig. 5.

In this particular case the minimum shearing strength is obtained with a combination of the concepts expressed in formulae 1 and 2. After integration between 2 and 24 m depth (h) a total force (F) of 55 ton was obtained for the ultimate load necessary to mobilize the total shearing strength of the soil along the shaft of the pile. Because of the highly compressible clay the point resistance does not play an important part until a rather large settlement of the pile has taken place and the shearing strength along the shaft has been almost entirely mobilized.

Two pile tests were made with 40 cm diameter concrete section-piles pushed slowly with jacks from 2 to 24 m depth. The results of the tests are reported in Fig. 6. Loading test No. 1 was made 28 days after the pile was pushed. The duration of the test was 6 days, and this permitted maximum settlement of the pile to occur for each increment of load. The maximum load reached was 100 ton. The next load increment was not sustained by the pile; the load dropped immediately to 79 ton and a continuous settlement occurred under this load. Loading test No. 2 was made 20 days after the pile was pushed and the test occupied 3 days. A total load of 70 ton was sustained (Fig. 6a).

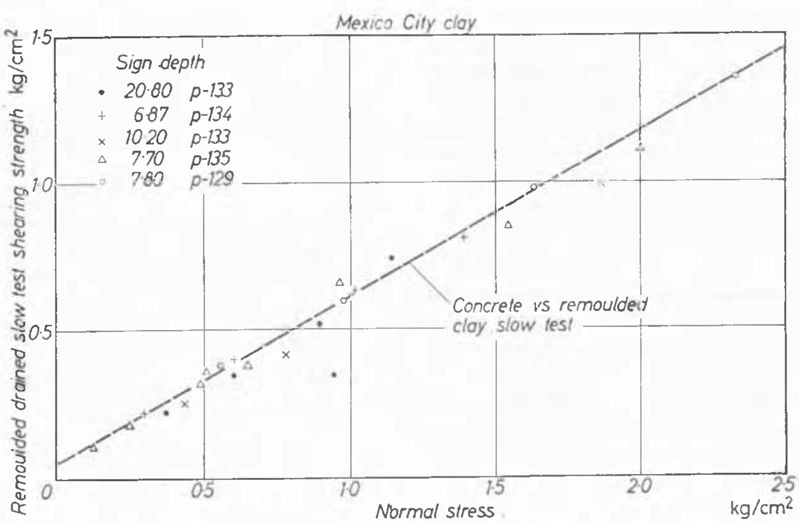
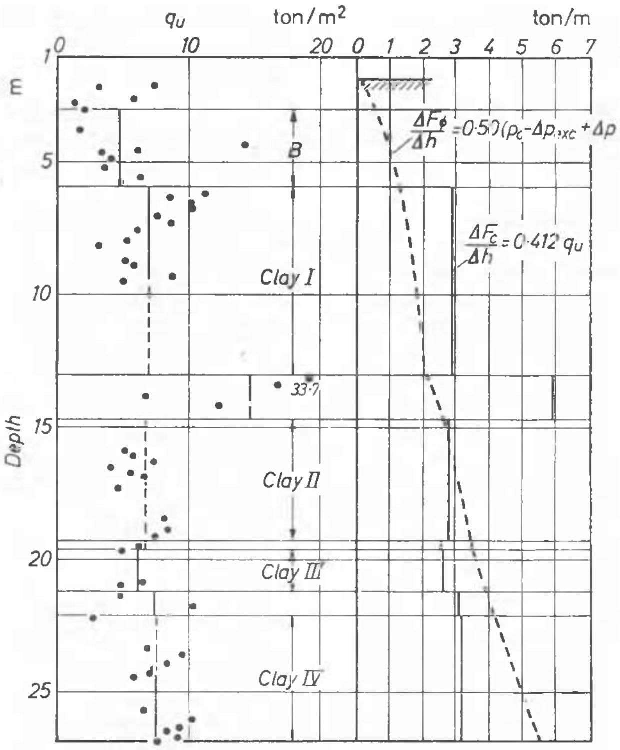


Fig. 4 Shearing strength of Mexico City remoulded volcanic clay
Résistance au cisaillement de l'argile volcanique remoulée de la Cité de Mexico

The pile was pushed into the ground in stages at a speed of 0·4 cm/sec. The minimum load required is given by the curve labelled *A* in Fig. 6b. At the start of each stage, after 10 to 20 minutes, a larger load to break the 'freezing effect' was required. In examining the pushing records it will be recognized that the point resistance played a very important part during pushing, since actually the pushing force remained constant with depth as the point passed through each clay layer.

The point resistance at a depth of 24 m reached a minimum value of 20 ton under the above-mentioned pushing conditions. This fact was again demonstrated in all the piles driven later in



Δp_{exc} = Relief due to excavation to drive piles

Δp = Produced by loaded pile. Assumed equal zero for pile test No.2

Fig. 5 Variation of friction resistance along pile shaft
Résistance du frottement le long du fût du pieu

this locality. Test No. 2, which may be considered as a quick loading test for point resistance, may be interpreted using the results so far explained in this paper. The total load on the pile is equal to the sum of the ultimate frictional strength of 55 ton, plus the point resistance that at least under these conditions was no smaller than 20 ton, thus giving a total load of 75 ton. This computed load is surprisingly close to the measured ultimate load of Test No. 2 (Fig. 6a).

Settlements

The proposed building after total compensation produced a net increment of pressure at 6 m depth of 0·326 kg/cm². The total pressure, including this increment of stress, is given by the curve labelled *A* (Fig. 8).

To reduce heave (Zeevaert, 1955) during excavation and subsequent settlement the author recommended a preliminary excavation to a depth of 3·5 m, corresponding to the depth of the basement. It was recommended that the subsequent 2·5 m of excavation, in which the foundation structure was to be placed, should be made in trenches backfilled with concrete, and that subsequently the spaces excavated between the concrete beams should be filled with water while the superstructure was under construction. The reduction in effective pressure due to excavation to a depth of 3·5 m only is equivalent to 0·384 kg/cm² (Fig. 1). The curve labelled *B* in Fig. 8 shows this reduction with depth.

An estimate of the settlement was computed using information obtained from subsoil tests and the curves labelled *A* and *B* in Fig. 8. Table 1 shows the computation. The total

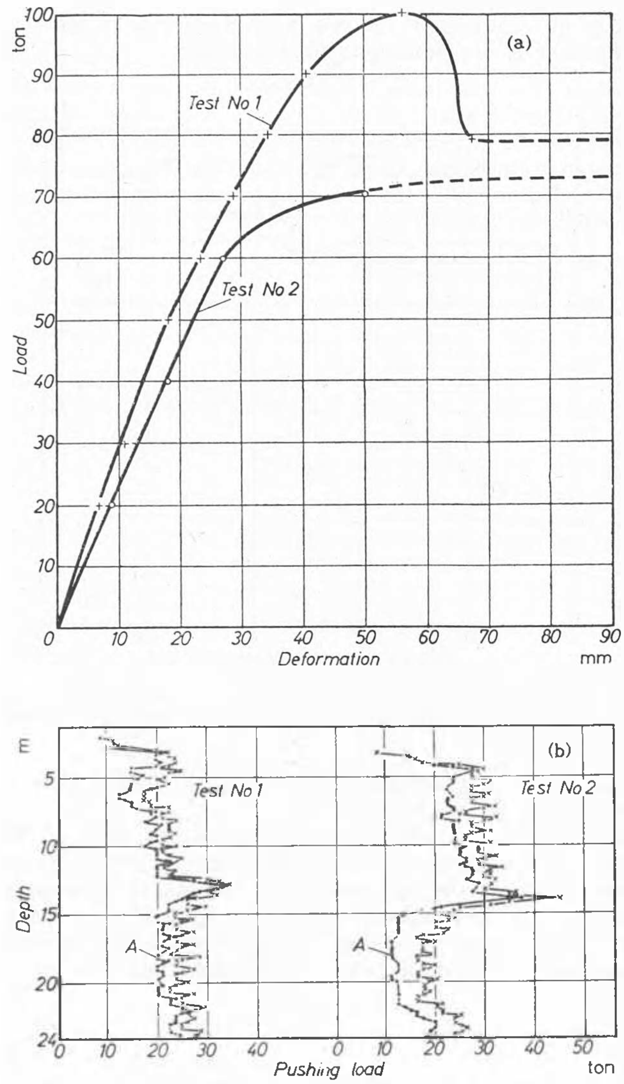


Fig. 6 Tests on friction piles
Essais sur pieux de frottement

Table 1

Depth m	e_0	m_v	*ΔH* cm	*Δp* kg/cm²	*Δs* cm
6·33 9·30	6·5 8·34	0·0714	710	0·665	33·75
16·70	10·1	0·0973	465	0·475	21·52
19·10	6·6	0·113	160	0·40	7·22
24·12	8·5	0·1275	470	0·32	19·20
29·30	7·3	0·1505	360	0·225	12·20
32·90	4·2	0·0481	460	0·18	3·98
37·20	4·91	0·0423	180	0·15	1·14
39·08	4·65	0·0408	220	0·13	1·17
				Total	100·18

settlement obtained was of the order of 100 cm. This value is considered conservative since in some places the critical pressure was reached. A more precise settlement analysis was made as a function of time taking into account the construction period and the secondary time effect using the settlement

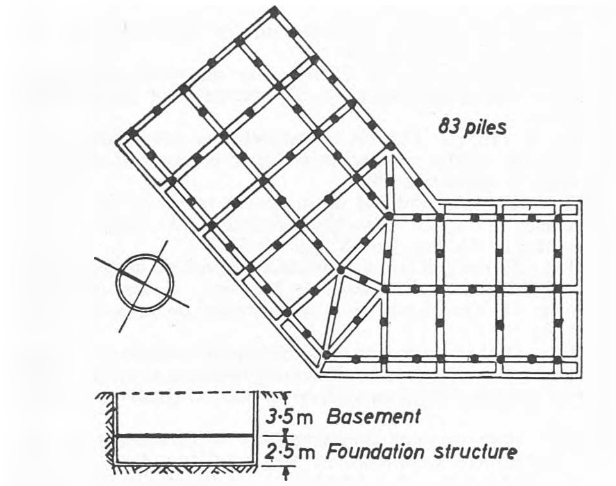


Fig. 7 Foundation plan and pile layout
Plan de fondation et localisation des pieux

analysis method described elsewhere (ZEEVAERT, 1951, 1953c). The results are reported in Fig. 9 curve *a*.

The settlement for the compensated foundation without piles is considered very large. Experience in Mexico City has

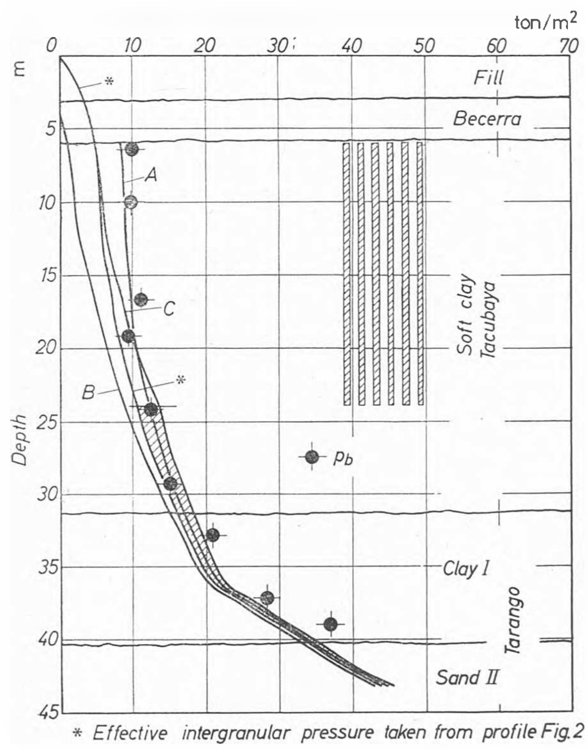


Fig. 8 Total stresses in subsoil
Contraintes totales dans le sous-sol

shown that differential settlements between buildings should remain under 10 cm in the first year and a total of less than 30 cm should not be exceeded in a long period, in order to avoid damage to neighbouring buildings and public utility services.

The very large settlement mentioned above favoured the decision to use friction piles to take care of the excess pressure of 0·326 kg/cm². The 83 piles used were driven to a depth of 24 m into the clay deposit unaffected by the reduction of piezometric pressures, thus assuring that in the future the building would follow the ground surface subsidence instead of emerging above the ground surface.

The friction piles served only to reinforce the clay deposits between 6 and 24 m depth, and also to reduce expansion during excavation and re-compression of the highly compressible clay surrounding the piles. The excess load of 0·326 kg/cm² produces a load of 26·0 ton per pile. Under these circumstances the block formed by the soil and the piles was assumed to act as a unit, transmitting the excess pressure in a favourable way to its base and on to the less compressible clay deposits.

The total stresses in the subsoil under this condition were computed by current methods and are given as curve labelled *C* in Fig. 8. The net increment of pressure was used for settlement computations as given in Table 2. The total settlement

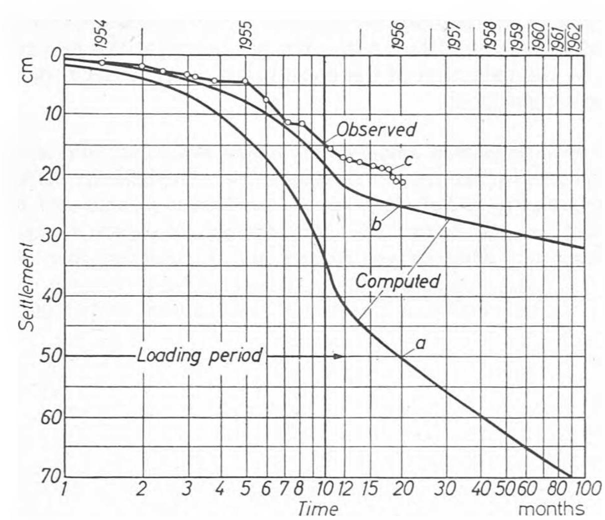


Fig. 9 Settlement curves
Courbes de tassement

obtained was of the order of 37 cm. The settlement *versus* time (ZEEVAERT, 1953c) was also estimated and is recorded in curve *b*, Fig. 9.

Table 2

Depth m	e_0	m_v	ΔH cm	Δp kg/cm²	Δs cm
24·12	8·26	0·149	280	0·285	11·89
29·30	7·29	0·202	360	0·25	18·18
32·90	4·21	0·048	460	0·14	4·19
37·20	4·92	0·0422	180	0·17	1·29
39·08	5·15	0·0348	220	0·14	1·07
				Total	36·62

Settlement observations were started on every column of the building as soon as the foundation beams were placed in the ground. The average observed settlement is referred to the centre of gravity of the loaded area and is recorded in Fig. 9, curve *c*. The building was unfortunately designed so as to be heavier towards the tower of elevators, stairways and sanitary services. Nevertheless, up-to-date differential settlements have been only of the order of 3 cm and no further increase has been noted.

Conclusions

From settlement computations and observations reported in Fig. 9, the great advantage of using a compensated friction pile foundation may be noticed. The following conclusions may be summarized from the present case and other similar foundations solved by the author with the same philosophy:

(a) Using friction piles the settlement in the very high compressible clay deposits was considerably reduced.

(b) A careful study and proper interpretation of stratigraphy, hydrostatic conditions and soil properties was necessary to obtain adequate information so as to design successfully the compensated friction pile foundation.

(c) The friction piles, designed with a shearing stress well below the shearing strength of the clay surrounding the pile shaft, acted as a unit with the soil. Thus, consolidation of the clay between the piles appears to have been eliminated. The excess pressure was taken by deeper and less compressible clay layers.

(d) The use of the compensated friction pile foundation eliminated the undesirable differential settlement between the building and the surface of the ground, caused at this particular site by the reduction of the piezometric water levels at depths greater than 28 m.

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