Long term settlements of tall buildings on sand

Les tassements à long terme des grands édifices dans le sable

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SUMMARY: The lower horizon of the São Paulo Tertiary Geological Basin consists of a loose to dense clayey sand deposit, on which many of the tall buildings of São Paulo are founded. A method to evaluate the settlement of buildings on sand is discussed, as compared to observed immediate settlement of some of these buildings. However almost 30 year after construction, the existance of a secondary compression of the sand layer has been shown. An investigation to determine the cause of such phenomenon, led to the conclusion that these foundation settlements were due to a semi-elastic immediate deformation followed by a long term viscous compression of the sand layer.

1 INTRODUCTION

The city of São Paulo, Brazil, lays on a Tertiary Geologic Formation mainly composed of two horizons: an upper clayey layer and a non-uniform lower sandy deposit, overlaying a gneissic rock substratum. Figure 1 shows a geologic cross section of the São Paulo Tertiary Basin. It shows a river valley that crosses the city in the East-West direction and separates two hilly areas rising to about elevation 810 m above sea level. The bottom of the valley is at elevation 730 and the sandy deposit upper surface is about elevation 740, coinciding with the natural ground water level.

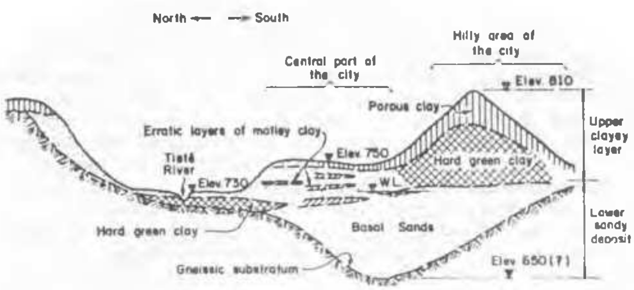

FIG. 1 - SKETCH OF A GEOLOGICAL SECTION OF SÃO PAULO TERTIARY BASIN

Many of the tall buildings in the city of São Paulo, are founded, by means of deep caisson or pile foundations, on this lower sandy deposit, locally called "basal sands". The settlements of some of these buildings have been observed since their construction about fourty year ago. The senior author presented a summary of the results of such observations in a contribution of the 5th I.C.S.M.F.E. held in Paris, in 1961. That paper will be refered to here as the previous paper (1).

The main purposed of the previous paper was to describe a method used by the author, to predict settlement of buildings founded on sands, based on the formula:

$$s_i = \int_0^\infty \frac{1}{E_z} (\sigma_z - \sigma_r)\, dz \qquad (1)$$

where: σ_z and σ_r are respectively, the vertical and radial pressures induced by the load of the building at depth z, on a vertical at centers of the foundation areas. When that area was retangular

$$\sigma_r = \frac{1}{2} (\sigma_x + \sigma_y)$$

and when, long and narrow $\sigma_r = \sigma_y$. E_z is the "modulos of compressibility" that for sands was assumed to increase linearly with lateral confining pressure.

$$E_z = \alpha\, (K_o \gamma z + \sigma_r) \qquad (2)$$

where: K_o is the coefficient of earth pressure at rest, γ is the natural unit weight of the soil and α is a dimensionless constant the value of which was estimated, by means of back analysis of settlement observations.

Combining (1) and (2):

$$s_i = \frac{1}{\alpha} \int_0^\infty \frac{\sigma_z - \sigma_r}{(K_o \gamma z - \sigma_r)}\, dz = \frac{A}{\alpha} \qquad (3)$$

A graphical method was used for computation of (3). First σ_z and σ_r were calculated by means of usual elastic methods and added to the lateral earth pressures at several dephts. Then the $(\sigma_z - \sigma_r)$ were divided by $(K_o \gamma z + \sigma_r)$ and the curve so obtained ploted on a graphic against depth. Finally this graph was integrated to obtain A, which was divided by α to obtain the settlements, at the center of the buildings.

The average value of α obtained in back analysis of the settlements observed in the tall buildings mentioned in the previous paper was 1427, with a maximum of 2175 and a minimum of 900. Less accurate observations done before, had led to admit an average value of 1000 for α. It can be observed from table 2 of the previous paper that the discrepancies of the α values from this average value refer to the cases of pile foundations which can be explained by the uncertainties of computation of applied pressures in such cases.

After presentation of the previous paper, the settlements, in at least the three of the tall buildings mentioned, continued to increase with time. This fact led to conclude that settlement of buildings in the São Paulo basal sands, develops in two phases: one immediate, due to an elastic deformation of the soil, and a secondary, deferred along time, probably due to viscous deformations of the clayey sands.

The settlement observations and computations mentioned in the previous paper refer to the immediate elastic phase. But, in the case of the Copan Building that, at the time of presentation of the previous paper, had only its structure completed, observation of settlement has continued regularly since completion of its construction in 1965, showing the appearance of secondary settlement phase. The São Paulo Institute of Technological Research then decided to try to measure present settlements of the Hotel Jaraguá and CBI-Esplanada Buildings in order to compare long time settlement of these three buildings among themselves. The purpose of this paper is thus do summarize results of such observations and their analysis.

2 GEOTECHNICAL PROPERTIES OF SÃO PAULO BASAL SANDS

The São Paulo basal sands are a granular material varying from coarse to fine sand with little clay content, as shown by granulometric curves in figure 2. Maximum grain sizes of these sands range from 0,4 to 3 mm and its percentage of fines (grains less than 0,1 mm diameter) vary from 20 to 60%. Their clay content varies from 7 to 30%. Notice that, in the granulometric curves, the silt content is almost absent. That is, basal sands are a mixture of pure sands and clay material, suggesting a double sedimentation process.

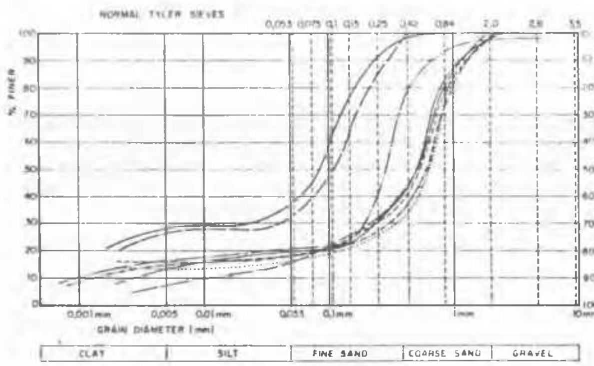

FIG 2 - GRANULOMETRIC DISTRIBUTION OF SÃO PAULO "BASAL SANDS"

Within this sandy deposit, erratic layers of clay may occur. Besides, as the water level coincides with the top of the substratum, the basal sand deposit is permanently submerged.

Density of these sands varies not only vertically but also horizontally, suggesting a more or less chaotic process of filling, of the Tertiary basin. Undisturbed samples taken from the bells of compressed air caissons, as well as by means of shelby-tube or Ivanoff samplers, have shown that the their average void-ratio and its standard deviation is $e = 0{,}65 \pm 0{,}12$.

Determination of maximum and minimum void-ratio of the basal sands, by several methods, permits to say that its maximum void-ratio is between 1,4 and 0,9 and the minimum between 0,4 and 0,5. So its relative densities would vary from about 0,5 to about 0,75. According to these results the basal sands compactness may vary from medium to dense; that is, its relative density is largely variable.

Figure 3 shows a boring profile, obtained at the Copan Building site, with graphs of dynamic standard penetration and static cone penetration resistances. It can be observed by these graphs that the average standard penetration resistance at this site is $N_{SPT} = 8$ with a maximum of 34 and a minimum of 3. Below pile points N_{SPT} averages 5. So basal sands at this site can be classified as loose. About the same situation was observed at the Jaraguá Hotel Building site, where the average $N_{SPT} = 11$, with a maximum of 34 a minimum of 3. On the contrary at the CBI-Esplanada Building, the average $N_{SPT} = 29$ with a maximum of 60 and a minimum of 5. So the basal sand at this last site can be considered as somewhat dense, however maintaining the same variability as in the other sites.

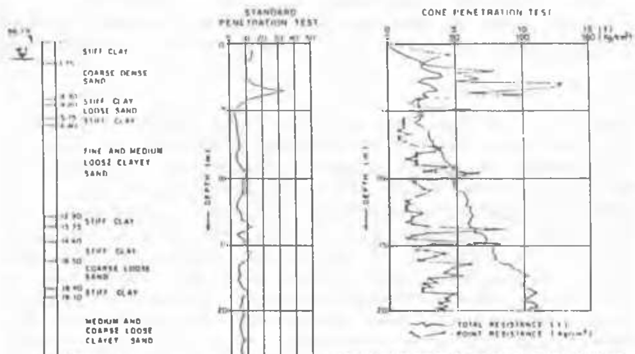

FIG 3 - SOIL PROFILE AND PENETRATION RESISTANCE BORING 1 - COPAN BUILDING

Attempts were done in order to determine $E_z = \alpha\sigma_3$ from laboratory triaxial compression tests on undisturbed and compacted samples of sands. To solve the question of at what strain should E_z be computed, a telescope bench mark was installed at the CBI-Esplanada Building, in order to measure differential settlement at several depths. Figure 4 shows a curves of accumulated deformations of the ground, versus depth observed at end of construction. From that curve it is possible to see that the average strain, within 5 meters below the foundation, is only 0,11%; at the next 5 meters, it is 0,2% and drops to 0,07%, from 10 to 15 m. It was then decided to take E_z as the tangent modulus in the triaxial compression test curves.

A few triaxial compression tests done on undisturbed samples have shown $E_z = 570\sigma_3$ for a sample with a void-ratio of $e = 0{,}78$; and $E_z = 880\sigma_3$ for void-ratio $e = 0{,}65$. Those values are lower than

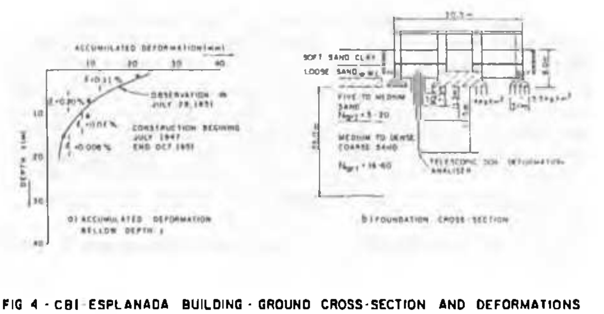

FIG 4 - CBI ESPLANADA BUILDING - GROUND CROSS-SECTION AND DEFORMATIONS the observed in the field; perhaps due to sample disturbance or uncertainties in the determination of tangent at origin of stress-strain curves.

Investigations that are being carryed on by Feliciani [2] on the geotechnical properties of São Paulo basal sands have shown values of α coefficient from: $\alpha = 1600$ for a compacted sample, with void-ratio $e = 0{,}45$, $\alpha = 470$ for $e = 0{,}588$ and $\alpha = 200$ for a compacted sample with void-ratio $e = 0{,}68$; even lower than those obtained with undisturbed samples.

In the oedometer tests, basal sands exhibit an almost instantaneous primary consolidation, followed by a secondary compression. The slopes of straight lines in semi-log strain-time curves are the coefficients of secondary compression C_α. Value of C_α varies with applied consolidation pressure as, for instance, in a compacted sample, with an initial void-ratio $e = 0{,}666$, $C_\alpha = 0{,}144\%$ for a $0{,}15\ kP_a$ consolidation pressure was obtained; $C_\alpha = 0{,}222\%$ for $250\ kP_a$ and $C_\alpha = 0{,}356\%$ for $1500\ kP_a$.

3 OBSERVED LONG TERM SETTLEMENTS OF THREE BUILDINGS

Copan Building, about 130 m high and covering an area of 2500 m^2, was designed to be founded on sand by means of compressed air caissons based at 10 m below street level. But the sand was considered too loose to support the proposed pressure of 1000 kP_a to be transmitted by the caissons to the ground. So it was decided to compact the upper coarse sand layer (see figure 3) by driving piles, with aid of followers, before the caissons were sunk. Figure 5 shows a plan view and a cross-section of the foundations thus accomplished.

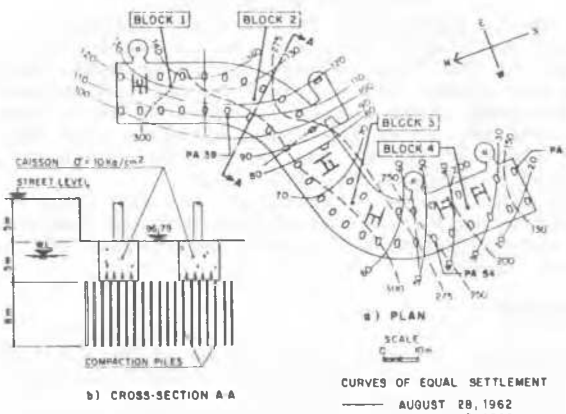

FIG. 5 - COPAN BUILDING: FOUNDATION PLAN AND CROSS-SECTION, SHOWING CURVES OF EQUAL SETTLEMENT AT END OF CONSTRUCTION (AUGS. 28, 1962) AND RECENTLY (MARCH 5, 1988)

Figure 6 shows observed maximum, average and minimum settlements versus time. The building maximum settlement, computed before construction of the building, was estimated in 150 mm. This calculation was based on an estimation of the α value by means of back-analysis of settlement of a viaduct founded on basal sands with a type of foundations similar to the Copan Building. At that time, such computation was considered too pessimistic. However by the curves in figure 6, it can be seen that 150 mm corresponds approximately to maximum semi-elastic immediate settlement at the end of construction, in Blocks 1 and 2. Settlement in Blocks 3 and 4 and smaller due to coarser and denser sand layer at their sites.

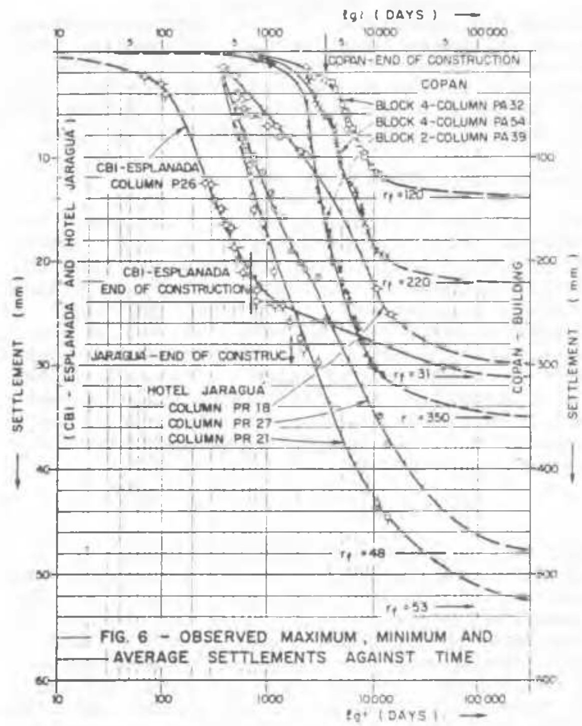

FIG. 6 - OBSERVED MAXIMUM, MINIMUM AND AVERAGE SETTLEMENT AGAINST TIME

It is worth to be mentioned that the East columns of Block 1 had to be underpinned during construction due to excessive settlement in regard to those of other columns, probably due to inefficiency of the pile compaction of the upper sand layer in this area. This job was described by José Machado at the 2nd Panamerican Soil Mechanics Conference [3].

Figure 6 shows a curve of settlement of the CBI-Esplanada Building column P 25. Maximum observed settlement, just at this column, was 26 mm. The computed immediate settlement, by the previously mentioned method, assuming $\alpha = 1000$, was 30 mm, which is not too far from the observed maximum. Recent observations (June 6, 1988) have shown a long term settlement of 30,2 mm, at column P 26.

In figure 6 settlement versus time curves for the Hotel Jaraguá Building can also be found. Computed semi-elastic settlement at end of construction by the above mentioned method, assuming, $\alpha = 1000$, was 48 mm, and the observed settlement at center of foundation area, 29 mm. Recent measurement of settlement (May 9, 1988) shows 47,4 mm maximum settlement at center of foundation area, about 35 years after construction.

Foundation of the Hotel Jaraguá Building was done by means of 295 cast-in-place piles, transfering to the soil a load of 80 tons each pile, covering an almost circular area of 700 m^2. The piles are supposed to transmit to the soil a vertical pressure averaging 250 kP_a from 20 m to 60 m below basement level.

As can be seen, by the settlement curves in figure 6, the long term settlement of CBI-Esplanada and Hotel Jaraguá buildings are quite modest as compared to the maximum long term settlement of Copan Building. However the first two cases confirm the existence of long term deformations of the São Paulo basal sands. Obviously the difference of magnitude between the amount of settlement in the three cases are due to difference in the applied pressures and relative densities of the sand layers at the three sites.

4 THEORETICAL RESULTS OF ANALYSIS

Taking into consideration that the rate of settlement must decrease toward an end, otherwise it would lead to the conclusion that the specific deformation difference $\Delta h/h$ would attain an infinite value and, therefore, the height of the sample would tend to zero, which is an absurd, Gibbson and Lo [4] presented a theory of secondary compression, based on viscous deformation. According to this theory, in the cases where primary compression is immediate, the compression strain would be expressed by the following equation as function of time.

$$\varepsilon\,(t) = \Delta\sigma \left[a + b\,(1 - e)^{\frac{-\lambda t}{b}} \right] \qquad (4)$$

where a is a primary compression index; b, a secondary compressibility coefficient, $1/\lambda$ the viscosity of the soil structures; and $\Delta\sigma$ the increase of pressure applied to the sample.

The case of the basal sands is not a case of consolidation but the ideas of Gibbson and Lo can be applied in view that the semi-elastic deformation of those sands are almost immediate and the time deferred deformations are,possibly, of a viscous nature.

So the rheological model of a spring in series with a couple of a spring and a dash-pot, can be applied to the establishement of strain relation of basal sands. This model, reffering to the compressibility of a layer of thickness dz, at a depth z; would lead to the following expression:

$$\frac{ds}{dz} = \frac{\Delta\sigma}{E_z} + \frac{\Delta\sigma}{F}\,(1 - e^{-\lambda F.t}) \qquad (5)$$

where E_z is the "modulus of compressibility" of soil due to the semi-elastic deformation, F and λ are respectively the coefficient of compressibility and of viscosity, of the soil structures. Equation (5) is valid only for $t > t_o$ (after completion of the semi-elastic settlement at end of construction).

The settlement of the foundation on sand at a certain time t, is:

$$s_t = \int_0^\infty \frac{\Delta\sigma}{E_z}\,dz + \int_0^\infty \frac{\Delta\sigma}{F}\,dz\,(1 - e^{-\lambda F.t}) \qquad (6)$$

The first integral is the immediate settlement s_i and the second contains, the final secular viscous settlement s_v. The expression (6) can be put in the following form, if λ and F coefficients are assumed to be independent of depht.

$$s_t = s_i + s_v\,(1 - e^{-\lambda F.t}) \qquad (7)$$

where s_t is the settlement at time t; s_i is the immediate settlement at the end of construction; and s_v is the settlement due to the viscous deformation. The sum $s_i + sv = s_f$ is the final secular settlement.

Rearranging equation (7) and taking the logarithm of both sides:

$$\ell g_{10}\,(s_f - s_t) = \ell g\, s_v - 0{,}43\,\lambda F.t \qquad (8)$$

Figure 7 shows semi-log plots of maximum settlements of the three buildings in question. In spite of the few points it confirms equation (8) (valid only for time after end of construction). So the observed long term settlements were due to viscous deformations.

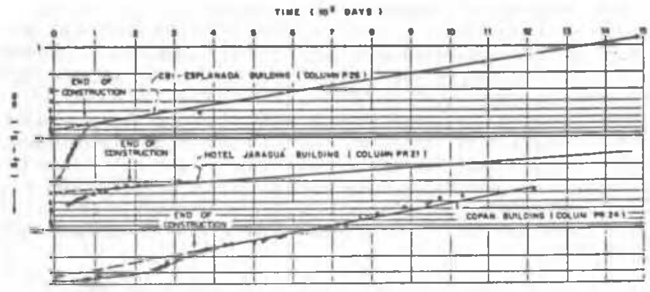

FIG 7 - SEMI-LOG PLOT OF SETTLEMENTS DUE TO VISCOUS DEFORMATION

5 CONCLUSION AND ACKNOWLEDGEMENT

From the above mentioned observations and analysis it is possible to conclude that the settlement of tall buildings founded on the São Paulo basal sand deposit are due to a semi-elastic immediate deformation followed by time-dependent long-term settlements of a viscous nature.

The viscous deformation of CBI-Esplanada was the lower, because in its site the sand was denser and the average applied pressure was lower. At the Hotel Jaraguá Building site, the sand was looser but the average applied pressure in the sand layer was about the same as in the CBI (250 kP_a). At the Copan Building site the sand layer is looser and the average pressure applied is much higher (700 kP_a).

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