Main Towers of La Plata City Cathedral. Reinforcement of Foundations.

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SUMMARY: The Cathedral of La Plata City, in Argentina, is a monumental neo-gothic church completely built in massive baked brick masonry. Foundations were opened on November 19, 1885, and 47 years later the Cathedral was officially inaugurated, even though the two main Towers had not been concluded.
The spread foundations of the unfinished Towers (nowadays only 40m high) are placed about 5 to 6 metres below street level and weigh 7,000 tons. To finish them, a load of the order of 2,000 tons per Tower should be added.
Due to the magnitude of the new loads, it was necessary to improve the existing foundations. For this purpose, large sets of micropiles have been designed and calculated. This paper describes the adopted solution, calculation parameters and performed load tests.

KEY WORDS: Micropiles – Foundation Reinforcement – Load Tests – "Pampeano" Formation.

1 INTRODUCTION

The Cathedral of La Plata City is a monumental religious building constructed in neo-gothic style; its colourful external appearance is very peculiar because its bearing walls are built of red bricks. Its approximate dimensions are 120 m x 75 m, and its 7,000 m^2 of useful surface can shelter 12,000 people. Its construction began in the last decades of last century; however, its two main symmetrical Towers are currently being completed to reach their final height of 112 m above the street level (Figure 1). Their present height is of the order of 40 m.; each Tower weighs approximately 7,000 tons.; the new load is expected to be of about 2,000 tons. The Towers are founded at about 4 or 5 metres deep with respect to the site surface; the overall supporting area of each Tower is approximately 160/170 m^2. The subsoil geotechnical characteristics are appropriate for the present loads "Trevisán & Mauriño (1963)" and "Nuñez & Trevisán (1993)", but the new loads derived from the completion of these Towers determine that it is necessary to reinforce the present foundations "Trevisán & Nuñez (1996)". To fulfil this requirement, high-pressure laterally post-injected with cement grouting concrete and steel micropiles have been used; all of them are embedded into the foundation masonry and joined by means of a reinforced concrete cap.

2 SUBSOIL CHARACTERISTICS

La Plata City subsoil belongs to the "pampeano" formation, made up of preconsolidated by desiccation silty clays "Bolognesi (1975)" and "Nuñez & Micucci (1986)". These quaternary sediments of about 45 m thick lay on medium and fine dense sand, from the "puelchense". It is considered that the upper two thirds of the "pampeano" formation have an aeolian origin whereas the lower third has a fluvial one. The sedimentary package has experienced in all its thickness the effect of dry and wet periods, and the consequent action of desiccation and cracking. Besides, carbonates and oxides of calcium and magnesium have cemented inhomogeneously the original silts and clays by precipitation, infiltration and/ or impregnation. All these circumstances have determined the presence of more or less continuous layers, and erratically distributed lens and banks, with different compressibility and strength properties. Thus, soils with relatively low calcareous impregnation, called "suelos toscosos", are compact to very compact soils; but the highly cemented parts –regionally called "toscas"– are very hard soils with so highly resistant properties that, in some cases, can be classified as "very soft rock". The aforementioned heterogeneity does not allow pointing out a priori the subsoil morphology and mechanical properties of a determined location. Moreover, the original water tables – which belong to a general system that drains towards La Plata River– are currently depressed because of the existing plumping action in the city to supply water to the urban area. In fact, in all the works performed in relation to this project, the level of underground free water could never be reached.

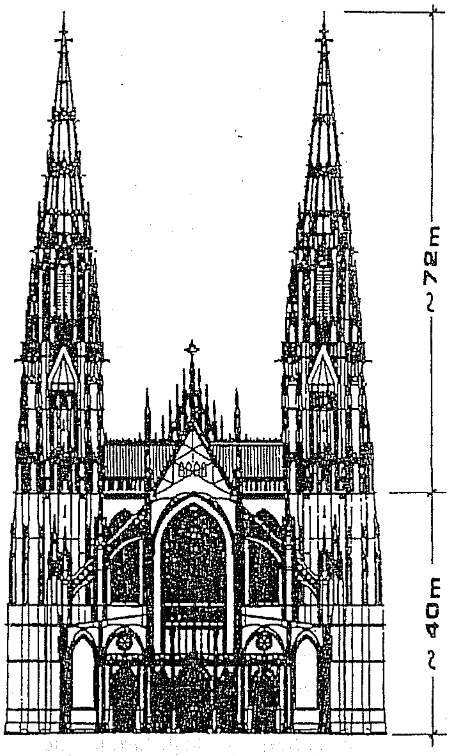

Fig. 1. La Plata front view (with Towers).

3 PREVIOUS WORKS AND THEORETICAL ANALYSIS

3.1 Geotechnical Research

Considering the general knowledge of the "pampeano" formation, the geotechnical research mainly aimed at determining the particular stratigraphic conditions of the site where the Towers are located. The employed methodology was the conventional one which includes the use a manually driven drill and injection of light bentonitic mud in order to facilitate the advance of the tool and the cleaning of the excavation detritus. Every 1 m a standard penetration test was done by means of the sampler with interchangeable shoes commonly used in Argentina which is driven into the ground applying the same technique as that employed for the SPT; a specimen of about 2" in diameter that is inside a PVC tube inserted in the inner part of the sampler –which later allows a hermetic sealing– is collected.

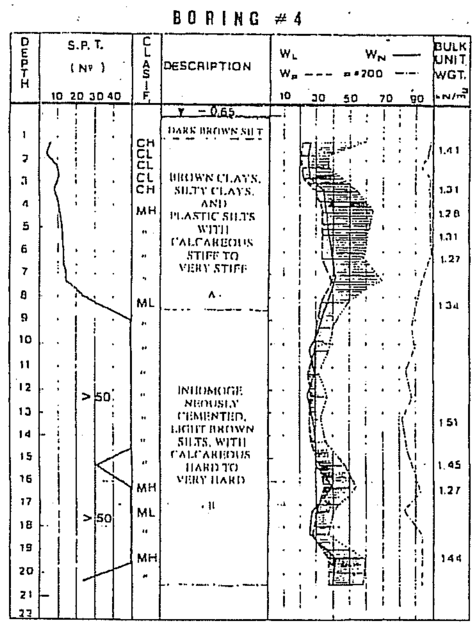

Fig. 2. Subsoil profile under La Plata Cathedral.

The obtained number of blows N' correlates quite well with the value N of the SPT by means of the expression N = 0,8 N'. With these "representative" samples, the following parameters are determined: natural water content w_n, liquid limit w_l, plastic limit w_p, and the percentage that goes through the nº 200 sieve; out of those samples that do not show visible signs of alteration, unit weights are determined and stepped undrained triaxial tests are carried out. Soils are classified according to the A. Casagrande System. The results of all the tests obtained on the site and in the laboratory were translated to a stratigraphic diagram which allows distinguishing main features from physical and mechanical characteristics of subsoil layers. In the case we are analysing, 4 perforations were made: 2 in each Tower; Figure 2 shows a stratigraphic table including typical results. Complementarily, open pits performed beside each Tower were used to take "undisturbed samples" in order to test compressibility and strength. Figure 3 shows some typical results of triaxial tests carried out on samples of compacted and very compacted soils. It also includes algebraic expressions that give a very clear idea of the notation used to define the geotechnical parameters employed in the analysis.

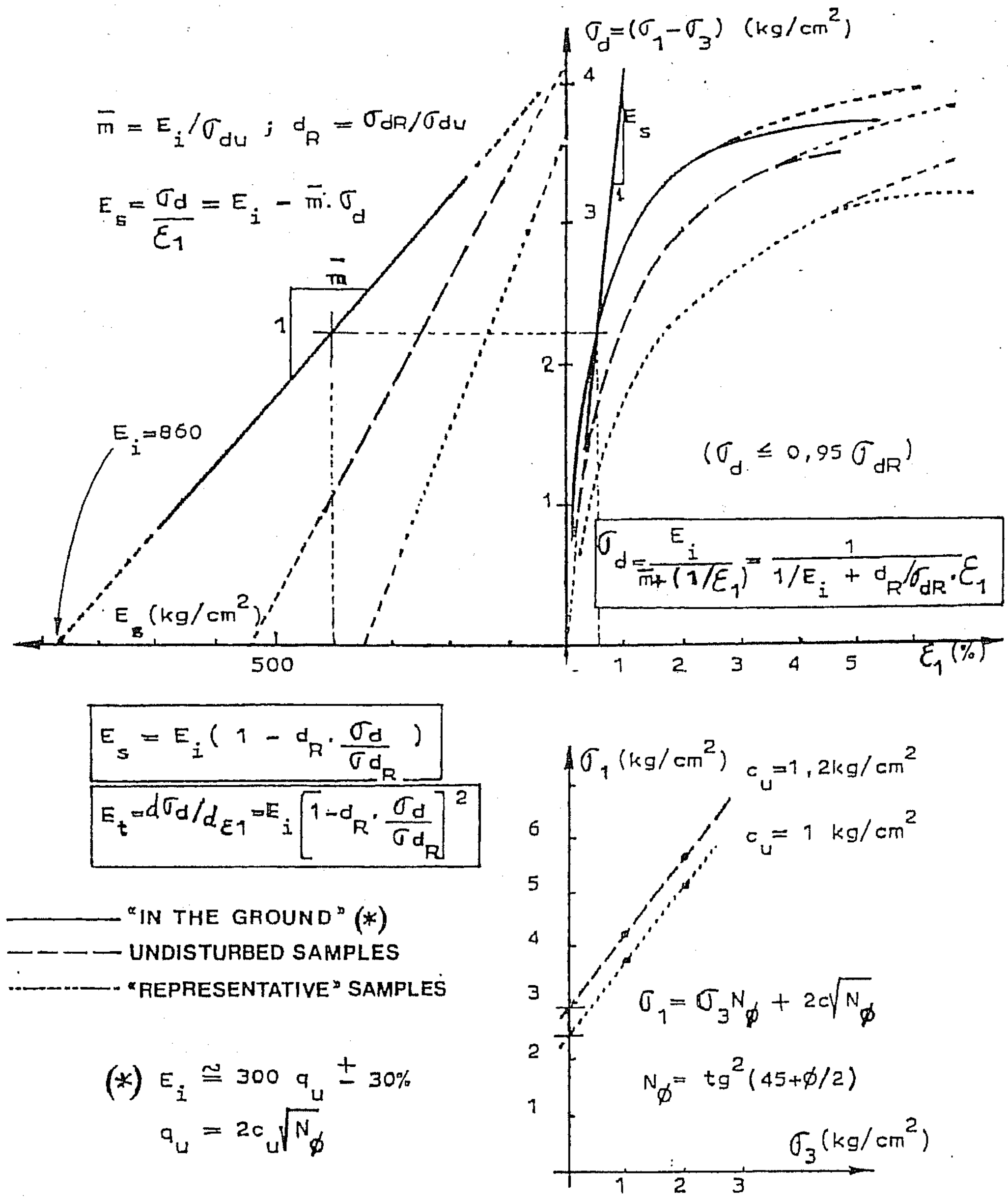

Fig. 3. Stress-strain-strength-modulus relationships.

3.2 Selection of Mechanical Parameters for the Design.

After a pondered analysis of the values obtained from the triaxial tests performed for this project and on similar soils belonging to the "pampeano" formation, some comments can be made: a) The c_u values are consistent; b) The ϕ_u values are within a narrow range, with an average of about 10°; c) The "representative" samples tested show consistently lower c_u and ϕ_u values than those obtained out of "undisturbed" ones; this shows the deterioration of the cement bonds among particles derived from the dynamic action of the sampler; d) The correlation between the soil saturation degree (approx. 80%) and ϕ_u seems reasonable; e) A lineal relation between the secant modulus $E_s = \sigma_d$

$/\varepsilon_1$ and $\sigma_d = (\sigma_1 - \sigma_3)$ can be defined; this means a parabolic relation between E_t (tangent modulus) and σ_d, and a hyperbolic variation between σ_d and ε_1, in agreement with what Kondner demonstrated; f) The comparative analysis of the results obtained from the tests of undisturbed and representative samples, and those derived from retrospective analyses of the actual behaviour of structures, show that the fall of strain modulus due to the sampling is very significant whereas the reduction of strength is much lesser. Consequently, in the case of the so-called "suelos toscosos", located between the foundation level of the Towers and an approximate depth of 9 metres, values not inferior to

$c_u = 10$ Tn/ m^2, and

$E_i = 7{,}500$ Tn/ m^2

are adopted; for underlying "toscas" and "suelos toscosos",

$c_u = 12$ Tn/ m^2, and

$E_i = 2 \times 7{,}500 = 15{,}000$ Tn/ m^2.

For the whole subsoil profile affected by the foundation, $\phi_u = 10°$ is considered. For soils belonging to the "pampeano" formation similar to those of the upper part of the stratigraphic profile, a quite good correlation is

$E_i \cong 300\ q_u \pm 30\%$

So, on the site,

$E_i = 300 \times 2 \times 1.2 \times 1.19 = 860$ kg/cm^2

can be considered; in some cases, this value is conservative.

3.3 Conditions of Original Foundations

Figure 4 clearly shows the original situation: the foundation level in the high part (zone A) at a few metres of the "roof" of the very resistant layer. The acting pressures, with a load of 7,000 tons each Tower, exceed 4 kg/cm^2; consequently, to bear the new loads derived from the completion of the structures, it is necessary to reinforce the foundations.

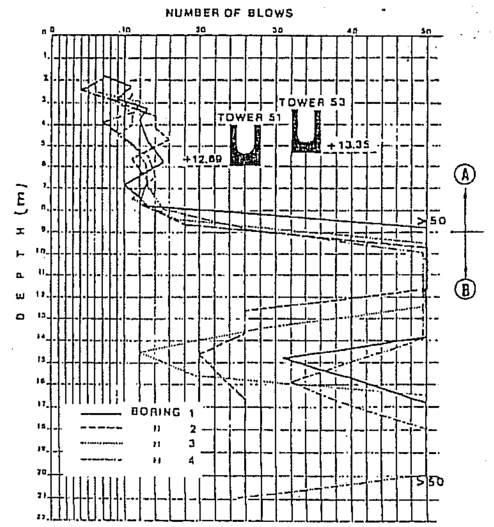

Fig. 4. Standard Penetration Test (SPT) and present foundation depths.

3.4 Selection of the System to Reinforce Foundations. Previous Assumptions.

Among all the analysed alternatives to reinforce the foundations, the construction of a group of micropiles embedded into the existing foundation masonry was chosen "Nuñez & Trevisán (1996)". At the very beginning, it was considered to use 12-metre long and 15-cm in diameter micropiles with a central metal tube through which fine grain concrete is pumped to fill in the play between the hole walls and the metal tube.

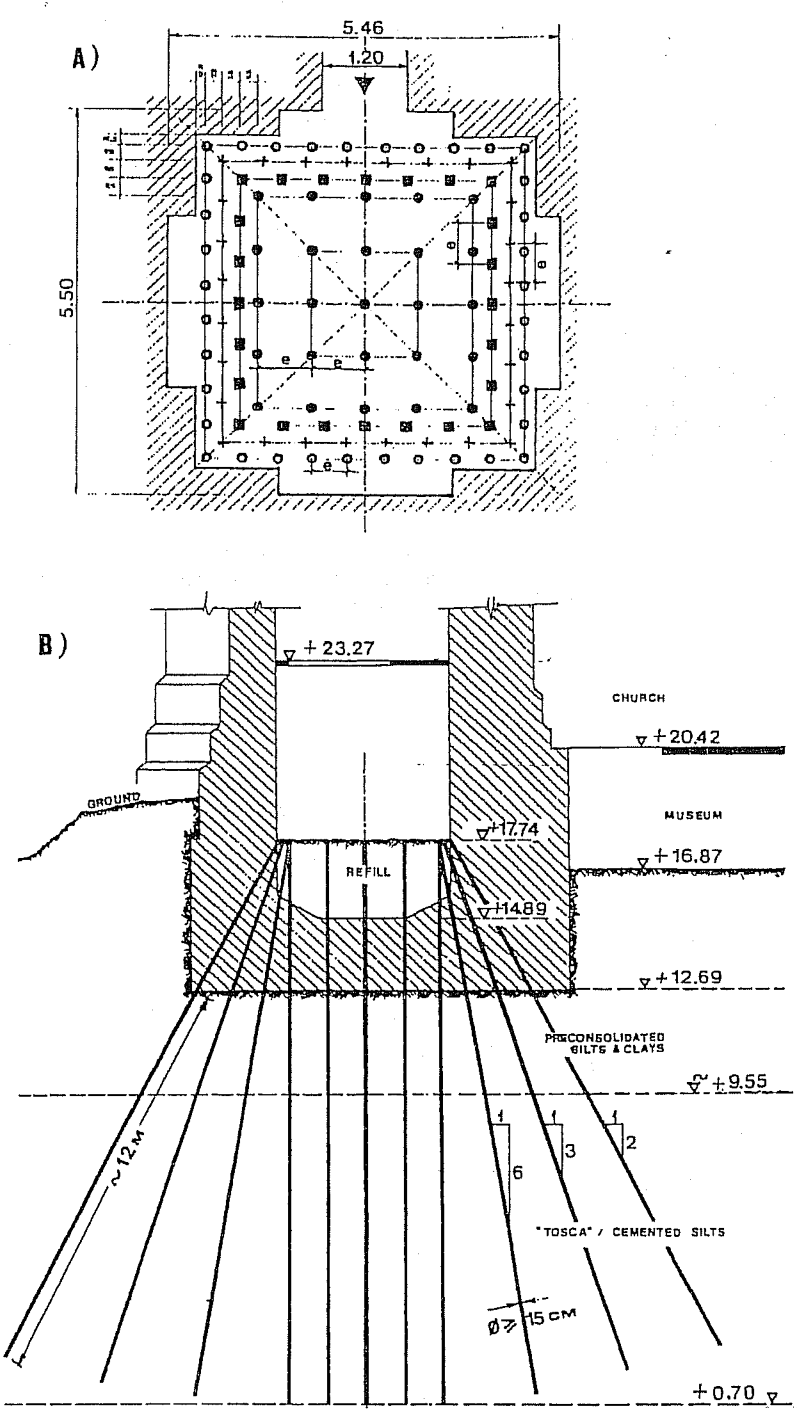

Fig. 5. Tower 51. Reinforced foundation with micropiles: a) Plan; b) Section.

The metal pipe, or "framework", is built with some elastic sleeves, or "rubber manchette", which cover the orifices through which cement grouting can be injected at high pressure after breaking the cylinder of concrete in the process of hardening. Thus, a lateral deformation of the ground and a possible radial cracking are produced which allow the penetration of the grouting into the subsoil. It was considered to establish a minimum of three layers of injection, ridges of the metal tube to ensure the shearing of the concrete-steel interface, minimum pressures to pump concrete and injected cement grouting, and a concentric layout of the micropiles with an inclination of up to 2 (V): 1 (H) for the exterior covering set (Figure 5). Taking into account the available data, the load capacity was estimated as follows:

Unit friction:

$f_{s\,aver.} = 0,3 \times 10\ Tn/m^2 + 0,7 \times 15\ Tn/m^2 = =13,5\ Tn/m^2$

Minimum calculation friction:

$f_{s\,calc.} = 2/3 \times 13.5\ Tn/m^2$

$f_{s\,calc.} = 9\ Tn/m^2$

Allowable concrete-steel friction:

$f_{s\,all.} = f_{s\,calc.}/ 3 = 3\ Tn/m^2$

Point unit strength:

$P_R = 600\ Tn/m^2$

$P_{all.} = 600/3 = 200\ Tn/m^2$

Micropile settlement load:

$P_R = 3,14 \times \phi_e \times 12m \times 9\ Tn/m^2 + A_p \times 600 Tn/m^2$

It was decided to make perforations of 0.15m diameter, so:

$P_R = 51\ Tn + 10.6\ Tn = 61.6\ Tn$

$P_{all.} = 61.6 / 3 = 20.5\ Tn$

Consequently at first the number of micropiles was 100 units for each Tower; however, the final number was 113 to have a symmetrical distribution. To estimate the settlements, the following calculation line was used:

a) Compared to its initial value, the secant modulus decreases as the stress increases in relation to the final failure value. An expression that represents this variation is:

$E_s = E_i\,(1 - d_R \,.\, \sigma/\sigma_R)$.

In the same way, the values of the subgrade reaction coefficient can be:

$k = k_i\,(1 - \overline{d}_R \,.\, p/p_R)$

and also:

$K = K_i\,(1 - \overline{d}_R \,.\, P/P_R)$ being $K = k \,.\, A$.

b) For the first layer:

$E_{i1} \cong 300 \times 2 \times 1.2 \times 1.2 \cong 860\ kg/cm^2$.

For the second layer, a value equal to the double of the first one can be estimated. If the lower value is considered,

$E_{i1} \cong 0.7 \times 860 = 600\ kg/cm^{-2}$, and

$E_{i2} \cong 2 \times 600 = 1{,}200\ kg/cm^2$.

c) Accepting: 1) a lineal distribution with the "equivalent" depth of the pressure that transmits the foundation to a depth 2B = 2 x 12.8 = 25.6 m; 2) the first layer of about 4 m thick; and 3) an average contact pressure of 4.3 kg/cm^2, a "global modulus E" can be introduced by means of this equation:

$\delta = p_{m1} \times 400\ cm/ 860\ kg/cm^2 +$

$p_{m2}\ 2{,}160\ cm/2 \times 860 = (4.3/2) \times 25.6/E$,

being $p_{m1} = 4.3 \times 23.6/2 \times 12.8 =$
$= 3.964\ kg/cm^2$, and

$p_{m2} = 4.3 \times 10.8/2 \times 12.8 = 1.814\ kg/cm^2$,

so:

$E \cong 4.3 \times 2{,}560/ 4.2\ cm \times 2 \cong 1{,}310\ kg/cm^2$.

This value is taken as the "initial global modulus of the foundation subsoil".

d) $k_i \cong 1.5\ E_i/B \cong 1.5 \times 1{,}310/1{,}280 \cong$
$\cong 1.54\ kg/cm^3 = 1{,}540\ Tn/m^3$

and also:

$K = k \,.\, A = 1{,}540 \times 164\ m^2 = 2{,}520\ Tn/cm$

($A = 164\ m^2$, footing area).

From the expressions described in a), it can be said that:

$\delta = P/K\ .\ (1 - d_R\ .\ P/P_R)$

provided that the initial settlement for the acting load has been:

$\delta_o = 7{,}000 / 2{,}520\ (1 - 0.8 \times 7{,}000 / 16{,}400) \cong \cong 4.2$ cm,

and that, with a live load: 2,000 tons, is:

$\delta_o + \Delta\delta = (7{,}000 + 2{,}000) / 2{,}520\ (1 - 0.8 \times (7{,}000 + 2{,}000) / 16{,}400 = 6.4$ cm;

i.e.: $\Delta\delta = 2.2$ cm.

e) It has been considered that the construction of micropiles may prestrain the soil and/or eventually reinforce it by penetrating the cement grouting with pressure "Nuñez (1977)". It is expected that the pile should "answer" the action of the loads with a value of the "spring coefficient":

$K_i = 100$ Tn/cm $\pm$ 30%.

For the pile, $d_R = 0.9$ has been projected; so,

$\delta_{MP} = P_{MP} / K_i\ (1 - 0.9 \times P_{MP}/P_{MPR})$.

The value of P_{MPR} was considered equal to 60 tons and thus, the deformation due to the 20-ton load is of about:

$\delta_{MP} = 20/ 100\ (1 - 0.9 \times 20/ 60) = 0.3$ cm.

Although each micropile is relatively far from the surrounding ones, a magnification of 2.5 times has been considered, taking into account the action of the group. The $(P - \delta)$ values are shown numerically in Table 1, and graphically in Figure 6

Table 1: Present Footings and Micropiles

FOOTINGS		MICROPILES			
P	δ	P_{MP}	δ_{MP}	$P_G = 100\ P_{MP}$	δ_{GMP}
Tn	cm	Tn	Cm	Tn	cm
7,000	4.2	5	0.054	500	0.135
8,000	5.2	10	0.118	1,000	0.294
9,000	6.4	15	0.194	1,500	0.484
10,000	7.8	20	0.286	2,000	0.714
11,000	9.4	30	0.545	3,000	1.364

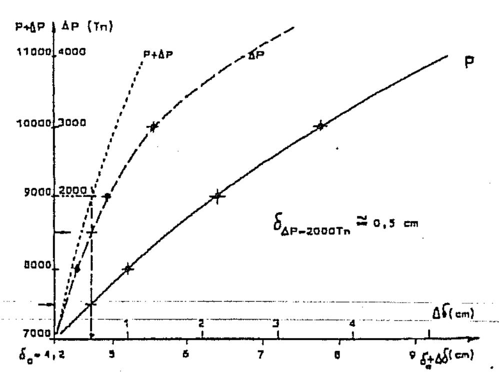

Fig. 6. Interaction between the present foundations and the group of micropiles.

f) It is probable that the only action of injection and compression of injection in correspondence with layer 1, changes it into a sufficiently resistant material so as to transfer the new loads to layer B. In this case, the deformability of the latter should be taken into account and its corresponding bearing capacity is:

$k = 1.5 \times 2 \times 860/ 12.8 = 2{,}040$ Tn/ m^3;

$K = 2{,}040 \times 164m = 3{,}350$ Tn/ cm

$\Delta\delta = 2{,}000 / 3{,}350\ (1 - 0.8 \times 2{,}000/ 3 \times 6{,}000) = 0.65$cm.

4 IN SITU WORKS

4.1 Implementation of the Micropile System.

Micropiles were built following the general specifications included in the original design. A steel metal tube was used $\sigma_e = 4{,}200$ kg/ cm^2, $\phi_{ext} = 60.3$ mm and e = 8.74 thick. Fine grain concrete H $\geq$ 25 had a cement content no lower than 380 kg/m^3 using a superfluidificant and an expander, with a relation water/ cement $\cong$ 0.35 to 0.4. To make the perforation, a rotopercuting equipment with a air-sweep back hammer was used.

The groutings had a relation w/c $\cong$ 0.6, using a pump with a capacity of 80kg/cm^2. The quantity of injected cement per valve varied from 50 kg in the case of inner vertical micropiles to 150 kg in the case of outer inclined ones.

The "cracking" of the annular concrete was generally made between 8 and 10 hours after its pouring, and pressures varied between 15/20 kg/cm^2 in upper levels and 50/60 kg/cm^2 in deeper valves.

4.2 Micropile Load Tests

Before building the micropiles inside the Towers, a test to adjust methodology, sequences, mixtures and pressures was carried out. Figure 7 shows the load test results of a "prototype" pile which was built in the nearby area, outside one of the Towers.

Besides, Figures 8.A and 8.B show the results of load tests on two randomly chosen micropiles, one of each Tower. These tests prove a good correlation between the projected P_R values and the extrapolated ones, but the K_i values are higher than those previously estimated. Besides, up-lift tests were performed in structural bars embedded in the Tower foundation masonry. In the case of load tests carried out inside the Towers, existing micropiles closely-tied to the foundation masonry were used in order to dispose of the necessary anchorage reactions. In all the tests, it was proved that the behaviour of the transference members joint to the masonry was satisfactory within the stress level considered in the design.

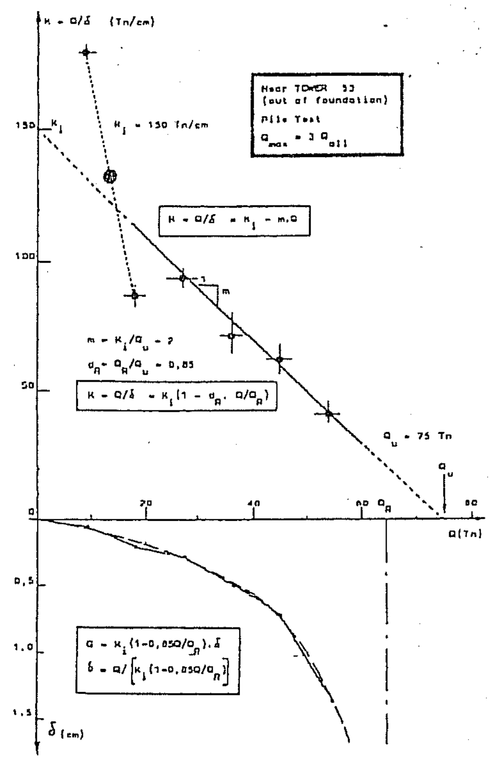

Fig. 7. Load test on a prototype micropile (outside the building).

5 ANALYSIS OF THE WORKS CARRIED OUT AND PREDICTION OF THE BEHAVIOUR OF REINFORCED FOUNDATIONS.

a) The foreseen load capacity is very similar to that one observed in the interpretation of load test results.

b) The spring coefficient K_i is higher than the one projected. Consistently, the value is higher in the case of the micropiles that form part in a group, something that is coherent if it is understood that the foundation ground has experienced a significant lateral predeformation.

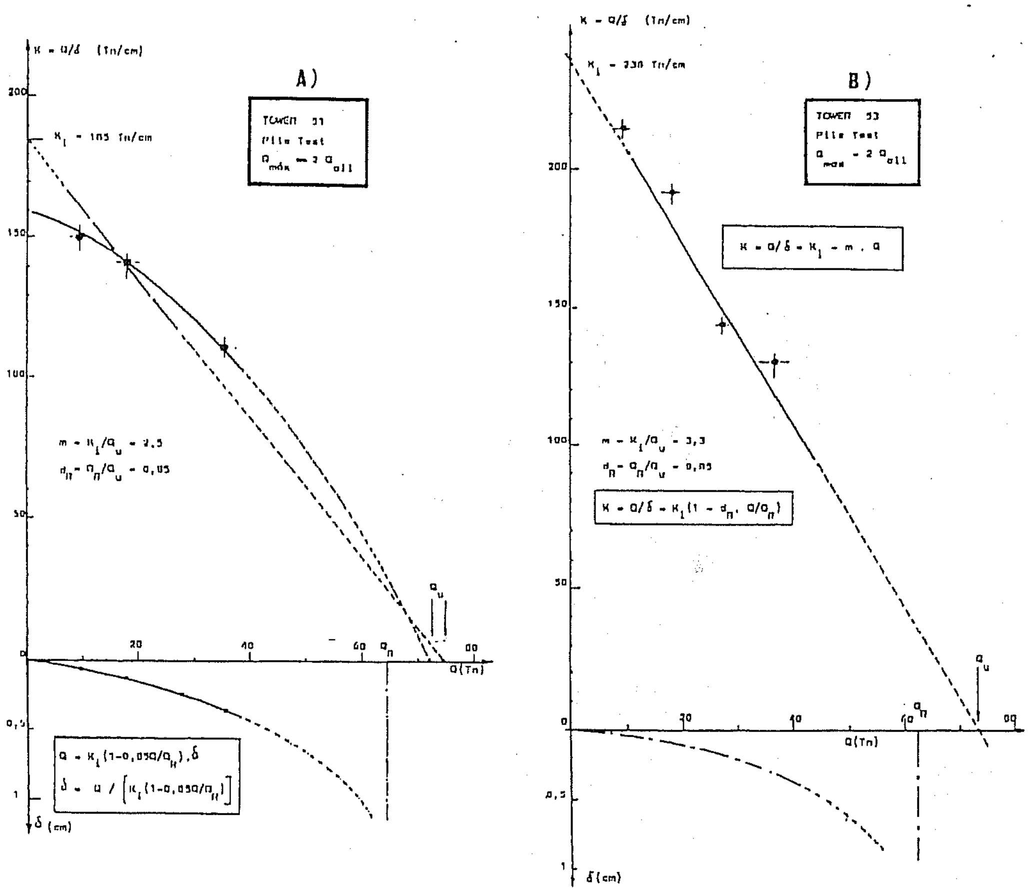

Fig. 8. Micropile load tests: A) in Tower 51; B) in Tower 53

c) Regarding the volumes of injected cement and employed pressures, no ascending movement of the foundations was observed in both Towers during the works.

d) If in the calculation of the design carried out before the pilework, the amplification factor is increased and, at the same time, the actual values of K_i are considered, the planned magnitude of the reinforced foundation settlement does not practically change.

6 CONCLUSIONS

The works of completing the Towers up to their final height are currently being carried out without a break. Settlements in relation to the increase of the applied new loads are registered. At this moment, the increase of load is of the order of 20/25% of the final new load being the settlement less than 1mm. Taking into account all test results and observations to this day, it can be concluded that this type of micropiling can be built in a relatively easy and economical way; this is so even when micropile diameters and lengths are significantly increased, and consequently, the capacity of these structural members suitable for transferring loads deep into subsoil.

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