BEHAVIOUR OF FOUNDATIONS IN THE REDDISH SOILS OF L. MARQUES

COMPORTEMENT DES FONDATIONS DES SOLS ROUGES DE L. MARQUES
ПОВЕДЕНИЕ ФУНДАМЕНТОВ, СООРУЖЕННЫХ В ЛАТЕРИТАХ Л. МАРКЕС

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SYNOPSIS. The origin and nature of the weak soils with unstable structure(loess type) of Lourenço Marques area is described. Physical and mechanical characteristics of those soils and results of field tests (dutch) cone, S.P.T.,tests on plates and on piles)are given.The actual settlements of buildings have been measured and from their rotations and curvatures have been calculated. Also the most frequent compressibility of the soil has been calculated and compared with that obtained from laboratory tests.The different behaviour of driven and bored piles is refered and the relative magnitude of the settlements in shallow and piled foundations is presented.

1. INTRODUCTION

A lot of experience and test results has been collected in connection with the behaviour of foundations in the reddish fine graded soils of Lourenço Marques.Granulometric analysis, moisture content,bulk density, permeability, compressibility, shear strength and other laboratory tests have been performed for more than ten years and also field tests e.g., dutch cone, standard penetration, plate. The settlement of buildings with shallow and deep foundations have been measured. A full account of so many results cannot be given in the limited space avaible for this paper. However relevant features are presented. The results refered in the paper have received statistical treatment and values most frequent and the interquartile ranges are given.

2. ORIGIN AND NATURE OF THE SOILS

The soils are quartzitic red sands, fine to medium, monogranular with round grains. They have 16 ± 3,6% of silt and clay (d<0,076 mm), the latter not exceding 3%, and only 2,9 ± 1,5% of coarse material (d 0,599 mm). They form old dunes, 30 to 50 meters high deposited in the Quarternary (Superior Plistocenic and Epiplistocenic). A full account of the geological aspects of these soils is given by M.J. Macedo (1971). The climates may be considered as semi-arid, megathermic, with a null excess of water (Silva Soares, 1964).

3. PHYSICAL AND MECHANICAL CHARACTERISTICS IN LAB.

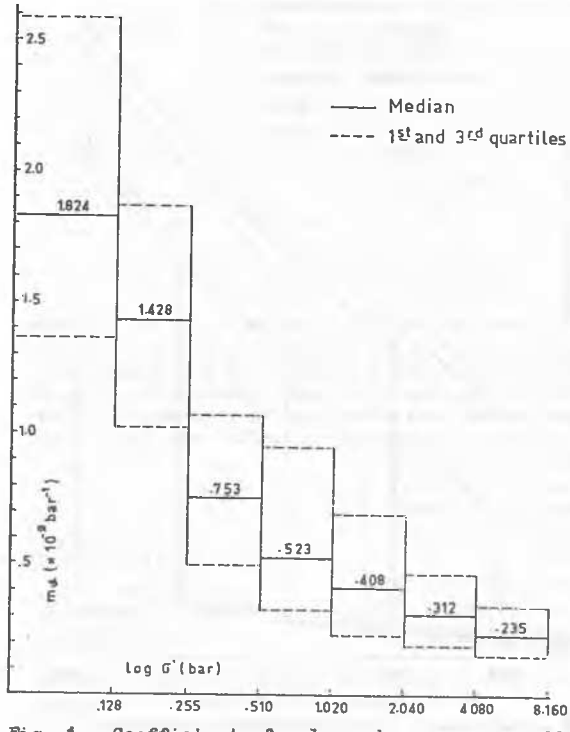

Fig. 1 – Coefficient of volume change versus effective normal stress.

The following table gives the physical properties of the soil.

	Most frequent	interquartile range
Specific gravity	2,65	2,62 to 2,65
dry density	1,61	1,65 to 1,56
moisture content	5,25%	6,04 to 4,23

A correlation between void ratio e and tan ϕ' has been attempted according to the relationship $e \times \tan \phi' = A$. The most frequent value for the shear box tests has been A = 0,43 with a interquartile range of 0,40 to 0,47.
The average values of the permeability measured in the laboratory is $2,25 \times 10^{-3}$ cm/s. The volumetric compressibility m_v at natural water contents decreases rapidly with effective pressure , mainly in the interval = 0,128 to = 0,225 bars (Fig 1).
Oedometer tests have reveal instability of soil structure when water is added to the loaded soil. Fig. 2 shows drops in settlement when water is added to the samples for the usual stages of loading of 0,128, 0,255, 0,510, 1,020, 2,040 bars. The maximum ratio between the settlement after addition of water to the dry settlement (18,4) was recorded for the pressure of 0,255 bars.
However, in what concerns practical aspects of foundations, no important problems have arrived from such instability of the soil structure.

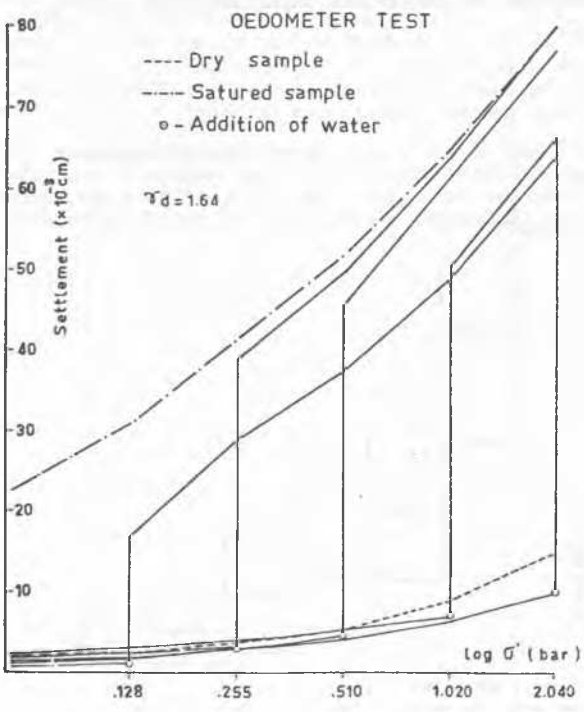

Fig. 2 – Settlement versus effective pressure.

4. PHYSICAL AND MECHANICAL CHARACTERISTICS IN SITU.

The dutch cone point resistance have been measured on 91 points and the results are showed in Fig. 3

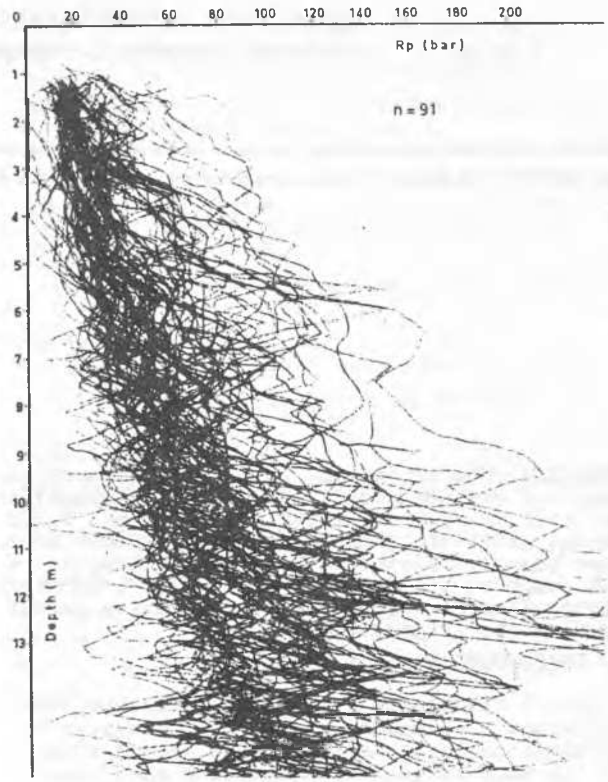

Fig. 3 – Dutch cone point resistance versus depth.

The ratio between the point resistence R_p and the number N_b of blows for one foot penetration in the Standard Penetration Test has been calculated and treated statistically. In 180 pairs of tests the most frequent value obtained is K = 5,24 and the interquartile range is 6,96 to 4,00. Linear regression and parabolic regression have been performed between Rp and N. The parabolic regression fits better the recorded results.

Tests with circular plates of diameters 0,30 m 0,60 m and 0,75 m have been performed in a few sites. It appears that in what concerns early settlements, these increase with plate size for the same average pressure. However the limiting pressures decreased with increasing size in the case shown in fig. 4. Tests have been carried according to ASTM Standards.

5. BEHAVIOUR OF SHALLOW FOUNDATIONS.

Most of the foundations of buildings up to ten floors in the town are on footings, either isolated or strip. Footings of the same building may have large differences in size to bear the same pressure.

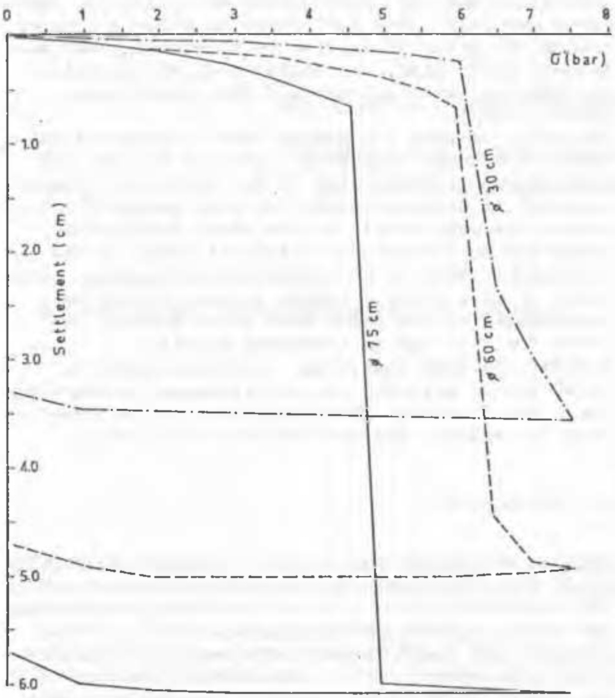

Fig. 4 – Circular plate tests.

However in the settlements recorded in 9 buildings (about 200 columns) variations are very large (between 1.1 and 8.8 cm) and the statistical treatment of the data did not reveal a "most frequent" settlement.

The maximum values registered were:

Differential settlement (between a column and the next)	3,7 cm
Ratio of the settlement to the span	$\tan\theta \cong \theta = 3,2 \times 10^{-3}$ radians
Curvature (ratio of the increment of to the average span)	$\frac{1}{R} = 1,0 \times 10^{-3}$ radians permeter

These values were taken immediatly after the buildings have been finished. As time goes on there are still important increasing of the total settlements. In one building after five years, the total settlements increased by a maximum amount of 66%. On the other hand the variation of the differencial settlements with time is quite irregular. The rate of settlement of a column is cleary influenced by the rate of settlement of its neighbours. If these tend to settle faster the rate of settlement of the column also increases. In one case, twenty months, after the construction ended the differencial settlements increased by an amount variable between 66 and 228%.

In what concerns the maximum recorded ratio of the differencial settlement to the span, $\tan\theta$, is greater than the maximum allowed in some Code of Practice e.g., U.S.S.R. Code (0,002). Nevertheless there are no sings of cracking of fissuring. Authors understand that $\tan\theta$ is not the only critical parameter in what concerns stresses induced in the structure. These are related to curvature more than to the differential settlment i.e., to $\tan\theta$. In fact, if the building tilts like a rigid body the differential settlements may be very large and no stresses induced in the structure. In one case a row of footings has rotated by an amount of 3×10^{-3} radians. But, since it was a nearly rigid body rotation there was no fissuring. In another case greater rotation caused no stresses but created difficulties in the verticality of the tracks of the lifts.

In what concerns curvature in one building observed during five years large oscilations were recorded after the end of the construction. This may be due to seasonal variations of temperatures and moisture content in the soil, giving rise to variations in the interaction of soil-structure as time goes on.

6. COMPRESSIBILITY AS OBTAINED FROM SETTLEMENT MEASURES AND OEDOMETER TESTS.

In order to bring statistical into the measurements either in field and in laboratory, compariosons have been made between the field compressibility C as obtain from measured settlements according to the usual formula (e.g. Jumikis, 1962)

$$\Delta = 0,79 \frac{p}{C} w A \qquad (1)$$

where
Δ = Settlement of a rigid footing
p = average pressure
w = shape coefficient
A = area of the footing
$C = E/(1-\nu^2)$
E = young modulus
ν = Poisson's ratio,

and

$$C_{(lab)} = \frac{E}{1-\nu^2}$$

with $$E = \frac{1}{m_v} \times \frac{(1+\nu)(1-2\nu)}{1-\nu} \qquad (2)$$

where m_v is the Terzaghi's volume compressibility as measured in oedometer tests (Giroud, 1969). The value taken for the Poisson's ratio was $\nu = 0,3$. Formula (2) already takes into account the null lateral deformation of the oedometer. Under these conditions the following values were obtained

	C_{field} (bars)	C_{lab} (bars)
The most frequent value	195	204
Interquartile range	183 to 249	122 to 368

The number of laboratory tests was 44 and number of points with measured settlement was 96.

Although the most frequent C values obtained from measured settlements and from laboratory tests are not much different, conditions in the laboratory are different from those in the field, although the samples have been tested at "natural" water content and for the same range of pressure (1 to 2 bars). In fact it has been noticed that between 2 and 24 hours after the beggining of the lab. tests there was no significant settlement of the samples, and, in contrast the settlements in the field go on for years. Surely formula (1) and (2) assume soil an elastic body and therefore should only be used for instantaneous loading and corresponding settlement. These conditions are impracticable in the field. Also, formula (1) does not take into account the depth of the foundation and stresses induced under a footing by its neighbours.

7. BEHAVIOUR OF PILES AND PILED FOUNDATIONS.

Four driven piles, two bored with water and one bored without water, have been tested.

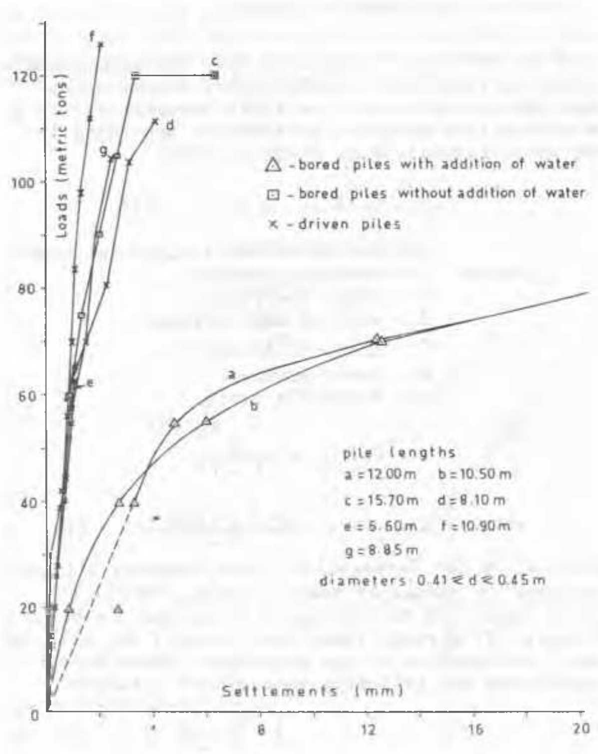

Fig. 5 - Pile tests.

Tests results are given in Fig. 5. It is clear from these results that piles bored with water have a much lower bearing capacity and a much larger settle ment for the working load (60 ton), than those driven or bored without water. However the settlements of the buildings founded on those bored piles were not significantly greater than those recorded for the other piles. This may be due to the fact that the piles are tested a few weeks after its concreting and the construction of a building takes about one year, time sufficient to allow the water around the piles to migrate to surrounding non satureted soil. Also, the caping slab of the piles may take an important part of the overal load (Vésic, 1969).

The ratio between the median total settlement measured in columns founded on isolated footing (34 milimeters) to the median of the total settlements measured in columns founded on pile groups (5 milimeters) is very large. In the cases dealt with there was no significant effect of group in the settlement. This is understandable for driven piles since if in a group a larger volume of soil is compressed, on the other hand after driving the piles the soil has an increased density.

However the same reasoning does not apply to bored piles, possibly due to piles-cap interaction, i.e., the piles cap slab plays also an important role in reducing the settlement of the group.

8. CONCLUSIONS.

Reddish soil in Lourenço Marques has a very regular grain size distribuition. However its mechanical characteristics have a large scattering. Therefore any study of those characteristics and its relations to the field measurements acquires significance only when there is statistical treatment of the data. It is authors believe that this is also the case of most experimental data in Soil Mechanics.

Settlement of pile groups is much less than antecipated from load test on a single pile, possibly due to pile-cap interaction which should be carefully investigated.

To obtain safe conclusions there is a strong need to construct reliable and cheep devices to measure the actual load at any moment in the columns of a building, in order to obtain the load settlement curves not divorced from reality.

9. AKNOWLADGMENT.

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