XIII ICSMFE, 1994, New Delhi, India / XIII CIMSTF, 1994, New Delhi, Inde

HORIZONTAL LOAD TESTS UP TO FAILURE ON VERTICAL CONCRETE PILES

PIEUX VERTICALS SOUMIS AUX FORCES HORIZONTALS JUSQU'A LA RUPTURE

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SYNOPSIS: Horizontal load tests up to failure were performed on vertical concrete piles. The piles, with a length of 26 or 42 meters and a diameter of 1.0 or 1.2 meters, were instrumented up to a depth of 20 meters with strain gauges and inclinometer tubes. Displacements and rotations of pile head were measured by means of electronic theodolites and clinometers. As the embedment soil was not an homogeneous stratum, the problem of fitting reliable curves for representing the strains along the shaft is raised. It is suggested that B-splines fitted by a weighing least-squares algorithm may overcome the problem. Once the curvatures have been obtained from the strains, the bending moments could have been reached, even after the beginning of the concrete cracking, by means of a non-linear stress-strain reinforce concrete model. Some of the main results are presented.

INTRODUCTION

Full scale tests, similar to those described in this paper, have been performed by some other researchers and research teams. However, the majority of these tests were carried out on steel piles, both commercial profiles and hollow cylinders (pipes). The interpretation of the responses of such piles is straight-forward since its material obeys to an elastic-linear law and the flexural stiffness may be assumed to be constant. Moreover, the tests are usually performed in homogeneous deposits (at least, the zone of the pile where the larger movements and bending moments take place is embedded in a practically homogeneous soil). Nevertheless, no doubts should remain about the usefulness of these tests which represents a considerable amount of work in site investigation along the past 10—15 years.

Unfortunately, in the current practice, it is very difficult to have all those "ideal" conditions. Rather, one has layered soils and reinforced concrete piles. And these must be studied. After the experience gained in "ideal" conditions, it is believed that the time for looking at this problem has arrived.

Following the research program of the Laboratório Nacional de Engenharia Civil (LNEC) on the behaviour of laterally loaded vertical piles, which began in 1985, two tests were performed at Alcácer do Sal, where a bridge over the river Sado was being built. With these tests, a better knowledge of the behaviour of laterally loaded concrete piles was achieved.

BRIEF DESCRIPTION OF THE MAIN FEATURES OF THE LAYOUT OF BOTH THE TESTS AND THE INSTRUMENTATION

As mentioned before, two tests were performed. In the first one, a single pile was loaded against a pair of piles, which were caped by a virtually rigid cap. These piles were 26 m long with a diameter of 1 m. In the second test, two individual piles were loaded against each other. These piles were 42 m long with a diameter of 1.2 m. Figure 1 represents the cross sections of the sites.

The five piles were cast-in-place by means of an outer steel tube which was recovered as the hole was being filled with concrete. Before filling the hole, the reinforce was placed. For the piles of the first test, 30 longitudinal irons of ϕ20mm and a bond of double irons ϕ10mm apart 20 cm were used. For the piles of the second test, 35 longitudinal irons of ϕ25mm and a bond of double irons ϕ12mm apart 20 cm were used.

In both tests the loads were applied in steps by means of an hydraulic jack. Figure 2 shows the load diagram of test n°2. The load apparatus was completed by a steel beam, with a high flexural stiffness, by an electrical load cell (from which the loads were measured) and, for ensuring the centricity of the loads, by two hinges which were placed at the ends for the apparatus.

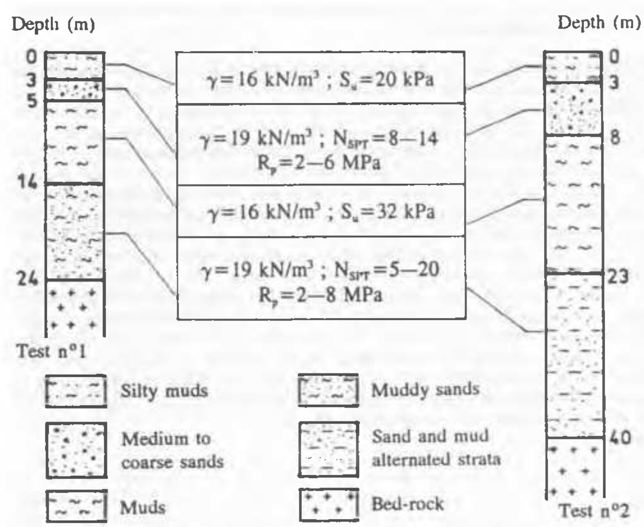


Fig. 1 — Cross sections of the sites of the tests

All piles were instrumented with strain gauges which were glued inside of steel tubes (Tavares Cardoso *et al.*, 1989). In each pile two tubes were placed, as far as possible at diametrically opposite position and in the vertical plane of the acting forces. The tubes were bonded to the reinforcement. Experience has shown that only the length equivalent to the first 10 diameters have significant movements; nevertheless, it also has been shown that the nullifying of the movements, bending moments and soil reactions is only achieved at greater depths. For these reasons, and taking into account the results of previous tests (Sousa Coutinho *et al.*, 1990*a,b,c*), the gauges were disposed every meter in the first 10 meters, and every two meters in the second 10 meters. There was, then, fifteen points of stain measure in each tube, which length was 20 meters. Details can be found elsewhere (Sousa Coutinho *et al.*, 1991*a,b*).

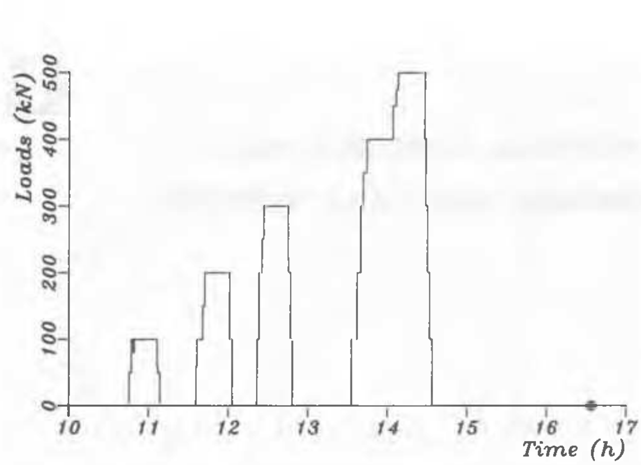


Fig. 2 — Load diagram of test n°2

In order to have independent measures along the shaft of the piles, an inclinometer tube was placed inside each pile (likewise the strain tubes, they were bonded to the reinforcement).

At the head of the piles (or at the cap, in the case of the test n°1) rotations and (horizontal) displacements were measured, respectively, by clinometers and electronic theodolites. The measurements took place only at the top and bottom of each load cycle.

RESULTS OF THE TESTS

Unfortunately, due to space limitations, it is not possible to discuss all the results of both tests. The main emphasis is settled on the results of the gauges, which allowed to determine the strains along the shaft of the piles. From these, the curvatures were calculated which, in turn, allowed to reach the bending moments. All the other section resultants and the movements of the shaft were also accomplished.

When dealing with steel piles and homogeneous soils, the measured strains along the shaft of the piles are well-behaved, and the resulting curve may be established by a single polynomial or by an interpolation of a polynomial spline. This is also possible with concrete piles in homogeneous soils, provided the cracking had not begun; in fact, at that deformation level it might be said that the piles exhibit a linear elastic behaviour. Some of the earliest tests of LNEC research program were interpreted, with acceptable results, as stated (Sousa Coutinho *et al.*, 1990*a,b,c*), since the loads applied to the piles were limited in such a way that the strains never could reach the value of 100×10^{-6} (for the concrete of the tested piles that value was less than the value for which the cracking would begin). In detail, the curves representing the strains along the shaft were established by fitting to the experimental points a polynomial of a certain degree *m* by a weighted least-squares approximation (Conte and de Boor, 1980, modified by Sousa Coutinho, 1990, for comprising boundary conditions). When tension cracking begins, the strains in the compressed region of a section are very different from those in the tension region, as shown in figure 3, where the strains of the gauges of pile n°1 at the depth of 4 m are represented.

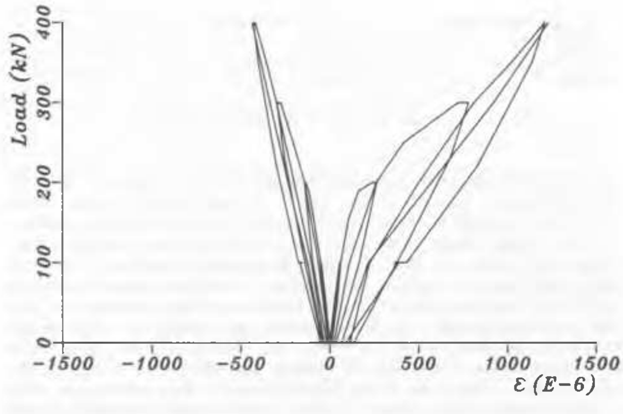
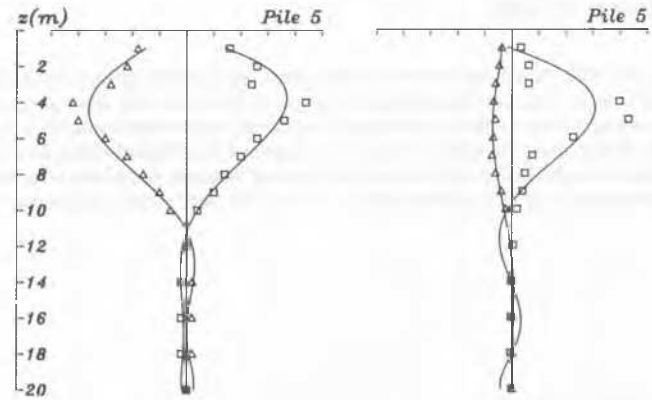


Fig. 3 — Strains of pile n°1 at 4 m depth versus the applied loads

At Alcácer do Sal, however, there were two new problems to deal with: the embedment soil was not homogeneous (see figure 1); and the loads were raised up to pile failure (or up to a state near the failure).

Although the first part of the tests, that up to the beginning of concrete cracking, had been studied with the polynomials (Sousa Coutinho *et al.*, 1991*a,b*) it was evident since the beginning that a new approach should have been taken. The usefulness of a more wide approach for dealing with layered soils had been already pointed out in the past by Sousa Coutinho *et al.* (1990*c*) and it was confirmed by Portugal (1992). It can be seen in figure 4 that even a polynomial of the six degree does not fit accurately the strain results. After the beginning of the cracking, the unfitness is increased.

In the following sections, a wider approach, that can take into account the effect in the behaviour of a pile of both layered soils and concrete cracking, is described.



Load: 100 kN; Scale: 5×10^{-6}/div

Load: 400 kN; Scale: 100×10^{-6}/div

Fig. 4 — Six degree polynimials fitted to measured strains

THE CURVATURES: B-SPLINES LEAST-SQUARES FITTING

The problems of the achievement of a reliable curve for the strains,.and thereafter for the curvatures, were overcome by fitting B-splines to the experimental data by means of least-squares. The B-spline function $S(z)$ that represents the strains along one strain tube may be written as (Cadete, 1980):

$$S(z) = \sum_{i=1}^{n+m+1} c_i \, B_{i,m}(z) \tag{1}$$

where m is the degree of the B-spline (de Boor, 1978, has written $B_{i,k}$ instead of $B_{i,m}$, where k represents the order of the spline $(=m+1)$), n is the number of knots t_i inside the interval of interest $[a,b]$ (in this case, this interval covers the length of the tube), c_i are the coefficients of the B-spline, z is the depth and $B_{i,m}$ is the product of $(t_{i+m+1}-t_i)$ by the divided difference of the order $m+1$ of the function $(t-z)_+^m$ at the points $t_i, t_{i+1}, \ldots, t_{i+m+1}$. The number of experimental points NP must be greater or equal to the number of the coefficients of the spline, $n+m+1$ $(NP \geq n+m+1)$.

In addition to the n knots taken inside the interval $[a,b]$, $m+1$ additional knots have to be taken at each interval extreme. The particular position of these knots, outside or at the boundaries of the interval, will allow to consider the boundary conditions of the B-splines to be fitted. Then, the disposition of the knots is $t_1 \leq t_2 \leq \ldots \leq t_{m+1} \leq a < t_{m+2} \ldots \leq t_{m+n+1} < b \leq t_{m+1+n+1} \leq \ldots \ldots \leq t_{n+2(m+1)}$ where no more than $m+1$ consecutive knots are equal.

The coefficients c_i may be attained by solving the usual system of equations of the least-squares methods:

$$\frac{\partial}{\partial c_i} \sum_{j=1}^{NP} \left(S(z_j) - e_j\right)^2 = 0 \quad for \quad i=1,2,\ldots,m+n+1 \tag{2}$$

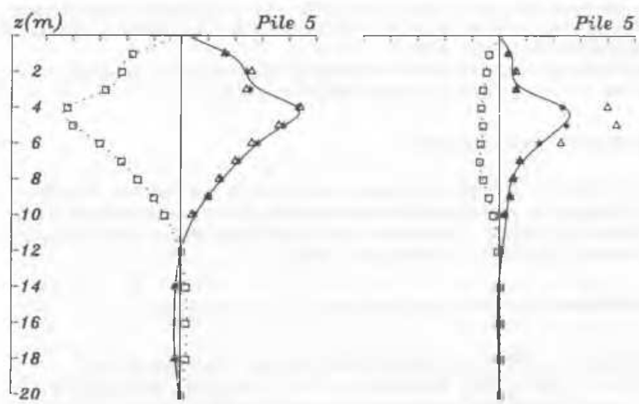
However, for each boundary condition one more equation results. For example, considering the strains to be zero at the point where the load is applied, z_q, the knots may be chosen as $t_1 < z_q < t_2 = t_3 = \ldots = t_{m+1} = a = 0$. As B-splines have a small support (in fact, $B_{i,m}(z)=0$ for $z \notin]t_i, t_{i+m+1}[$), the value of the coefficient c_i is reached directly. In this case, the system (2) should be solved only for $i=2,3,\ldots,m+n+1$. This principle is to be applied in all similar situations.

After calculating the strains, one may determine the curvatures, since the latter are directly related with the former:

$$\phi = \frac{\Delta\varepsilon}{\Delta h} \tag{3}$$

where $\Delta\varepsilon$ is the semi-difference of the strains measured at two points of the section, and Δh is the semi-distance of the two points projected on the direction of the load. The semi-differences are taken because in some cases the influence of the axial force has to be separated (as it happens in piles *n°2* and *n°3* of the first test). Note that the Euler-Bernoulli hypothesis of conservation of the shape of the sections is assumed.

In figure 5 two cases are presented, where the dotted lines (squares) represent the compression strains, while the solid lines (asterisks) represent the *curvature* strains, $\Delta\varepsilon$ (triangles represent tension strains). They both concern the same pile, but at two distinct stages of the test. In figure *5a*, a load of *100 kN* was applied and concrete cracking had not begun then. Looking at figure 1, one may understand why the strains (and the curvature) have a *hump* at the depth of *4m*. In figure *5b*, it is obvious that concrete cracking is generalized at depths of *4—5m*, having begun at *6m*, and how difficult, if not impossible, would be to adjust a single curve to that strain distribution. The adjusted B-spline is a third degree ($m=3$) one, supported on *7* knots (not including the additional knots). Consequently, there were $7+3+1=11$ constants to be determined.



Load: 100 kN; Scale: 5×10^{-6}/div — Load: 400 kN; Scale: 100×10^{-6}/div

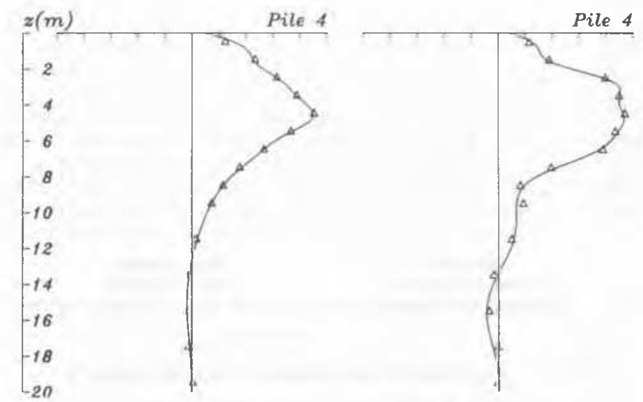
Fig. 5 — Third degree B-splines fitted to compression and *curvature* strains

Of course, it would have been possible to use both an interpolation algorithm and polynomial splines instead of the smooth algorithm and B-splines. However, an interpolation algorithm would imply a much bigger system of equations (assuming a third degree spline, it would result in 4×14 constants, plus those referred to the two boundary sections). Moreover, it would not be possible to smooth any experimental point that could be in error. Although very few (usually ≈ 1.5—2%), there are some points which results are not entirely reliable but, even so, they give some information that should not be disregarded. A smooth algorithm can deal with this problem. Furthermore, this may include a process of weighing those values (Sousa Coutinho, 1993). In fact, the final chosen algorithm was a weighted least-squares algorithm. In figure 5 it may be seen that the third point of the tension tube has a value somewhat different that one might expect — and the algorithm took this into account. All the other points took a weight equal to the unity.

THE BENDING MOMENTS

The bending moments, M, may be computed from the curvatures, ϕ, if the flexural stiffness of the pile, EI, is known: $M=\phi EI$. However, when the strains begin to increase, EI is no longer constant, due to concrete cracking in tension and also due to the non-linearity between stresses and strains. The former phenomenon causes the variation of the inertia, I, of the section; the latter the variation of the elasticity modulus, E. The problem of how M should be calculated after knowing ϕ was overcome thanks to the cooperation that exists between all LNEC's Departments — in this particular case with the Dynamics Division (in LNEC, all major specialties of Civil Engineering have their place in the Institution). Briefly, it was used a computer program that calculates the bending moments of a section if the history of both the curvatures and axial forces is known (Trancoso Vaz, 1989). The modified Park-Kent model is used for modeling the concrete (Park *et al.*, 1982); in addition to all the referred aspects, it also takes into account the raising of the strength due to the bonds, plus the hysteretic behaviour. For the steel, the Giuffré-Pinto model (Giuffré and Pinto, 1970) was used. It is also assumed that Euler-Bernoulli hypothesis holds. Details can be found elsewhere (Trancoso Vaz, 1992).

After obtaining the moments, at the depths of the gauges of the tension tube of the pile, another B-spline has to be fitted. In fact, as EI is no longer constant (E and I are both function of the depth), the variation in depth of M is not the same as ϕ. Figure 6 shows examples of diagrams of bending moments, where it can be seen that a plastic hinge is definitely installed. These B-splines are of the fifth degree and a support of eight knots. Three boundary conditions were imposed at the top and bottom of the pile. At the top, the shearing force is equal to the applied load; the moment is equal to the applied force times the distance to the "depth zero" (*16 cm* in the case of this pile); and soil reaction is zero. At the bottom, all these quantities are null.



Load: 200 kN; Scale: 80 kNm/div — Load: 500 kN; Scale: 240 kNm/div

Fig. 6 — Fifth degree B-splines fitted to bending moments calculated from a non-linear model for concrete

ROTATIONS AND DISPLACEMENTS

As known, rotations can be readily achieved by the computation of the first integral of the curvatures. According to Cadete (1989), the integral of a B-spline in a certain interval, $t_l \le z \le t_p$, is given by:

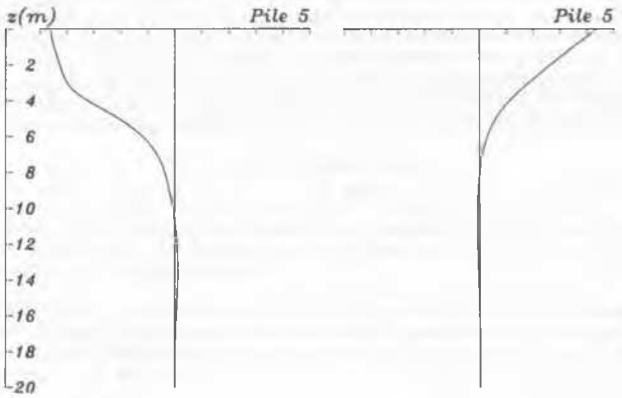
$$\int_{t_l}^{z} \sum_{i=l}^{r} c_i b_{i,m}(x)dx = \sum_{i=l}^{p-1} c_i \frac{t_{i+m+1}-t_i}{m+1} \sum_{j=i}^{p-1} B_{j,m+1}(z) \tag{4}$$

In order to have a definite integral, it is necessary to add a constant. This constant could be the rotation of the pile head which were measured by clinometers. Another choice could be to assume a null value for the rotations at the bottom of the strain tube, *i.e.*, at a depth of 20 m. It is obvious, when analysing the curvatures, that there are no movements at that depth — and the only that could have happen with a null curvature (excluding a vertical movement) was a rigid body motion, which in this kind of problem is out of question. The latter choice is a better one, for the possibility of computing the rotations at each load step. Comparing *a posteriori* the pile head rotations measured by clinometer and those computed from the integral of the curvatures, it is observed that the values read by clinometers are somewhat different. Looking at the problem in another way, if these were taken as the integral constants, the rotations at the bottom of the strain tubes would not be null. Seemingly, the accuracy of the clinometers that were used is not appropriated for a test of this kind. Table 1 presents the rotations of the head of pile 5 at the top of each load cycle, computed from the strains. From those values one may understand why the clinometer failed to measure the rotations of the pile heads (in test n°1 this was much worse, since piles 2 and 3 rotated much less).

For computing the displacements, the second integral of the curvatures, or the first of the rotations, should be established. Following the process of the establishment of the first integral of a B-spline (see Cadete, 1989, or de Boor, 1978), it can be shown that for $t_l \le z \le t_p$ (Sousa Coutinho, 1993)

$$\int_{t_1}^{z} \sum_{i=1}^{p-1} c_i \frac{t_{i+m+1}-t_i}{m+1} \sum_{j=i}^{p-1} B_{j,m+1}(x)dx = \sum_{i=1}^{p-1} c_i \frac{t_{i+m+1}-t_i}{m+1} \sum_{j=i}^{p-1} \frac{t_{j+m+2}-t_j}{m+2} \sum_{k=j}^{p-1} B_{k,m+2}(z) \tag{5}$$

Again, the constants are taken at the bottom of the strain tubes. This time, the displacements at the top of the piles are only slightly different from those measured by theodolits. These differences are not very significant and they might be owed to experimental errors. With these, a researcher has to live with. Figure 7 shows an example of rotations and displacements of pile 4.



Rotations
Scale: 200 sec/div

Displacements
Scale: 5 mm/div

Fig. 7 — Rotations and displacements of pile n°5 due to a load of 400 kN

Table 1 — Rotations and displacements of the head of pile n°5

F (kN)	100	200	300	400	500
θ (sec)	102	242	432	919	1423
δ (mm)	2.0	5.4	10.0	21.8	33.8

One way to overcome the problem of integral constants, is to measured the displacements at two different points of the pile head. This is absolutely necessary if, for some reason, one can not assume null values at the bottom of the measurement interval (at Guadiana bridge, this was impossible, since the pile diameter was 2 m, which would imply 40 m of instrumentation). At Alcácer do Sal, in fact, this was tried by means of LVDTs (Linear Variable Differential Transformer) which were to be measured at the same time of the strains, *i.e.*, in all load steps, but, unfortunately, a failure in the support of the LVDTs disallowed the measures. Nevertheless, it is believed that this should be done in all future tests.

SHEAR FORCES AND SOIL REACTIONS

The first and second derivatives of the bending moments give the shear forces and the soil reactions. The expressions of the derivatives can be found elsewhere (Cadete, 1989; de Boor, 1978). Figure 8 shows an example of both derivatives. The increasing of the soil resistance in depth, due to the sand stratum, is clearly shown. From these kind of diagrams, the stiffness of the embedment medium of the piles could have been computed.

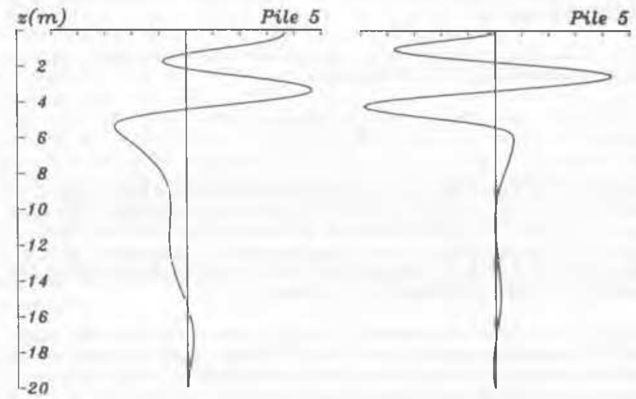
CONCLUSIONS

Some conclusions can be drawn up:

a) B-splines fitted by a weighted least-squares algorithm may overcome the problem of fitting reliable curves to the measured strains, even if the embedment soil is not homogeneous, from which the curvatures can be accomplished;

b) Despite the concrete cracking, the bending moments were computed up to failure;

c) Integrals and derivatives of B-splines are easy to compute, which allow to obtain the depth distribution of the displacements, rotations, shear forces and soil reactions;



Shear force
Scale: 110 kN/div

Soil reactions
Scale: 140 kNm^{-1}/div

Fig. 8 — Shear forces and soil reactions of pile n°5 due to a load of 400 kN

d) From a), b), c), it seems that the behaviour of the piles was studied accurately up to failure;

e) Although not quite shown in this little work, clinometers and inclinometers are not reliable tools for tests of this kind; on contrary, electronic theodolites seem to be useful;

f) Despite e), in the future the head displacements shall be measured at least at two (or three) distinct points by means of LVDTs.

ACKNOWLEDGEMENTS

Permission for publishing this paper was given by the Director. Thanks are also due to the Portuguese Road Board (JAE) for its authorization for publishing these results. The author is also in debt with Prof. Odete Cadete, Dr. Trancoso Vaz and Dr. Maranha das Neves.

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