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LOADING TESTS ON SLENDER DRIVEN PILES IN CLAY

ESSAIS DE CHARGEMENT DE PIEUX TUBULAIRES BATTUS DE GRANDE LONGUEUR

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SYNOPSIS: A new highway in southern Italy crosses the river Garigliano with a long bridge, including a number of prestressed concrete spans with a length of 38 m and two stayed spans with a length of 90 m each.
The subsoil consists of soft silty clay with some organic matter, underlain by a sand and gravel base at a dept of 50 m; accordingly, the abutments and the piers of the bridge have been founded on piles. Tubular steel piles closed at the tip and driven with a mandrel have been preferred to large diameter bored piles.
The design of the foundation involved load testing to failure of some test piles; later on, a number of proof loading tests have been performed during the construction. Three tests to failure and three proof tests have been carried out on piles instrumented to measure the deformation of the shaft, by means of a removable extensometer developed by the LCPC.
The paper reports the results of the loading tests together with the relevant data on soil properties. The information gathered proved very useful in the design of the foundation of the bridge, and add some new elements to the data base on which the design of this type of piles is based.

INTRODUCTION

The new highway between Napoli and Formia, in southern Italy, crosses the river Garigliano 65 km north of Napoli by means of a major bridge (fig. 1) with a length of 1,132 m. The bridge includes: 22 double deck prestressed concrete spans with a length of 38 m each; 4 double deck steel spans with lengths between 55 and 80 m; two stayed spans, over the river, with a length of 90 m each. The central tower of the two stayed spans is 32 m high over the deck.
Due to the occurrence of soft fine grained soils down to a depth of around 50 m, the abutments and all the piers of the bridge have been founded on piles. The preliminary design involved large diameter piles bored under bentonite mud and cast in place; for a number of reasons, tubular steel piles driven with a mandrel and subsequently filled with concrete have been finally employed.
In the early stages of the construction, some piles have been load tested to failure to check the design analyses. Later on, a number of proof loading tests have been carried out. Some of the test piles have been instrumented to measure the axial deformation of the shaft. On the whole, 3 tests to failure and 3 proof tests on instrumented piles are available.

In the present paper the subsoil properties and the criteria of choice of the pile type are briefly reported. The results of the loading tests are then presented and discussed, drawing some conclusions on the behaviour of the piles.

SOIL PROPERTIES

The bridge is located in the outer part of the alluvial plain of the river Garigliano; the soils have been deposited under a coastal marsh environment. The subsoil consists of silty and sandy clays with a substantial organic content down to a depth exceeding 50 m, where a sand and gravel base is found.
Layers and pockets of sandy soils and some peat horizons are scattered within the clayey sediments; in general they occur more frequently north of the river, where a tributary stream joins the Garigliano.
Mandolini and Viggiani (1992) present a documentation of the subsoil properties, showing that the nature of the soils is rather uniform all over the bridge length. The soils north of the river appear to be overconsolidated, with values of OCR ranging between $6 \div 8$ near the ground surface and $1{,}5 \div 2$ at a depth of $45 \div 50$ m. The soils south of

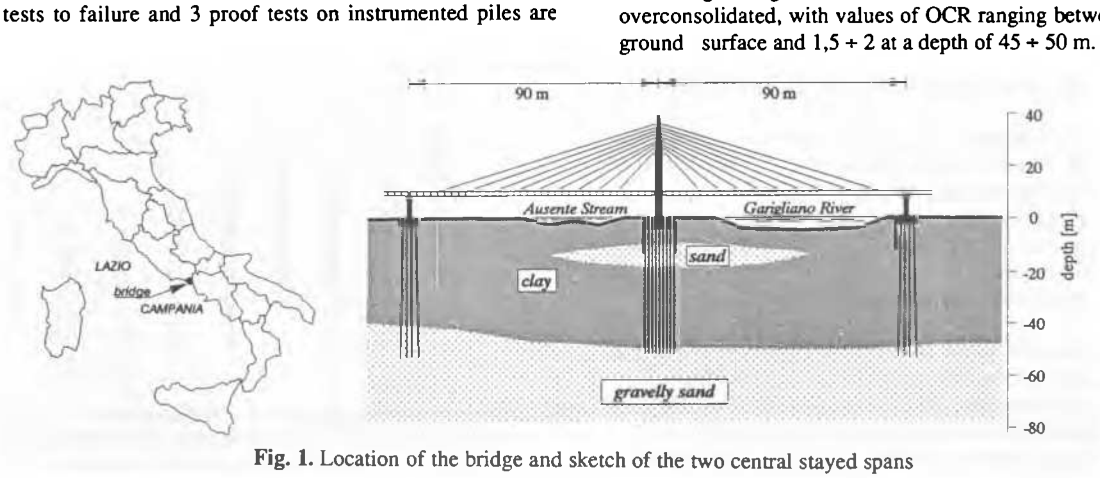


Fig. 1. Location of the bridge and sketch of the two central stayed spans

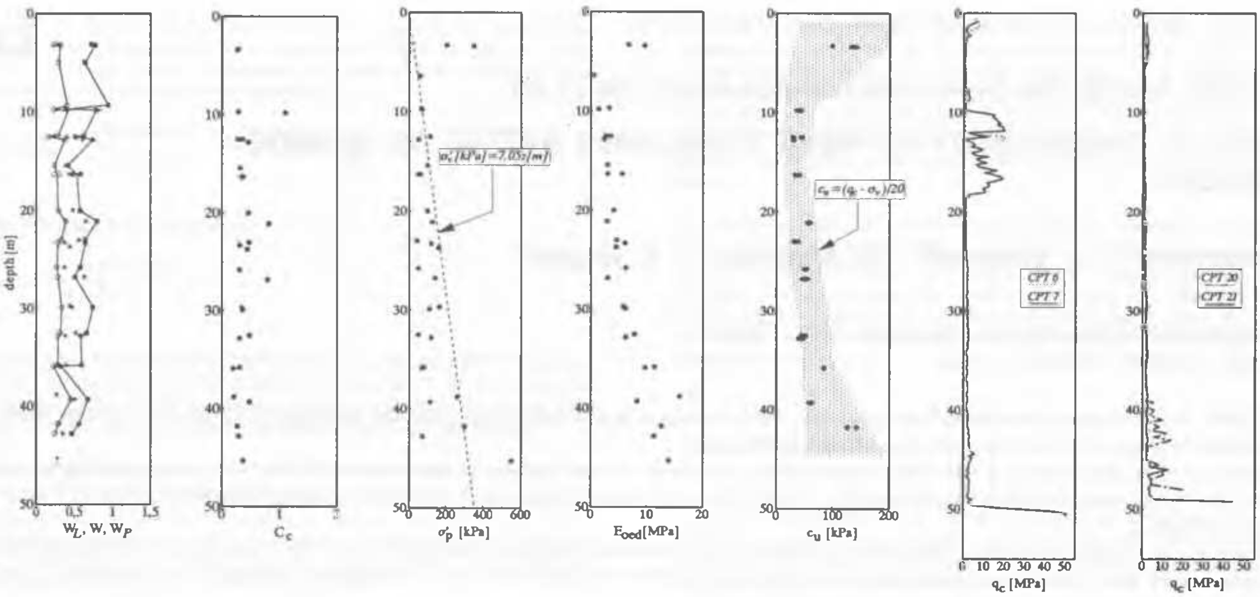


Fig. 2. Soil properties. The preconsolidation pressure σ'_p is obtained by oedometer tests; E_{oed} is computed at the effective stress in the field

the river, on the contrary, are normally consolidated with the exception of an OC surface crust 4 ÷ 5 m thick.

All the loading tests presented herein have been performed in the NC soils on the south side of the river; accordingly, only some data relating to these soils are presented.

In fig. 2 the consolidation properties and the undrained shear strength of the fine grained soils are plotted against depth, together with some profiles of the static cone resistance q_c measured in CPT soundings. The CPT n. 6 and 7 may be considered as typical of the foundation of the pier n. 7, where four of the six loading tests presented have been carried out. It may be seen that the subsoil at this location is characterised by the occurrence of a significant sand layer between the depths of 9 and 19 m. The sand and gravel base underlying the clay is found at a depth of 50 m below ground level.

The CPT n. 20 and 21 is representative of the situation prevailing at the locations of the other two tests (piers 18 and 20). Here the clay deposit is rather uniform; at the bottom, the transition to the sand and gravel occurs trough a number of thin alternating sandy and clayey layers.

The shear strength of the clayey soils in terms of effective stress may be characterised by c' = 0 and φ' = 35°.

The ground water regime is very nearly hydrostatic, with a ground water table located less than 1 m below the ground surface.

INSTALLATION OF THE PILES AND LOADING TESTS

The piles adopted are proprietary of ICELS Pali s.p.a. and are commercially known as "Multiton" piles (Fioruzzi *et al.*, 1991). Their installation involves the following stages (fig. 3):

- driving of the lower element (ϕ 356 mm), closed at the tip by a welded steel plate. In this stage, the driving mandrel acts on the bottom plate and on the upper edge of the tube;
- welding of the upper element (ϕ 406 mm) through a special connecting piece;
- driving of the two elements to the desired depth, with the mandrel acting on the bottom plate, on the connection and at the upper edge;
- installation of the reinforcement cage, if any. For the piles to be instrumented, at this stage a central steel tube is also installed (inner diameter = 50 mm; outer diameter = 60 mm). This tube, closed at the lower end and provided with watertight external joints, is designed to receive the removable extensometer;
- concreting of the pile shaft.

The main reasons for having selected Multiton piles for the Garigliano bridge are:

- high output called for by a very tight construction schedule;
- ease and safety of execution; ease of inspection;
- very low environmental impact, as compared to the bentonite piles.

The test load has been provided by a single 10 MN hydraulic jack reacting against concrete blocks kentledges; the load has been measured and controlled to an accuracy of 0.1% by means of a strain gauges load cell. The settlement of the pile head has been measured by means of four LVDT, four mechanical dial gauges, and by optical levelling.

The day before each test, the inner tube was equipped with the removable extensometer type EX-89 developed by LCPC (Bustamante *et al.*, 1990), with 12 measurement levels as diagrammatically shown in fig. 4. Table 1 reports the main elements of the six loading tests performed on instrumented piles.

The test piles n. 2, 3 and 4 belong to the foundation of the pier n. 7, that is the central pier of the two stayed spans of the bridge. The pile n. 1, loaded to failure, had been installed close to the same pier. All these piles have been driven to reach the sand and gravel underlying the clay (see CPT n. 6 and 7 in fig. 2, noting that the piles have been driven from a working plane 3 m below the ground level).

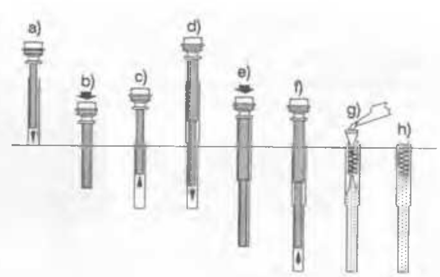


Fig. 3. Stages in the installation of a Multiton pile. a), b) c) driving the upper tube; d), e), f) final driving; g) concreting; h) completed pile

Table 1. Pile loading tests

Test pile n.	Location	Type of test	L (m)	Max. test load (MN)	Max. settlem. (mm)
1	Pier n. 7	Failure	48.8	4.25	85.1
2	"	Proof	48.6	2.00	9.1
3	"	Proof	49.2	2.00	8.3
4	"	Proof	49.2	2.00	8.3
5	Pier n. 18	Failure	48.1	2.50	112.4
6	Pier n. 20	Failure	48.7	3.00	108.6

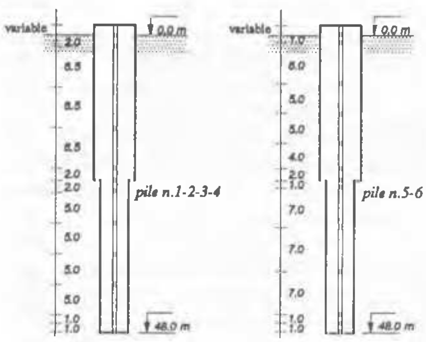


Fig. 4. Position of the extensometer within the test piles; the black dots represents the positions of the "*bloquers*"

The test piles n. 5 and n. 6, loaded to failure, had been installed respectively in the vicinity of the piers n. 18 and 20. The corresponding q_c profiles are those of CPT n. 20 and 21(fig. 2).
It may be seen that the tips of both piles do not reach the sand and gravel base; they have been driven to the transition zone where alternating layers of sand and clay are found.

LOADING TESTS RESULTS

Group effect

The load-settlement curves of the proof loading tests on the piles n. 2,3 and 4 are reported in fig. 5; in the same figure, a sketch shows the position of these piles within the group of 144 piles belonging to the foundation of the pier n. 7. The pile n. 1, though not belonging to the group, is close enough to justify the assumption that the subsoil conditions are the same; the driving records too substantiate this assumption. Accordingly, the initial part of the load-settlement curve of the loading test to failure on the pile n. 1 is also reported in fig. 5.
It may be seen that the load-settlement response of the piles in the group is definitely stiffer than that of the isolated pile n. 1.

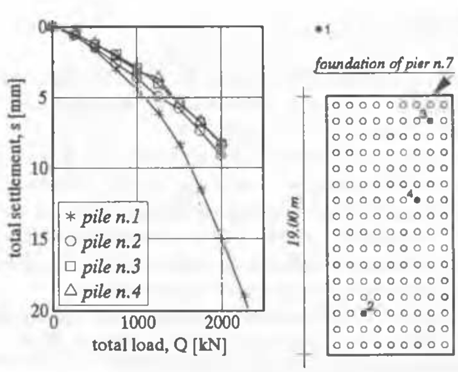


Fig. 5. Loading tests at the location of the pier n. 7

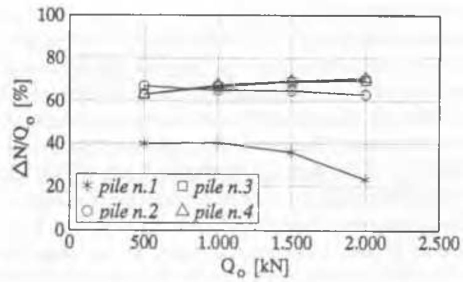
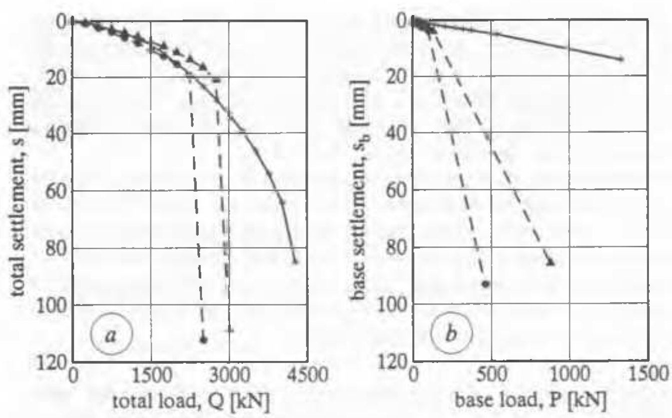


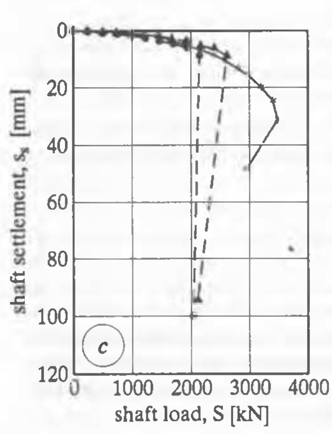
Fig. 6. Load ΔN transferred by skin friction to the sand layer in percentage of the applied load Q_o

An analysis of the load distribution along the shaft of the piles reveals that this effect is mostly due to an increased load transfer capacity by side shear in the sand layer occurring between the depths of 9 and 19 m below the ground level. In fact (fig. 6), for the isolated pile n. 1 the load ΔN transmitted by side friction to the sand layer amounts to 40% of the total applied load Q_0, decreasing to 25% as the applied load increases from 500 to 2,000 kN. For the piles within the group, and for the same load levels, the percentage $\Delta N/Q_0$ is as high as 60% ÷ 70%.
It is evident that the sand layer has been densified and the horizontal stress increased by driving the piles of the group.

Point resistance

The load-settlement curves of the three loading tests to failure (piles n. 1, 5 and 6) are reported in fig. 7a.
In fig. 7b the curves of the tip load P versus the tip settlement s_b are also reported. The values of P and s_b have been obtained from the measurements of the axial deformation of the shafts carried out by





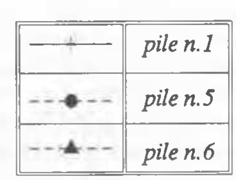


Fig. 7. Loading tests to failure. a) Total load; b) base load; c) shaft load

means of the removable extensometer.
A significant point resistance has been developed only for the pile n. 1, end bearing on the sand and gravel. For this pile the compression of the shaft has been such that the tip settlement attained a maximum value of only 14 mm, while the corresponding settlement of the head of the pile was as high as 85.1 mm. The maximum mobilised point resistance P equals 1,330 kN (13.3 MPa), and is probably smaller than the ultimate failure value; for comparison, it may be recalled that the cone resistance q_c of the CPT in the sand and gravel exceeds 30 MPa (fig. 2).
For the piles n. 5 and 6 the compression of the shaft has been substantially smaller (around 2 cm instead of 7 cm); with a maximum settlement of the pile head of around 11 cm, the tip settlement attained a maximum value of around 9 cm, more than sufficient to fully mobilise the ultimate point resistance.
The ultimate values measured for the unit point resistance are as high as 4.7 MPa for the pile n. 5 and 8.9 MPa for the pile n. 6. It seems that the former pile ends in a clayey layer, while the latter in one of the transition sandy layers that are apparent in the CPT n. 20 and 21 in fig. 2.

Shaft resistance

In fig. 7c the curves of the mobilised shaft resistance S versus the mid-height settlement s_s are plotted. The shaft resistance is defined as:

$$S = \int_0^L \pi \, d \, \tau \, dz$$

where d and L are the diameter and length of the pile and τ is the shear stress at the pile-soil interface. For all the piles loaded to failure, S attains a peak followed by a decrease to an ultimate value. The latter accounts for 70% to 80% of the ultimate total load; at the settlement where the peak of S occurs, 90% to 95% of the applied total load is resisted by side shear.
A prediction of the shaft resistance S had been obtained by assuming that the mobilised unit resistance equals the undrained strength c_u of the soil (i.e., by using the so called "α method" with $\alpha = 1$). A value of S = 2,370 kN had been obtained, to be compared with peak values of 2,150 and 2,600 kN, and ultimate values of 2,050 and 2,100 kN measured for the piles n. 5 and 6, with the shafts entirely in clay.
The mobilisation of the unit shear resistance at different depths along the shaft is depicted by the transfer curves reported in fig. 8. The data of pile n. 1 have been omitted, because the related measurements present some uncertainties. On the contrary, some data measured on the proof piles n. 2, 3 and 4 have been added; they refer to the upper levels of these piles, where the vertical displacement have been large enough to attain a significant shear stress mobilisation.
Most of the transfer curves in fig. 8 exhibit a more or less pronounced unstable behaviour. Such a behaviour, together with the high axial deformability of the pile shafts, originates a progressive failure phenomenon that explains the decay of the shaft resistance from the peak to the ultimate value.
An estimate of the reduction of S, based on the procedures suggested by Randolph (1983) and Semple and Rigden (1984), gives reduction factors ranging from .7 to .8, to be compared with the observed values of (2,050 ÷ 2,100)/2,370 = .86 ÷ .89. The above procedures, in the present case, seem to be slightly conservative but substantially sound.

CONCLUDING REMARKS

The loading tests confirmed that all the piles, either isolated or within a group, were capable of taking in full safety the design service load of 1,2 MN. Most of the service load is resisted by skin friction, the point resistance being very low for the piles end bearing in sand and practically negligible for the floating piles.

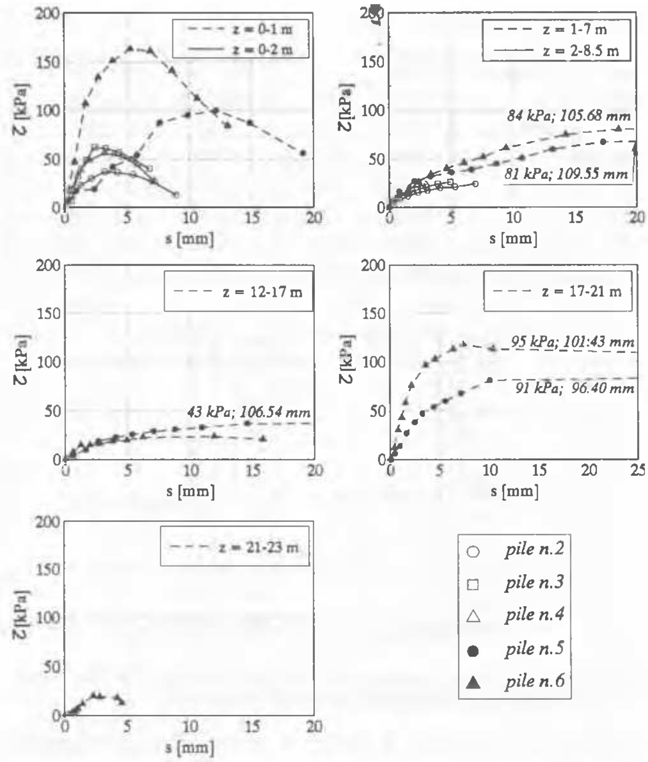


Fig. 8. Transfer curves of the shaft resistance at different levels along the pile shafts

Owing to the high slenderness of the piles, a progressive failure occurs in the shaft resistance. The ultimate value of the shaft resistance may be evaluated following the suggestions of Randolph (1983) and Semple and Rigden (1984).

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