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How to continue La Plata City Cathedral – Geotechnical approach

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ABSTRACT: The upper part of the two symmetrical unfinished towers will be added to complete the Cathedral. The spread foundation rest directly on the shallow portion of Pampeano formation,wich is a soil lightly cemented by carbonates, 3 to 4 m. above of very hard cemented "tosca" soil. Due to the magnitud of additional loads, it will be necessary to improve the present foundation.

1. INTRODUCTION

The general characteristics of the Monument have been described in the first part (10) of this presentation. A geothecnical analysis was performed, to gather all the technical information necessary to take the final decission on the foundation design in order to continue the Chathedral and complete the towers.

The calculation hypotheses and values were:

1.Weight of existing masonry work	7000 Ton.
2.Estimated weight of tower enlargement	2000 Ton.
3.Total weight: 1 + 2	9000 Ton.
4.Wind pressure	250 Kg/m^2
5.Wind action	100 Ton.
6.Lever Arm of wind force in relation to foundation level	75 m.
7.Foundation Ground contact area	164 m^2

2. THE FOUNDATION SUBSOIL

The Cathedral foundation was built the last decades of the past century. It rest approximately five meters below the ground surface, and its principal features are common with the spread type one.

The subsoil is typical of the region, named Pampeano formation. (1) (2) (3) (4) (5). It is a thick sedimentary deposit of silty clays and clayed silts; the third lower part of the profile has a fluvial origin, and approximately the upper two third has an eolian origin. The soil were deposited and redeposited along the Quartary in an alternatively humid and dray environment; consecuently, all the soil profile is preconsolidated by dessication. Moreover, soils are inhomogenously cemented by calcium, carbonate and oxides. This cementation is weak in the upper part A of the profile (Fig.1) but strong in the middle deposit, portion B, where cemented soils appears as lenses or more or less continuous strata of irregular thickness. Generally, its color is brown and are locally named "tosca soil", but when the calcium carbonate are abundant and infiltrate the "matrix", the color is more light, the strength is greater and the name is simply "tosca". Generally they are fissured soils, and with a macroporous structure; due cementation they are very stiff to very hard with very low compressibility. All the cities located along La Plata River and Parana River, from Magdalena to San Lorenzo - including La Plata City and Buenos Aires City- are founded on this Pampeano formation; this circumstances permit to build up very high structures resting on shallow foundation as isolated bases and/or rafts. Generally the upper and middle part of the profile are above the ground water level. Then, this soils are partially saturated with saturation value around 80% to 90%. The coeficient of permeability is relatively high due to open structure and fissures, and its values fall between 5 x 10(-4) to 10(-5) cm/sec.

Fig.1 shows clearly the difference between zone A and B. In this figue are included the results of penetration tests obtained in the borings located close to each towers.

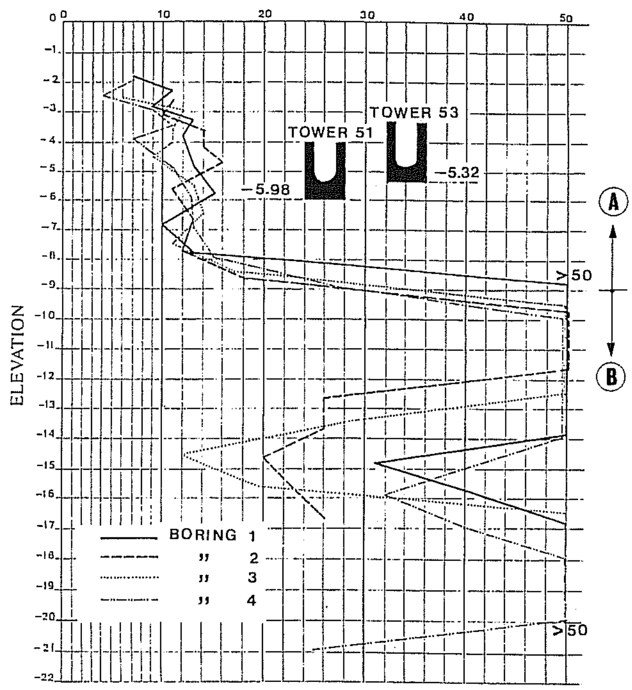

FIG.1 STANDARD PENETRATION TEST

3. THE ROUTINE GEOTECHNICAL INVESTIGATION

Fig.1 shows also the foundation level of each towers (numbers 51 and 53 refer to the name of the street near to each one) and allows visualize clearly that the towers are not founded on the "hard" level of "toscas" of the Pampeano formation -level B- but on the "tosca soils" with light cementation that distinguish the upper part of soil profile, here called "level A". On the other hand, the 3 to 4 meters of distance from the foundation level to the hard soils, allow to conclude that any transference of loads to more deep strata would be performed with small dificulties.

In fig.2 it can be seen the significative values obtained in the routinary subsoil investigation, as is commonly carried out following the local metodology (6).

A sampler Øe=2½", with variable and interchangeable thin cromo-niquel steel shoe with internal plastic linner is used. Shoes have variable lengths and an area ratio Ar (%) according to the soil resistance. The sampling procedure is similar to the Mohr's "wash boring method", though generally the steel casing is not used and the bore wall is stabilized with bentonite slurry. The sampler is driven into the ground following the same methodology as used in the SPT; in this case, the number of blows per foot of penetration is N´. The N value of the SPT is N~0.8 N'. A "representative" sample is recovered every meter. This permit to get values of natural water content W_N, Atterberg limits W_L and W_P and granulometric characteristics.

If the soil is lightly cemented, the samples are relatively competent to determined the unit weights and the approximate values of shear parameters Cu and Øu using an individual sample and appling the multiple stage triaxial test technics (7). Generally the strength values from this tests are less than the actual ground values, but permit to consistently correlate to the penetration resistance obtained in the SPT performed in the field. This methodology allows to the local expert, a contact and direct observation to the soil sample, and is of great support for typification and characterization of the subsoil that is being investigated.

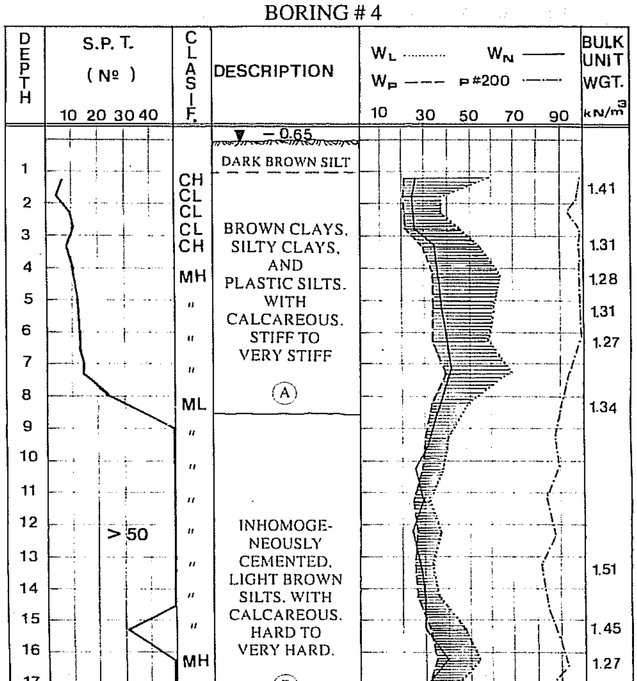

FIG.2 TYPICAL LA PLATA SUBSOIL PROFILE

4. THE USE OF THE SOIL PARAMETERS

In fig.3 some typical results obtained from triaxial tests using "representative" and "undisturbed" samples are shown; the last were recovered directly from test pits close to the towers. The triaxial tests were undrained, soils are the same, and the cell pressure was in each case 1 and kg/cm2; in this way it is easy to do the respective comparisons, and the following conclusions can be derived.

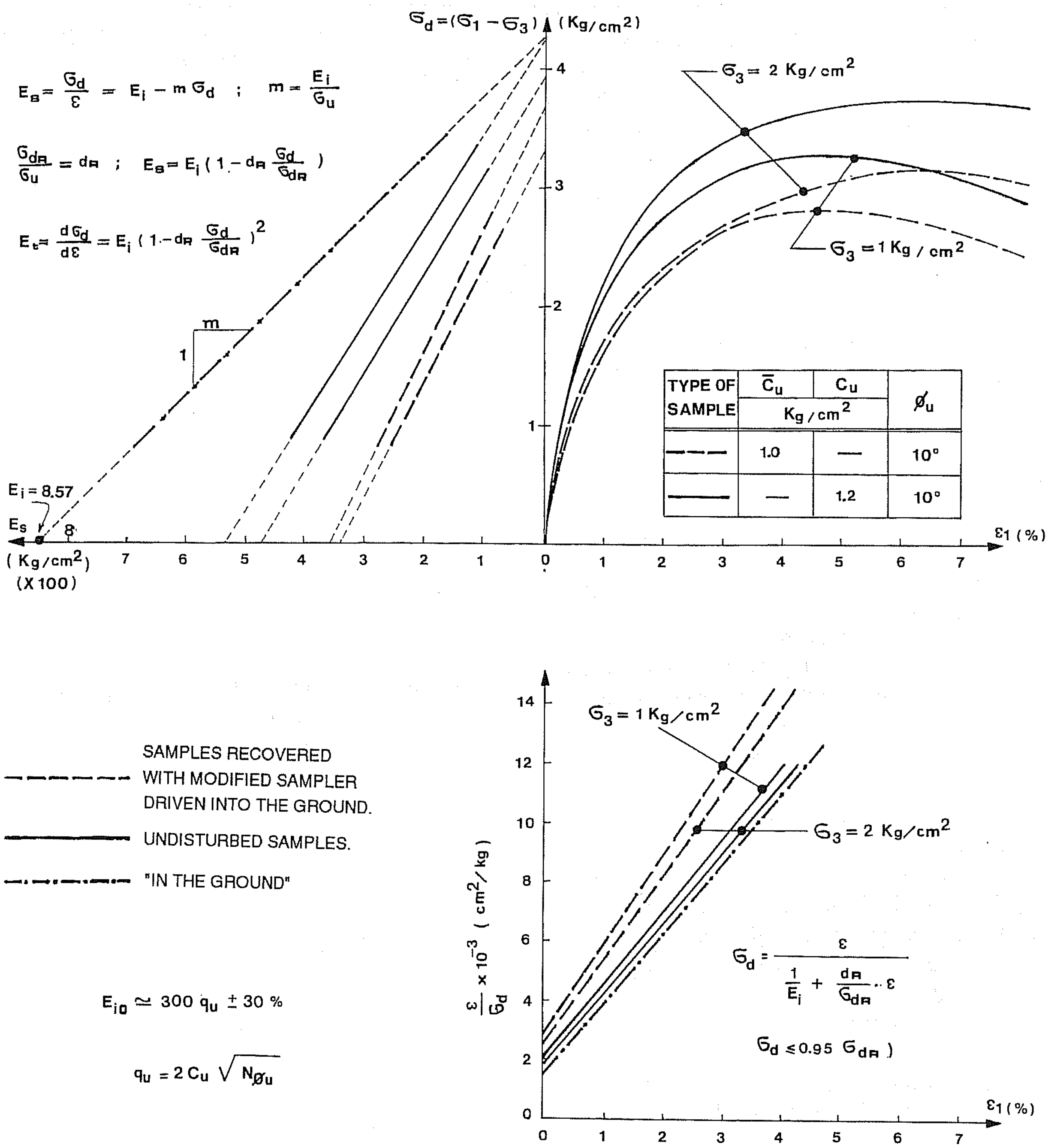

FIG.3 TRIAXIAL TESTS. LINEAR FITTING: SECANT MODULUS - STRESSES (SAMPLES FROM LEVEL "A")

a) With both type of samples the Cu and the ϕ_u values are consistent;

b) The ϕu values are the same; in this case, 10°;

c) The Cu value from "undisturbed" samples is greater than the values obtained from the "representative" samples - approximately 20 % - This difference shows clearly the damage of particles bonds produced by the dynamics effect of sampler driving;

d) The correlation between ϕ_u and the degree of saturation Sr (of the order of 80 %) appears to be good;

e) For both types of samples, the stress- strain results consistently show the linear relation between the secant modulus $E_S=\sigma_d/\varepsilon_1$ and the stress level $\sigma_d=(\sigma_1-\sigma_3)$, until a value near to $\sigma_{dR}=(\sigma_1-\sigma_3)_{max.}$ at failure. These relations permit to define the parabolic relation between the tangent modulus $E_t=d\sigma_d/d\varepsilon$ and the stress level, and the hyperbolic relation between the stresses σ_d and strains ε_1 as shown by Kodner.

f) Clearly appears the consequences of the damage in the particles bonds produced by sampling of the "representative" samples; this effect can be observed in the fall down of initial modulus Ei obtained from "undisturbed" and "representative" samples tested in the triaxial chamber.

g) Starting from correlations with other types of tests and taking into account some control messures performed in actual structures, studied with the common methods of back analysis, it is possible to conclude that in this soils and even for "undisturbed" samples, the only action of stress release, time and handling produce a drop of modulus Es, but this drop is not too much significative on the strength parameters.

h) As a consequence of this analysis, and for soils of "level A", the Øu values from "representative" samples, can be adopted as "ground values" while Cu values should be increased in about 20 % and Ei in at least 100 % . For very hard soil, strongly cemented, the difference between the previously given values should be as larger as the degree of soil cementation.

For soils similar to those shown at "level A", the available correlations point out that the initial undrained modulus is aproximately Ei≈300 qu±30 % being $q_u=2C_u.\sqrt{N_{\phi u}}$. If this expression is applied to the parameters previously included, we get Eiu ≈ 300 x 2 x 1.2 x 1.19 = 8,57 kg/cm^2, wich is a conservative value, and must be increased for larger confining pressures. For very high confining pressures, which lead to Øu ≈ 0 (Sr ≈100 %), the corresponding value of Cu should be adopted, to estimate the respective value of Eiu.

5. BEARING CAPACITY AND SETTLEMENTS EVALUATION

Fig.4 shows a tower section. The bearing capacity formulas where applied (9) using the strength parameters in terms of total and effective stress. For calculation, the following statistical values from test results were used:

LEVEL / PARAMETERS	UPPER PART (A)		MIDDLE PART (B)	
	UNDRAINED	DRAINED	UNDRAINED	DRAINED
COHESION (Tn/m2)	Cu=10	C´=5	Cu=15	C´=7,5
FRICTION (°)	Øu=10°	Ød=20°-25°	Øu=10°	Ød=20°-25°
MODULUS (Tn/m2)	Eiu=6000-7500		Eiu=12000-15000	

in all cases γ= 1.8 Ton/m^3

Brinch Hansen Formulas were used. The following loads were considered: the dead loads acting at the present time; the dead loads that will act with the completed towers; and the dead and live loads acting to the final stage of the structure. The calculations shown that the safety factor F for load acting at present time is not less than 2,5 to 3 , but for loads acting when the towers are completed, F drops to around 2, and even less than 2 when limit conditions of total actions over the towers are considered.

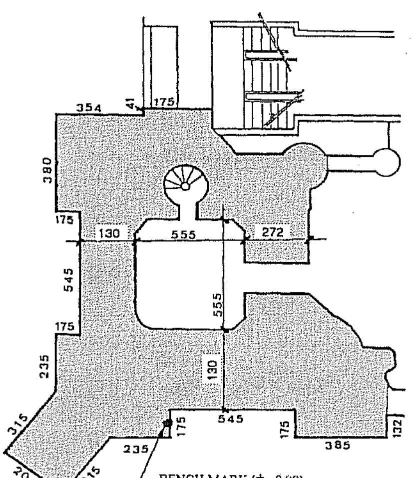

FIG.4 SECTION TOWER 53 (AT ELEVATION + 4.35)

For settlements calculations the initial modulus included in the table were used. The prediction indicates differential settlements values of approximately 2 cm ± 30 % (it must be considered the dificulties to modelize the interaction with the brickwalls of the towers). This value, and the possible additional rotations, ieads to the conclusion that some fissures would appears on the brickwalls. But the analysis also shown, that is very difficult to predict the actual picture frame of fissures.

Taking into account the uncertainty into the problem and the coefficients obtained in the calculations -with safety factor coefficients less than those generally accepted- the conclusion is: there are not geothecnical obstacles that prevent to complete the towers, but some underpinning works to enlarge the present foundation area, or to transfer loads to the deeper stiff strata of subsoil, should be necessary to be made previously.

6. PRELIMINARY ALTERNATIVES CONSIDERED

At the present time, two alternatives were analyzed:

6.1. Enlargement of current foundation area. (Fig.5)

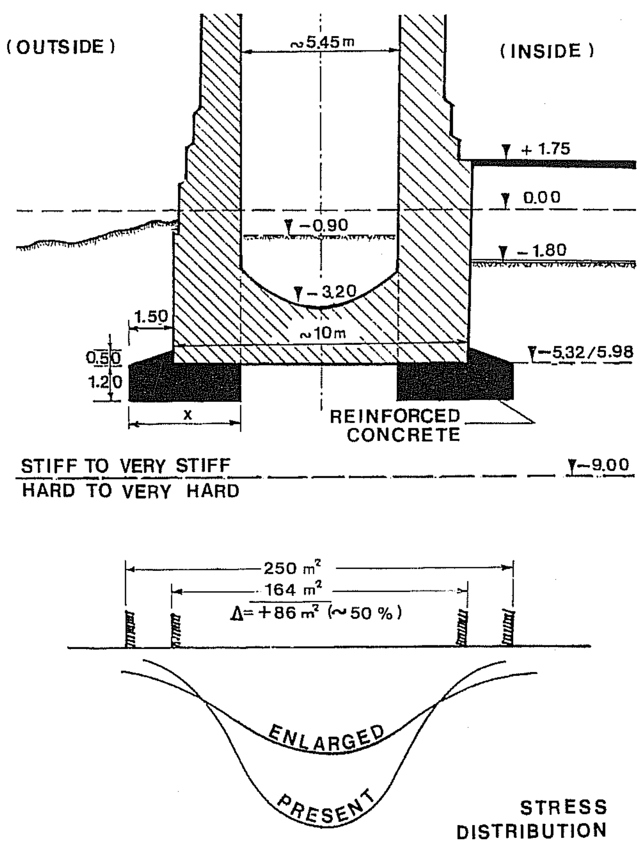

FIG.5 ENLARGEMENT OF FOUNDATION AREA

This is a conventional precautionary underpinning solution. In this case it is relatively easy to perform, by alternative sectors: a masive reinforced concrete ring is built under the brick foundation with a total area larger then the present one.

By this way it is possible to accomplish all the customary requirements regarded the allowable bearing capacity, but it does not clear the uncertainties related to the differential settlements of the other connecting parts of the brick structure.

6.2. Transfer the loads to a deeper more strength subsoil strata using piles.(Fig.6)

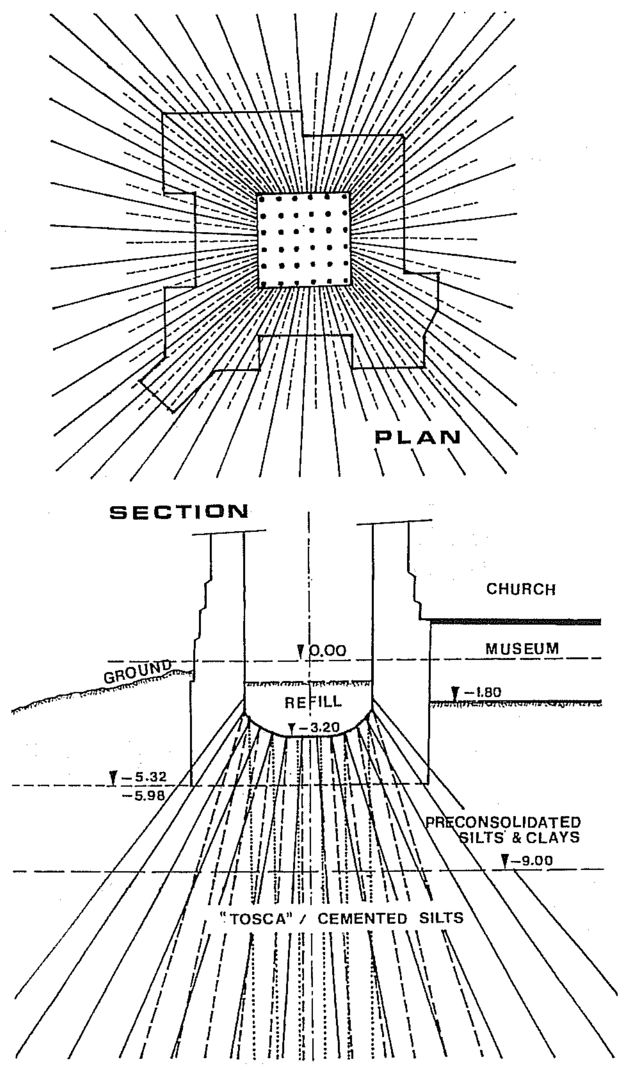

FIG.6 REINFORCED FOUNDATION WITH MICROPILES

This is a very feasible solution with the help of modern techniques available for micro piles performed with grout inyected under high pressures.

At the first approach it was considered—

among other solutions– to perform micro piles with 20 to 25 cm. of diameter reaching approximately -20 of elevation. The piles will be vertical and inclined, with an average length of 15 m. This micropiles can be used for normal service limited load about 30 Tn. each one, but this value can be increase 20 % / 30 %; for short or instantaneously actions. The bearing capacity along the shaft results 90/100 Tn.; the point capacity is about 30 Tn. These last values are valid for loads in the initial time of service life of pile, but the relief of stresses along time can diminish the values of resistance around 30 %; that is why it is not convenient to adopt service loads greater than those mencioned. The necessary number of piles is of the order of 100 ± 20 %, for each tower. The connection and juncture with the current brick foundation is easy to get and present no difficulties. The time of execution for this job could be estimated in 100 days and it is possible to complete the total work from the interior of each tower without any excavation and without any apparent change in the outward view of the Monument.

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