Load transfer in dynamically and statically tested pile

Transfert de charge dans un pieu testé statiquement et dynamiquement

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SYNOPSIS A special load test program was planned in a precast concrete pile, 35 m long, 80 cm diameter, driven through marine sediments to a very dense sandy silt, in order to evaluate the resistance of the pile foundation for the amplification works of Paranagua Harbour, Brazil. A dynamic test with stress-wave measurements followed by a Vertical Slow ML Test, with maximum load of 4.64 MN, were carried out before the definitive piles were driven. The instrumentation installed along the length of the pile allowed to verify the load transfer in the static conditions and results were compared with dynamic measurements, in spite of different loading conditions.

INTRODUCTION

The amplification of the Paranagua Harbour, one of the greatest export port in southern Brazil, located near the city of Curitiba, required the design and construction of a large number of piles. In order to know the real behaviour of these piles, a special load test programme was planned over a 0.8 meters diameter prestessed concrete piles, instrumented throughout its length, driven through marine sediments to a very dense sandy silt (residual soil).

Two kinds of test were performed in the pile testing programme: a dynamic one by means of stress-wave measurements during driving of the pile, followed by a static load test, with measurements of stress and strains in the pile using strain-gauges and tell-tales. Both of thest tests were carried out before starting the driving works of the definitive piles.

Since instrumentation installed along the length of the piles has allowed to evaluate the load transfer in the static case, results could be compared with dynamic measurements, in spite of the different loading conditions.Besides these comparisons, this paper reports the interpretation of some data from the static and dynamic tests.

SOIL CONDITIONS

The soil stratigraphy of this marine sedimentary site, below 12 m or so from maximum water table, is slightly erratic. Granular soil interbedded with variable thickness layers including organic clay and silty clay were found at about 26 m. Figure 1 shows the soil profile as well as the results of the standard penetration test - SPT - carried out close to the tested pile. This pile met refusal after the bottom achieves strata of very dense sandy silt (residual soil).

DESCRIPTION OF THE PILE TEST

The precast concrete pile used in the tests was 35 m long, 80 cm outer diameter with 15 cm wall thickness. It was prestressed using seven strands of 1.27 cm reinforced to about 2.0 MPa throughout 28.8 m of the length, while the upper portion of 6.2 m was made in normal reinforced concrete. Its static bearing capacity had been estimated at 2.75 MN by theoretical means.

The pile was driven on August 20, 1987, by an air-steam BSP 24 B hammer with a rated energy of 120 KN(12 tons ram weight), up to a final length embedad in soil of 20.7 m, nearby. The static load test - a Vertical Slow ML Test -was carried out according to the Brazilian Standard NBR 6121 in September 1st, 1987.

Dynamic Measurements

Dynamic measurements were conducted by PDI Engenharia, during the driving of the pile. Strains transducers and accelerometers were attached to the pile top, while data were analysed by a PDA - Pile Driving Analyser,using the Case method. The measured transferred energy varied from 70 to 100 kNm, corresponding to a hammer eficiency varying from 58 to 83%, and the maximum compression force obtained was aproximately 7.0 MN, which corresponds to a compression stress of 23 MPa. A high tension stress value of 273.7 MPA was also verified, but no uncertainties concerning to integrity of the pile was observed by dynamic instrumentation.

Static Instrumentation

The static instrumentation consisted in strain gauges and tell-tales installed along the pile, in 3 levels each. One more level of strain gauges was used, placed out of the soil influence, in order to allow the determination of the value of Young's modulus (E).

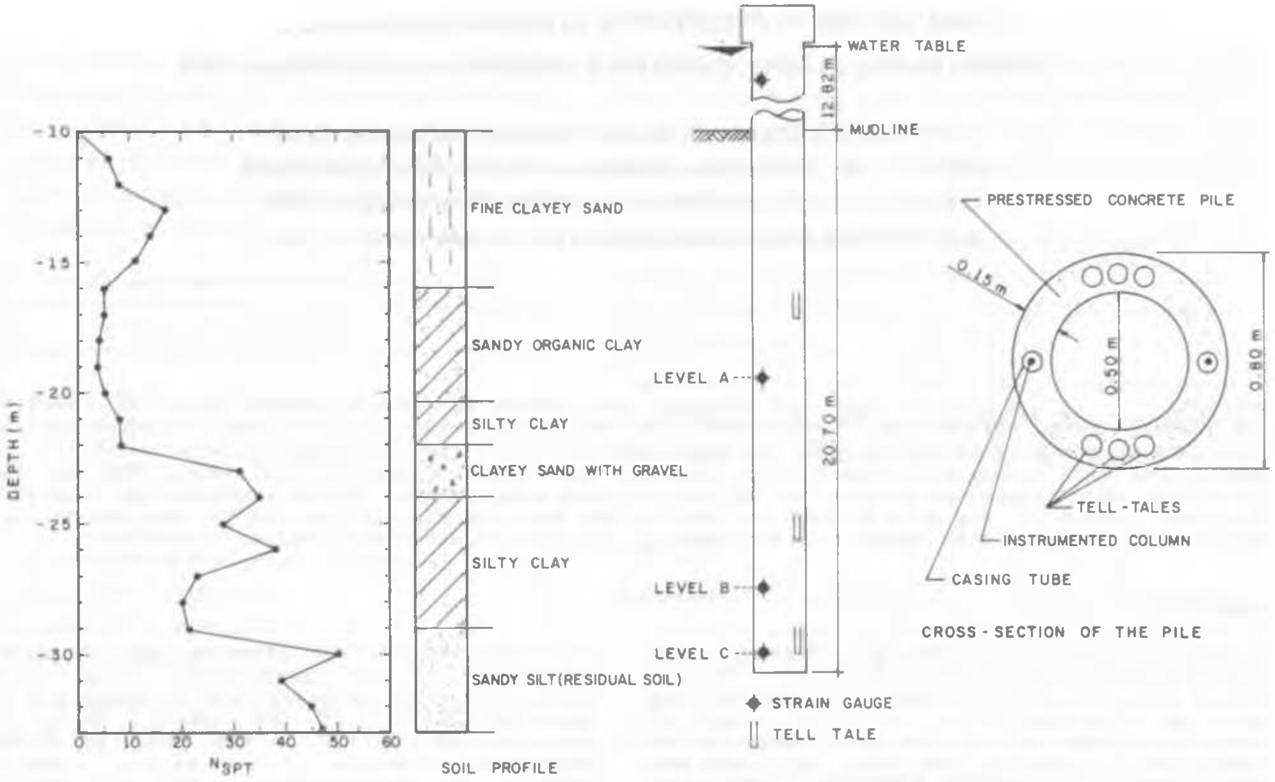

FIG. 1 - SOIL PROFILE AND SCHEMATIC OF INSTRUMENTATION

Figure 1 shows the type and location of the instruments. It also shows a cross section of the pile, where the position of some steel casing pipes placed previously inside the pile wall, with the intention of receiving the instruments after driving, can be noticed. In fact, it was decided to adopt this procedure as an attempt to avoid residual and/or odd stress as a consequence of the driving.

The strain gauges, with resolution of $5x10^{-6}$, were applied to a column, that is a 1.27 cm diameter steel bar, placed in the center of a 5.0 cm diameter casing pipe, on both sides of the pile, as indicated in Figure 1. The annular space between the instrumented column and the steel tube was filled with cement deposition. For the tell-tales, a 1.59 cm diameter alluminium road was installed into a 2.54 cm diameter casing pipe, also on both sides of the pile.

This instrumentation method was approved since all the instruments responded well during the load test and data obtained appear consistent.

TEST DATA INTERPRETATION

Analysis of dynamic measurements made during the pile driving were performed using the computer program CAPWAP. The particular blow analysed is illustrated in Figure 2, which represents the force-velocity record obtained at the end of the pile penetration. The ultimate pile static capacity, estimated from CAPWAP method, was 4.7 MN, with 2.55 MN due to shaft resistence and 2.15 MN due to the toe resistance. A damping factor of 0.4, selected in the field during the pile driving for bearing capacity assessment by CASE method with the PDA (Pile Driving Analyser), was confirmed in CAPWAP analysis. Side quake of 0.25 cm and toe quake of 0.50 cm resulted in the analysis.

Unfortunately, the soil-pile set-up effect could not be evaluated as the dynamic measurements were carried out only during the continuous driving, without redriving. Meanwhile, it is possible to suppose that, due to this effect, a rather larger resistance would be reached in restrike conditions, considering the results from the static load test.

Static load test was performed as a maintained load test, as mentioned before. Figure 3 shows the load-settlement curve obtained in this test. The load was applied in increments of about 0.7 MN each, to a maximum value of 4.64 NM, which represents nearly 170 % of the design static bearing capacity of 2.75 MN of the pile. The failure condition was not reached due to the reaction system limitations. However, the failure load was estimed at 5.5 MN according to Van der Veen's method, resulting in a theoretical safety factor of 2 (larger than that obtained from the wave-equation method). Furthermore, an uplift test was carried out also as a maintained load test to a maximum load of 1.22 MN and an extrapolated failure

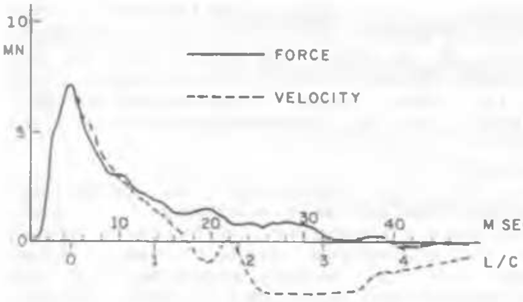

FIG. 2 - FORCE - TIME AND VELOCITY - TIME CURVES AT FINAL PENETRATION

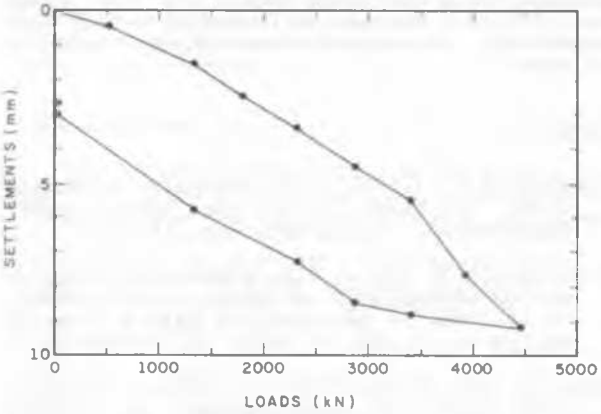

FIG. 3 - LOAD SETTLEMENT CURVE (STATIC TEST)

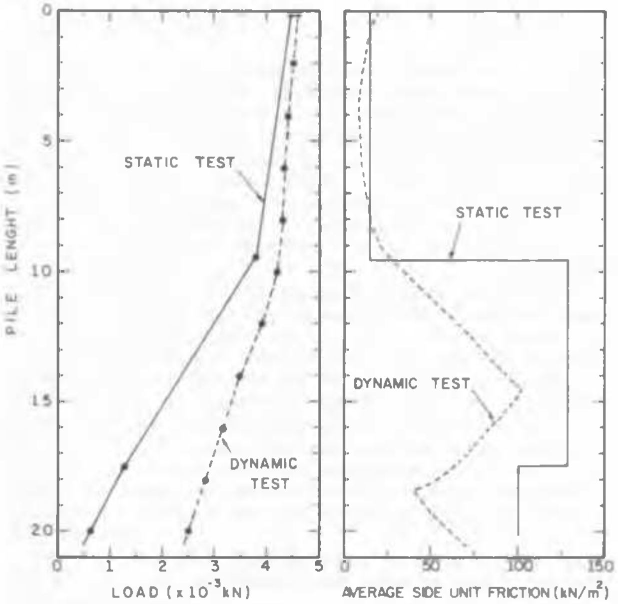

FIG.4 -DYNAMIC AND STATIC LOAD TRANSFER RESULTS

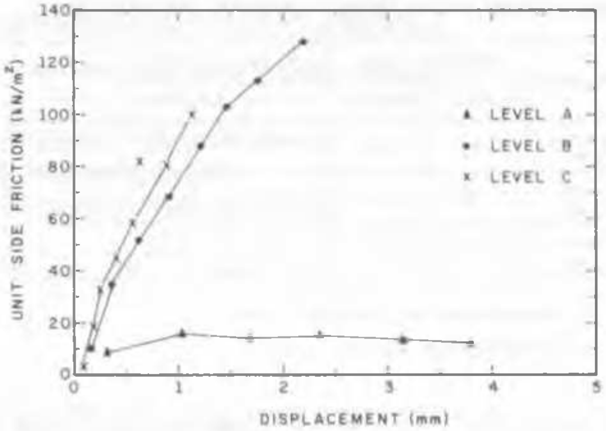

FIG. 5 - SIDE FRICTION VERSUS DISPLACEMENT OF THE TESTED PILE

load of 3.60 MN, obtained according to the same Van der Veen's method.

In Figure 4, total resistance distribuitions and side unit friction developments are shown, both computed by dynamic measurements and static instrumentation. Although representing distinguished conditions of loading, this figure allows a comparative visualization of load transfer patterns in the two situations. The major discrepancy was realized at the toe resistance percentage: 46 % by CAPWAP prediction against about 10 % from static computations.

This fact can be seen in another way by the unit skin friction-displacement relationship, computed from strain gauges and from tell-tales measurements at levels A, B and C of the pile (Figure 5). This diagram shows that only at level A the soil-pile interface adhesion achieves a unequivocal failure value of 15 Pa, which agreed vey well with CAPWAP results along this pile interval (see Figure 4-a). At others levels (B and C), the typical failure behaviour is not observed, because, possibly, the displacements at these levels were not sufficient to mobilize the full available skin friction value. Even so, the static values were larger in this portion of the pile, showing that the amount of shaft resistance was greater in the static loading.

Thus, it is valid to conclude that there was increase of shaft resistance, which influenced the load transfer curves - therefore the point resistance percentages as well.

COMMENTS ON TOE RESISTANCE

The significance of the results obtained by instrumentation, static or dynamic, must be considered with caution in open end piles, regarding the evaluation of total point resistance, as a traditional concept.

If the pile is plugged during driving, it is common to add part of the shaft resistance near

TABLE 1 - SUMMARY OF DIFFERENT RESISTANCE DISTRIBUTION

RESISTANCE	DYNAMIC TEST		STATIC TEST		UPLIFT TEST
	INITIAL	MODIFIED	INITIAL	MODIFIED	
LATERAL (MN)	2.15	1.85	3.94	3.64	3.58
POINT (MN)	2.55	2.85	0.61	0.91	–
TOTAL (MN)	4.70	4.70	4.55 (1)	4.55 (1)	3.58 (2)

(1) MAXIMUM LOAD ON THE STATIC TEST

(2) EXTRAPOLATED FAILURE LOAD

to pile tip (1 or 2 elements of pile discretization, according to usual practice) to the toe resistance value computed by CAPWAP method. This procedure is justified because part of the shaft resistance is originated from internal skin friction. In the presente case, using this modified process, a new point resistance of 2.85 MN could be estimated.

A similar fact occurs, in the static case, with instrumentation installed inside the pile wall. Herein, the point resistance is only referred to that present at the annullar area of the cross-section of the pile. Thus, considering the pile plugged and with simplified hypotesis, such as the uniform soil reaction pressure on the full section at the pile tip, it was possible to evaluate a modified point resistance of 0.91 MN, greater than 0.61 firstly computed.

The summary of different resistance distributions resulted from dynamic and static analysis is presented in Table 1. The discrepancy observed between dynamic and static loadings can be explained, firstly, by regain of shaft resistance with time rest and, secondly, because the available toe resistance was not fully mobilized in the static test. On the other hand, a good agreement between compression and uplift shaft resistances, which denotes, probably, the shaft resistance in the compression test was near to its failure limit, could be verified.

CONCLUSIONS

Dynamic measurements and instrumented static load test were able to verify the actual bearing capacity of the pile as well as its load transfer curves in both situations, alowing the following conclusions:

- The apparent proximity between bearing capacities of a driven pile, from dynamic and static methods, does not always mean that the resistance distribution in toe and shaft resistance are similar in both cases, especially when the dynamic measurements are not taken during redriving conditions.

- In terms of load transfer along the pile lenght, it is verified that skin friction obtained from dynamic measurements agrees well with static determination once the failure behaviour is clearly reached.

- Evaluations of toe resistance from instrumentation needs caution to be considered in the case of an open end pile, due to the pile plugging effect, which, if not taken into account in test data interpretation, will often leads to an underestimation of the results.

- Finally, it is important to carry out continuous studies on this subject, i.e., dynamic and static behaviour of driven piles, in order to accumulate global and local experiences for a better knowledge of the resistance mechanisms developed in both loading conditions.

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