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STAGE GROUTING PRELOADING OF LARGE PILES ON SAND

CHARGEMENT PRELIMINAIRE DES PIEUX CAISSONS EN MILIEUX SABLEUX PAR L'INJECTION EN ETAGES

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SYNOPSIS. The construction of two bridges across the Parana River has furnished opportunity to gather information on the behaviour of deep large piles on sands. The first part of this paper deals with the preloading by stage grouting technique. Then results of load tests are analyzed. Piles with Freyssinet's cells on the head and in the base have been tested without preloading, then unloaded, preloaded and tested again. A preliminary analysis of data from tests includes the determination of the average value of -inverse of the tangent modulus- of the sand below the pile base both before and after preloading, which clearly shows the disturbances created by the lowering of the shall and the stiffness developed by preloading by stage grouting. Theoretical estimated settlements of large pile supported footings are being checked by in situ measurements.

INTRODUCTION

Precompression of the sand below the base of large diameter piles by means of pressure grouting has been reported several times in civil engineering literature. [1] [2] [3]. Whoever has had experience with this technique realizes very soon its decisive importance on the behaviour of large diameter piles on sand. Due to the large loads which these piles must carry, it is mandatory that each of them perform safely. Preloading by stage grouting has two beneficial effects 1) An overall improvement of soil strength and shaft contact with the pile as a result of grout filling of macro and space voids originated by the construction procedure plus a strengthening of the soil below the pile tip by grout penetration of weak spots. 2) A precompression of the soil below the pile tip, which considerably decreases the settlement necessary to develop a certain amount of point load.

The construction of two bridges across the Parana River requires that several hundred piles, ranging in diameter between 1 and 2 meters with lengths up to 75 m, be installed and that every pile be preloaded. A large part of them is in place already and several load tests have been performed. As a routine, the same pile is tested before and after preloading and many have been equipped with Feyssinet´s cells both on top and bottom to measure the development of base and shaft resistance when the test load is applied.

This paper introduces and explains the preloading by stage grouting technique and presents the results of some pile test in which its beneficial effects are clearly brought forward.

THE PRELOADING BY STAGE GROUTING TECHNIQUE

All large diameter piles in water were constructed sinking an outer open-end shell of either reinforced concrete or steel, which was lowered into the river bed to foundation level by boring through either by means of the reverse circulation drilling method or by means of bucket augers. Some contractors use drilling mud, and others prefer clean water. Depending on soil conditions, piles located inland are provided with a steel shell or drilled without casing. Drilling mud is always employed in the latter case.

The preloading by stage grouting technique has been applied successfully in all cases though no definite conclusions are yet available when the shell is lowered by bucket augering. Fig. 1 shows the preloading cell developed for this purpose. It is composed of a pressure cell with some 40 grout holes covered with a rubber sheet with identical number of holes, which alternate in position to avoid grout rebound. Attached to the cell and hanging from it, a basket full of coarse gravel is placed, which is intended to serve both as a grouting and pressure distribution chamber.

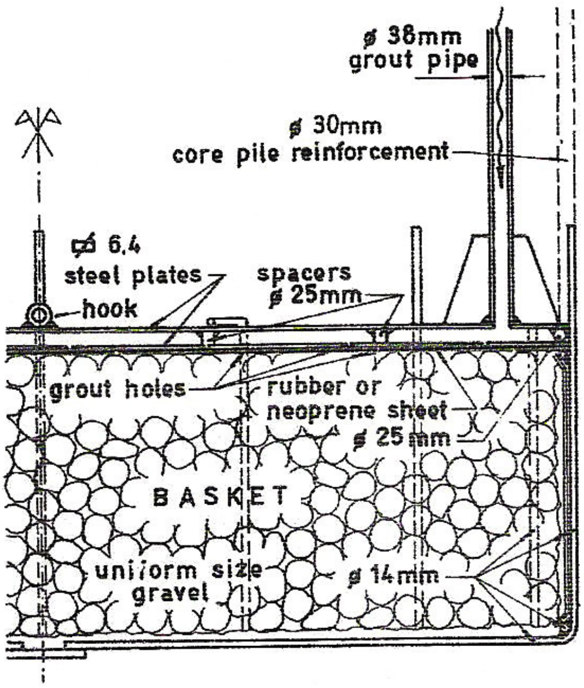

FIG. 1. PRELOADING CELL

The cell is welded to the bottom of the pile core reinforcement. Piles in water are heavily reinforced because they are designed as free standing columns fixed at the pile cap and in the sand formation. Buckling determines the diameter of the pile and the reinforcement. Length of embedment below assumed erosion line is usually established to assure the fixity of the pile. After the outer shell has reached the design depth and all the checks and corrections have been made, the core steel reinforcement is lowered with the preloading cell welded to its bottom end. The cell is usually detailed as shown on Fig. 1. When the pile is intended to be load tested, Freyssinet's cells are placed immediately above the preloading cell. These Freyssinet's cells are arranged in a unit with connections carefully prepared to adjust to the relative displacements that filling the pile with tremie concrete may originate.

After the core concrete has set, all grouting pipes, valves, filters, etc, are thoroughly washed with water under pressure. Regular grouting equipment, with pumps developing pressures up to 10000 kN/m^2 is being used. Grout mix is generally 1.5 parts portland cement and 1 part water, by weight. Each quadrant of the preloading cell has one grout pipe going up to the upper end of the pile.

Grouting starts by pumping clean water through one of the grout pipes with the upper ends of the other three closed or connected to pressure gages. Between 500 and 1500 kg of Portland cement is then grouted, depending on the diameter of the pile. This initial quantity must be enough to fill the voids in the basket of coarse gravel below the preloading cell plus a certain surplus and it is generally fixed on the basis of previous experiences. Grouting proceeds uninterruptedly as long as pressure gage readings are of the order of 5000 kN/m^2 and increasing. It concludes when a pressure of 10000 kN/m^2 can be maintained during 5 minutes or the pile rises 2 cm. This seldom happens.

Most frequently the 5000 kN/m^2 gaps pressure readings are not reached in the first stage. Grouting is then stopped and clean water is pumped until it comes out clean from the three grout pipes that were closed while pumping the mix. After a minimum elapsed time of approximately 12 hours another batch of portland cement is grouted. If necessary new grouting stages with the same amount of portland cement are repeated until the pressure of 10000 kN/m^2 is maintained during 5 minutes or the pile rises 2 cm. For piles 2,00 a in diameter 1000 kg of portland cement has been used for each intermediate stage.

TABLE I

Typical Grout Takes

Pile Diameter m	Average number of stages	Grout take Kg of portland cement
1.00 (SS, RC)	2	500
1.20 (SS, RC)	3	700
1.80 (RCS, RC, LG)*	3	3500
2.00 (WS, BA)	6	6000
2.00 (SS, BA)*	Still experimental	

SS: Steel Shell. RCS: Reinforced concrete shell. WS: Without Shell.
RC: Reverse circulation. BA: Bucket augering with drilling mud.
LG: Lateral grouting t increase skin friction of pile shaft.
* Piles in the river course.

The reaction to the grout pressure is furnished by the skin friction, which increases somehow in successive grouting operations. When it fails to give the required reaction, normally because of send disturbances while augering or lowering the outer shell, or because of the soil formation, preloading proceeds after building the pile cap. Experiences are under way aiming at a substantial increase of the skin friction by successive grouting from the preloading cell. Typical grout takes are given in table I.

Fig, 2 shows the stage grouting of a 2 m. diameter pile. The pile is embedded 5 m in sand. Boring was performed with bucket augers without casing using drilling mud. Seven stages were required to develop the preestablished 10000 kN/m^2 pressure.

It can be seen that it is relatively easy to reach the highest values once pressures start to build up in the latest stages. For this reason, it is highly convenient to preload as high as the equipment and the skin friction permit and repeat the operation, when necessary, after construction of the pile cap.

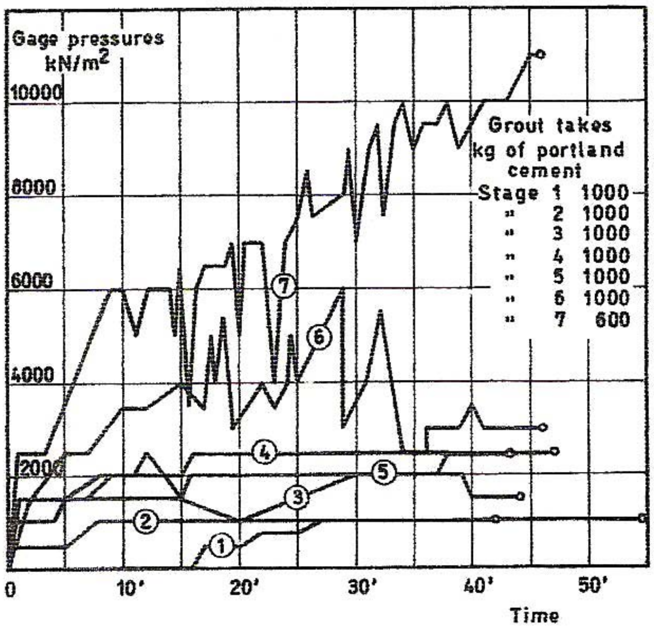

FIG. 2. STAGE GROUTING

LOAD TESTS

Testing piles up to 20000 kN is a major undertaking even for the big construction firms that participate in large foundation works. In the middle of a mighty river it is also dangerous planned. Inasmuch as preloading grouting improves the deformation characteristics of the sand below the pile base and also serves to appraise the development of skin friction along its shaft, it constitutes in itself a test. In fact, this is perhaps its most important function since it eliminates the risk of failure because of insufficient soil resistance. These considerations led to the following load testing procedure:

1.- A pile in a pile cluster is selected for load testing during the design stage.

2.- The pile cluster and the pile cap are designed to support the test pile reaction. Since both the piles and the pile caps are usually heavily reinforced this requires relatively minor alterations.

3.- The test pile is not concreted into the pile cap but adequately guided to control buckling and allow for the vertical movements which are required for testing purposes. Freyssinet's cells are located on top of the pile to center the test applied load correctly.

4.- First a load test without preloading by pressure grouting is performed (Before preloading load test).

5.- The pile is preloaded by pressure grouting.

6.- Then a new load test is performed (After preloading load test).

After removal of all measuring apparatus the pile is concreted into the Pile cap.

In this manner it is possible to measure both the weakening of the sand formation due to boring while lowering the shell and its recovery and improvement after preloading. To illustrate how load test can be used to evaluate these affects two cases are compared under Analysis of Results. Both 1.8 m. outside diameter piles, N°1 and N°3, have outer shells of reinforced concrete, which have been lowered by reverse circulation augering without using drilling muds. Skin friction had been improved before preloading by lateral grouting. The length of embedment in sand of pile N°1 is 2111 m. and of pile N° 3 is 21.6 m. The unsupported column lengths, Fig. 3) are respectively 27.1 and 25.5 m.

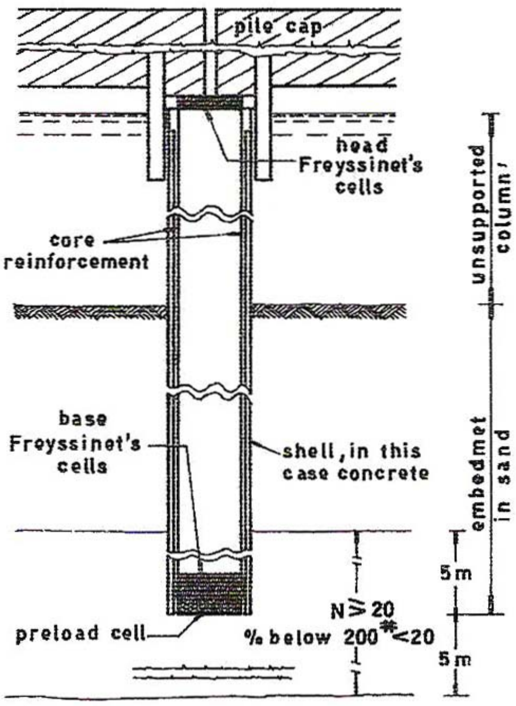

FIG.3. TEST PILE

In every other aspect there is the same similitude, except that while preloading pile N°1 it was possible to raise the grout pressure to the required 10000 kN/m^2 whereas only approximately 5000 kN/m^2 were reached under pile N° 3. The raising of the pile head at the end of the grouting stages to reach the pressures indicated above matches the differences in maximum grouting pressures. Pile N° 1 raised 16.2 between the unloaded condition at the end of the before preloading load test and the start of the after preloading load test, whereas pile N° 3 raised 52.9 mm.

ANALYSIS OF RESULTS

Fig. 4 shows the results of load test N° 3. Cycling is routine in all tests. The ordinate, s_h, is the pile head settlement. What follows is a step by step analysis of this test. When pertinent, results of load test N° 1 are introduced for comparison.

The calculated elastic deformation of the pile given by the line OA. Consequently it is also taken as the reference line for the pile tip settlement-applied load diagram. The primary loading curve is OB. The line OC gives the settlement curve after preloading.

There are important variations among tests on different piles, which are generally caused by the alteration of the sand properties awhile lowering the outer shell.

Fig. 5 shows results of two tests. The sand around and below test pile N° 3 suffered major disturbances. Test pile N° 1 was built following normal construction procedures. For both tests the dashed lines show results before preloading and the heavy solid lines after preloading. The thin solid lines are the calculated elastic deformations of the piles.

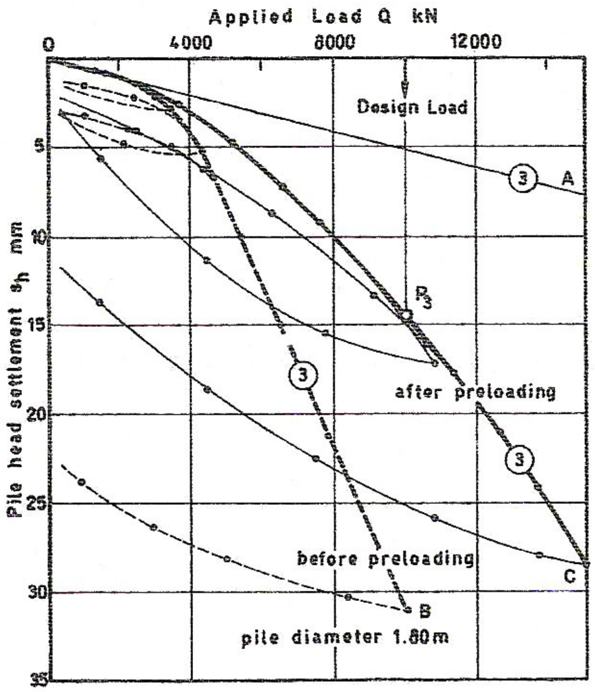

FIG. 4. LOAD TEST N° 3

By means of the Freyssinet's cells readings at pile head and at pile base it is possible to determine which part of the lead is carried by the base resistance and which by shaft resistance.

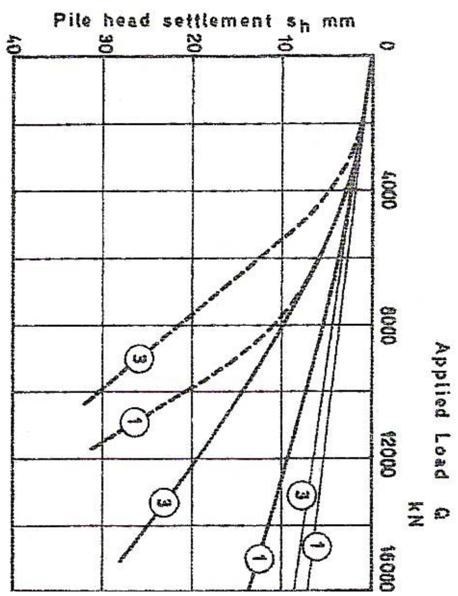

FIG. 5 LOAD TEST N° 1 AND 3

In Fig. 6 (a) curves a) to d) have the following meaning:

a) Settlement of the pile tip against applied load before preloading.

b) Same as a) after preloading.

c) Base resistance before preloading against settlement of the pile tip,

d = a) – c), shaft resistance as a function of settlement of the pile tip before preloading,

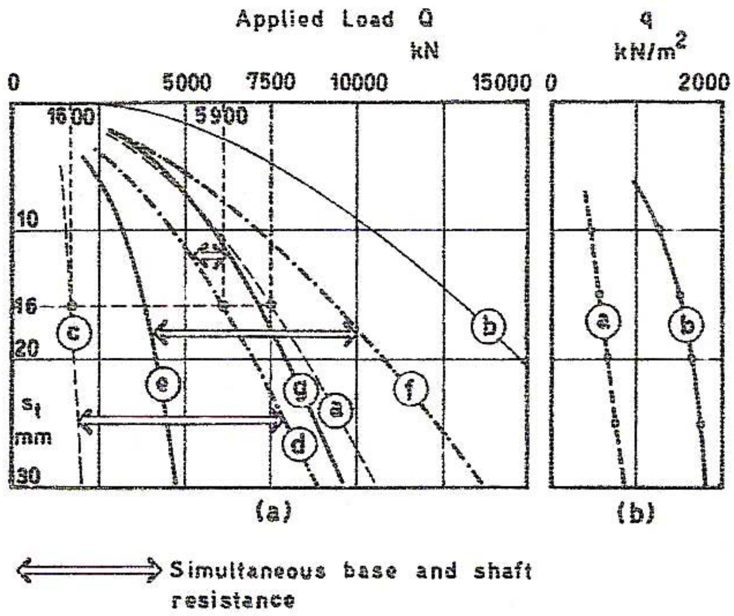

FIG. 6. APPLIED LOAD AGAINST PILE TIP SETTLEMENT DIAGRAM

Curves c) and d) show the base and shaft resistances while the pile tip settles under the applied loads shown on curve a). For example, under 7500 kN applied load, the tip settles 16 mm the point resistance is 1600 kN and the shaft resistance 5900 kN.

Loads on base were determined from load cell readings. Curve a) in Fig. 6 (b) shows base Freyssinet's cell readings against settlements of the pile tip before preloading and curve b) same as a) after preloading.

Preloading by grouting stiffens the sand and may probable create some kind of a mortar bulb which somehow enlarges the pile base. It also increases the skin friction. Therefore, in spite of having the Freyssinet's cells placed at the base, assumptions must be introduced to estimate how the load might be divided between base and shaft resistance after preloading.

If no mortar bulb is formed during preloading, the base resistance is determined multiplying the base pressure cell readings, Fig. 6 (b) b) by the pile base area. Curves e) and f), Fig. 6 (a); represent this Case:

a) Base resistance after preloading against settlement of the pile tip.

f = b) - e), Shaft resistance after preloading against settlement of the pile tip.

Therefore, equilibrium against applied load requires that a significant increase of the skin friction should be taken into account.

If the assumption that skin friction remains invariable is introduced, a mortar bulb must provide an enlarged base to carry the unbalanced load.
Equilibrium requires what the bottom diameter be increased to approximately 2.20 m. For this latter diameter curve g) represents:

g) Base resistance after preload against settlement of the pile tip,

The correct answer must be located between these two extremes, but at present there is not enough evidence to intent an estimate about this problem. There are, however, some indications of a possible improvement in skin friction, possibly in the lower part of the pile shaft.

CHANGE IN DEFORMATION CHARACTERISTICS OF SAND BY PRELOADING

Fig. 7 shows the elements that enter into a simple equation to compute settlements below individual piles, which can be applied for the preliminary analysis of date from tests.

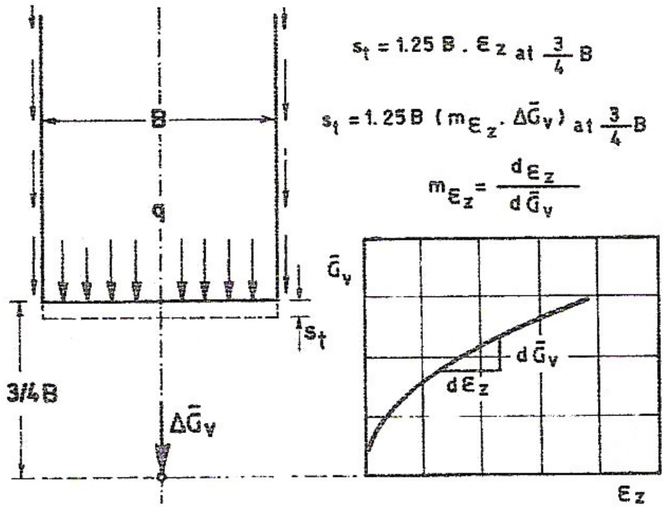

FIG. 7. RELATIONSHIP BETWEEN PILE TIP SETTLEMENT AND STRAIN AT AVERAGE POINT

Using the point located on the axis of symmetry at 3/4 B below the pile tip as "Average", published [4] strain measurements below shallow foundation indicate that settlement of pile tip, s_t, may be taken as 1.25 times the strain, ε_z , at "average point". Strains are equal to $m_{\varepsilon z}\ \Delta\ G_v$ where $\Delta\ G_v$ is Boussineq's induced pressure by the surface load and is the inverse of the tangent modulus.
For the consolidation test conditions $m_{\varepsilon z} = m_v$.

This method takes into consideration the non linear and path dependent deformation characteristics of sand. Applications have been published elsewhere. [5] [6] [7]

Consideration was given to the use of Mindlin's solution to compute ΔG_v . Nevertheless, because of the lack of tensile strength of the sand and of the large diameter of the pile, the Boussineq's solution is considered more fitting. The Mindlin's solution was used, however, to estimate the pressure induced at the "average point" by the skin friction. If the induced pressure - depth diagram from a point load is compared with the induced pressure- depth diagram from a skin friction load of equal magnitude, it is found that the area of the latter is less than 10 % of the former. Assuming that this relation is applicable to the sand formation, the induced pressures at average point after pressure cell readings, computed from the Boussinesq's solution, were increased to introduce the effect of the skin friction. In this manner a settlement of the pile tip against induced pressures at "average" point diagrams can be drawn, as shown in Fig. 8 (b). As in Fig. 6 (b) curve (a) applies before preloading and (b) after preloading.

These diagrams were drawn with data from direct pressure cell readings, Fig. 6 (b). To introduce the effect of skin friction, Fig. 8 (a) was used, where curves c), d), h) and i) have the following meaning:

c) Base resistance before preloading against settlement of the pile tip.

d) Shaft resistance before preloading against settlement of the pile tip.

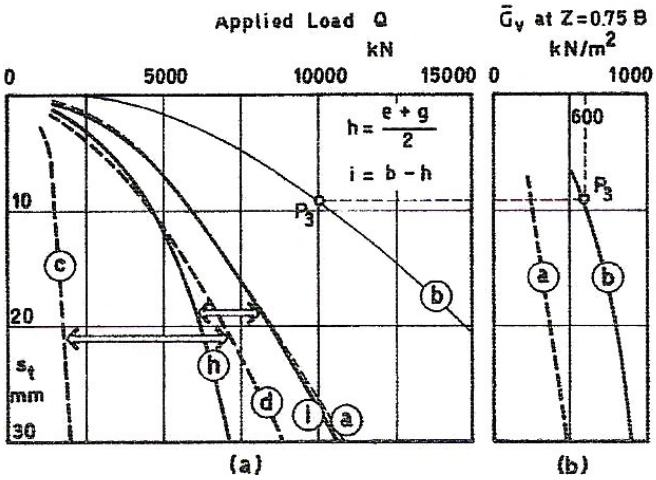

FIG. 8. APPLIED LOAD AGAINST PILE TIP SETTLEMENT DIAGRAM

h) Base resistance after preloading against settlement of the pile tip.

i) Shaft resistance after preloading against settlement of the pile tip.

From Fig. 8(b) and equation:

$$S_t = 1{,}25B \cdot \varepsilon_{z_{at\,3/4B}} = 1{,}25B \cdot m_{\varepsilon_z} \cdot \Delta\bar{G}_{v_{at\,3/4B}}$$

the diagrams G_v ε_z and G_v $m_{\varepsilon z}$ were derived, Fig 9 (c) and b), where as reference, the values of $m_{\varepsilon z}$ and ε_z for the K_o stress path of a medium dense fine sand are included.

As an example, point P_3 of Fig. 4 corresponding to the design load is represented in diagrams 8(a), 8(b), 9(b) and 9(c). When the pile is submitted to the design load after preloading, direct readings show that the strain at "average point" is 0,34% as the stress changes from overburden to overburden plus design load induced pressure. The corresponding average value of m $_{\varepsilon z}$ is 0.60 x 10^{-5} m^2/kN. The tangent modulus, inverse of m $_{\varepsilon z}$ is 1.666 x 10^{-5} kN/m^2.

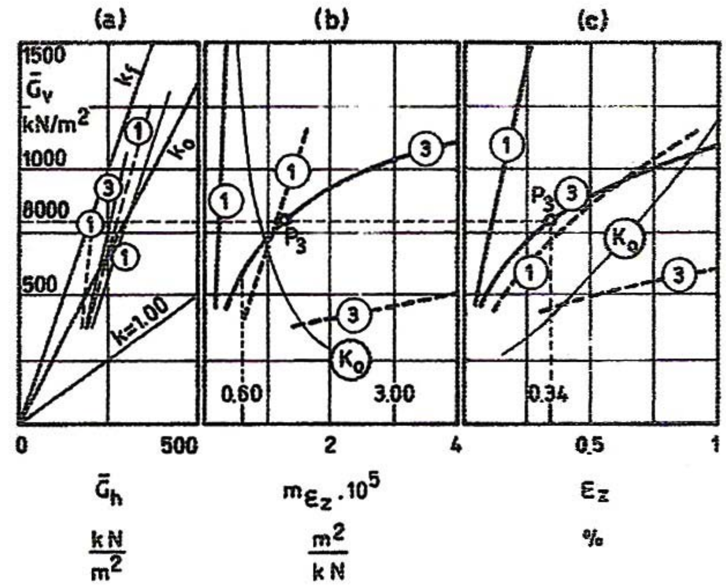

FIG. 9. STRESS PATHS AND STRAINS

As has already been pointed out, the sand around and below test pile N° 3 suffered major disturbances. Table II contrasts average values of $m_{\varepsilon z}$, when the sand is stressed to the design load, for a normally built Pile (test N° 1) and when major disturbances occurred (test N° 3).

TABLE II

Average value of $m_{\varepsilon z}$ x 10^{-5} in m^2/kN when sand is stressed to the design load.

LOAD TEST

	N°1	N°3
Before preloading	0.90 (1,110)	3 (333)
After preloading	0.15 (6,750)	0.6(1,666)

In parenthesis values of the average tangent modulus x 10^{-2} in kN/m^2.
Tentative stress paths of the "average point" are shown on Fig. 9 (a).

CONCLUSIONS

1) Large diameter piles are increasingly preferred for very deep foundations under water.

2) Due to the heavy load they carry it is mandatory that they perform as designed.

3) Inability to carry the loads on account of lack of sufficient strength of the sand below the pile base can be practically eliminated by using the preloading by stage grouting technique.

4) Rough, sturdy, satisfactory solutions have been proposed, tested and reported in this paper. There will be certainly more elaborate devices as the use of this technique spreads.

5) The settlement of single piles can be effectively controlled by this preloading technique.

6) If properly planned, even load testing under the strain to meet construction schedules can furnish information to judge: a) over the suitability of construction methods, particularly in relation to the sand disturbances they create; b) over the improvements of soil properties after preloading and c) over the stress-strain relations of the sand below the base, whose knowledge is necessary for analysis and prediction of settlements.

7) Precompression by pressure grouting acts only over a limited depth of the sand below the base. On large foundations, settlements take place because both the precompressed and the primary loaded zones compress. Settlement predictions require the introduction of appropriate moduli of deformation, which must take that fact into consideration. Measurements are being made on footings supported by 8 to 50 piles to check the results of theoretical analysis though there are not results to be reported yet.

ACKNOWLEDGEMENT

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