Deep Foundations on Bored and Auger Piles – Van Impe & Van Impe (eds)

Bi-directional instrumented load test of a pile bored in Guinea Bissau

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ABSTRACT: On July 2nd, 2007, LOADTEST ltd., the LCPC and MB-Foundations performed a static loading test with an Osterberg-cell on a bored pile of 1600 mm diameter. The 56.50 metre deep test pile constructed by Trevi Spa for Soares da Costa group, was in Guinea Bissau near the Cacheu River. This test was conducted to validate the design of the foundations of the Sao Vicente bridge; a project undertaken by the Ministry of Public Works of the Republic of Guinea Bissau and funded by the European Development Fund. The pile were loaded statically using the O-cell® bi-directional method 25 days after concreting. The piles were also instrumented using LCPC removable extensometers. This paper provides a very brief presentation of the bridge project and the test pile instrumentation: LCPC removable extensometers and O-cell® instrumentation. The results are detailed and analyzed herein and the combined use of these two instrumentation systems is a world première.

1 INTRODUCTION

As part of the Trans-Africa road project, the Ministry of Public Works of the Republic of Guinea Bissau and the European Development Fund set out to build a 620 metre long bridge over the Cacheu River. In order to evaluate the foundation design for the future São Vicente Bridge, it was decided to make a static load test using the Osterberg Method. To obtain additional information on the behaviour of the test pile, a complementary instrumentation system using the LCPC removable extensometer string (Bustamante & Doix 1991) was deployed.

The 1600 mm diameter bored test pile was constructed to a depth of 56.50 metres at a location near the river. This pile was also used to validate the drilling method for the project.

2 SITE SUB-SURFACE CONDITIONS

Many geological investigations were necessary (on land and in the river) to attempt to accurately determine the soil characteristics. The contract specified the use of the Menard pressuremeter method.

Table 1. General sub-surface description.

NGGB Level [m]	Description	Pressure limit average (Mpa)
+3.0 to +2.0	Backfill/ working platform	0.05
+2.0 to −7.9	Soft silt	0.1
−7.9 to −25.65	Clay	1.0
−25.65 to −51.35	Sands	2.0
−51.35 to −53.5	Compact clay	2.5

The sub-surface stratigraphy at the general location of the test pile and the Menard pressure limit (pl, results of pressuremeter test) were reported to Table 1.

3 TEST PILE CONSTRUCTION

Trevi Spa, employed by Grupo Soares da Costa S.A., began excavating the dedicated test pile on June 5, 2007 and performed the final cleanout and concreting on June 7, 2007. The 1600 mm test pile was excavated to a tip elevation of –53.50 m, under bentonite. The pile was started with a 1624 mm O.D. casing, installed by vibratory driving with a Soilmec VS-8.

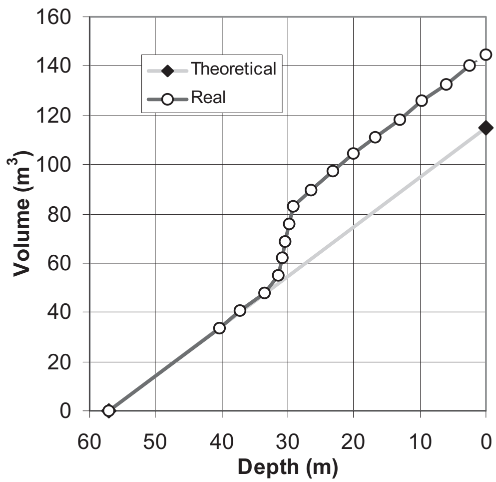

Figure 3. Concreting curves (real and theoretical).

Figure 1. Link Belt LS-418 with Soilmec RT3-ST.

Figure 2. The reinforcing cage with the O-cell.

To perform the piling operations, Trevi Spa utilized a Link Belt LS-418 drilling system attached to a Soilmec RT3-ST, see Figure 1. The bottom of the pile was airlifted after drilling. After cleaning the base, the reinforcing cage with attached O-cell® assembly (Fig. 2) was inserted into the excavation and temporarily supported from the steel casing.

Concrete was then delivered into the pile by a 300 mm O.D. pipe into the base of the pile.

The Figure 3 presents a comparison between the theoretical and the real concrete curves. There was an expected soft layer between approximately 33 and 29 metres where the over consumption of concrete required an additional 35 m^3 of concrete. This over consumption coincided with a level of grey sand, where the Menard pressure limit indicated less than 1 MPa.

4 OSTERBERG CELL TESTING

4.1 *Pile instrumentation*

Test pile instrumentation and assembly was carried out under the direction of LOADTEST with the assistance of LCPC and MB-Foundations. The loading assembly consisted of one 670 mm O-cell located 10.50 metres above the toe of the pile, see Figure 4. The Osterberg cell was calibrated to 13.6 MN and then welded closed prior to shipping by the manufacturer, American Equipment and Fabricating Corporation.

O-cell® testing instrumentation included four Linear Vibrating Wire Displacement Transducers (LVWDTs, Geokon Model 4450) positioned between the lower and upper plates of the O-cell assembly to measure expansion. Two telltale casings were attached to the reinforcing cage, diametrically opposed, extending from the top

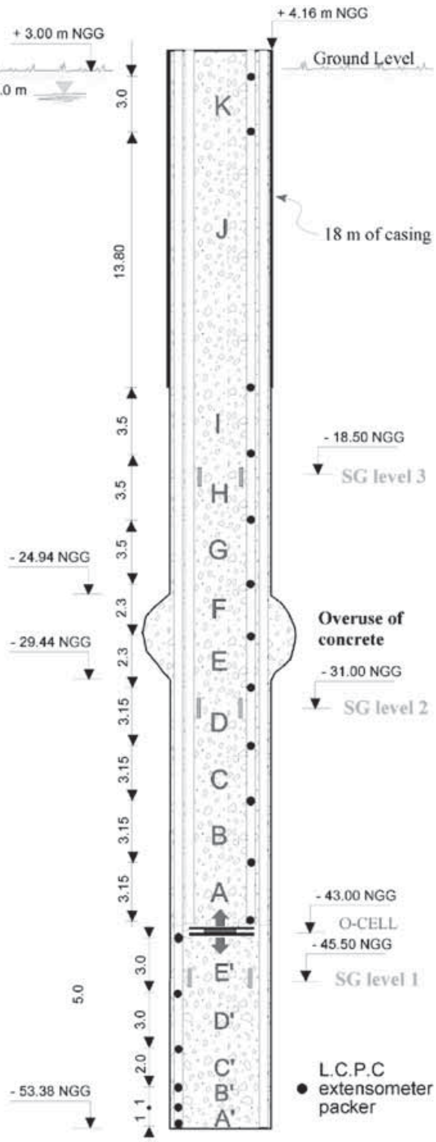

Figure 4. Schematic section of the test pile.

of the O-cell assembly to beyond the top of concrete to measure the total pile compression. Strain gauges were used to assess the side shear load transfer of the pile above and below the O-cell assembly. One level of two "sister bar" vibrating wire strain gauges (Geokon Model 4911) was installed, diametrically opposed, in the pile below the base of the O-cell assembly and two levels of two gauges were installed in the pile above it.

All telltale casings were constructed using a nominal 25 mm O.D. embedded steel pipe and 6 mm internal steel telltale rod.

Figure 5. Two LVDTs: one monitoring the top of pile from the reference beam and the pile compression above the O-cell.

Two lengths of steel pipe were also installed, extending from the top of the pile to the top of the bottom plate, to vent the break in the pile formed by the expansion of the O-cells. The O cell assembly was attached normal to the reinforcing cage on the ground and when lifted to vertical, racking of the cage caused a tilt of the assembly across the cage which was measured as 65 mm. This tilt would correspond to a required correction to the applied load of only 0.11% and was ignored.

4.2 *Test arrangement*

Throughout the load test, key elements of pile response were monitored using the equipment and instruments described herein. Pile compression was measured using 6 mm telltales installed in the 25 mm steel pipes and monitored by Linear Voltage Displacement Transducers (LVDTs). Two LVDTs attached to a reference system were used to monitor the top of pile movement, see Figure 5.

The reference system consisted of an 11.5 m 300 × 300 mm H-beam welded between a pile casing one end and a buried steel beam upright at a height of approximately 1.25 m. The supports were located approximately three pile diameters from the centre of the test pile. The beam was not shaded during the test. An automated digital survey level (Leica NA 3003) monitored the reference beam for movement during testing from a distance of approximately 15.85 metres to a precision of ±0.01 mm. A maximum upward movement of 4.62 mm was observed for the reference beam; this was assumed to be due to environmental effects and not settlement of the reference system which had been assembled two days prior to the start of the test.

Both a Bourdon pressure gauge and a vibrating wire pressure transducer were used to measure the pressure applied to the O-cell at each load interval. The pressure transducer was used for manually setting and maintaining loads and real time plotting. The Bourdon pressure gauge readings were used as a check on the transducer and for data analysis. There was close agreement between the Bourdon gauge and the pressure transducer.

In addition to the requirements for bi-directional testing, allowance was made during construction of the cage for the installation of up to two removable extensometer strings, to be monitored and reported on by LCPC as required. One string was intended to be used for the pile element above the O-cell arrangement and one for below. The extensometers defined a total of 16 measuring sections, see Figure 4.

4.3 *Data acquisition*

All the installed O-cell instrumentation was connected through a data logger (Data Electronics 515 GeoLogger) to a laptop computer allowing data to be recorded and stored automatically at 30 second intervals and displayed in real time. The same laptop computer synchronized to the data logging system was used to acquire the Leica NA3003 data.

The data acquisition of the strain gauges within the removable extensometer string was conducted manually.

4.4 *Testing procedures*

The test was begun by pressurizing the O-cell in order to break the tack welds that hold it closed (for handling and for placement in the pile) and to form the fracture plane in the concrete surrounding the base of the O-cell.

After the break occurred, the pressure was immediately released and the testing started. Zero readings for all instrumentation were taken prior to the preliminary weld-breaking load-unload cycle, which in this case involved a maximum applied pressure of 200 psi (1.38 MPa) to the O-cell.

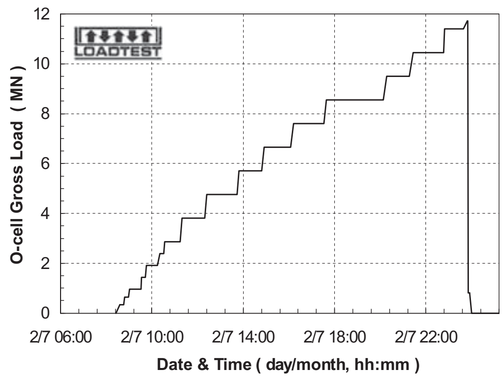

Figure 6. O-cell load increment—time plot.

The Osterberg cell load test was conducted as follows: The 670 mm diameter O-cell, with its base located 10.50 metres above the base of pile, was pressurized to assess the combined end bearing and lower side shear characteristics of the pile section below the O-cell using the skin friction above as reaction. The O-cell was pressurized in 17 loading steps up to 51.02 MPa (7400 psi) resulting in a bi-directional gross O cell load of 11.4 MN (Fig. 6).

Each successive scheduled load increment was maintained constant for a minimum of 30 minutes and up to a maximum of 180 minutes while automatically maintaining the O-cell pressure constant until a creep criterion of less than 0.05 mm per 10 minutes in each direction was met. The data logger automatically recorded the instrument readings every 30 seconds.

5 TEST RESULTS AND ANALYSES

On 2nd July 2007, LCPC and LOADTEST performed the static loading test. Figure 7 shows the instrumented pile head during the test.

The loads applied by the O-cell act in two opposing directions, resisted by the capacity of the pile above and below. It can be considered that the O-cell does not impose an additional upward load until its expansion force exceeds the buoyant weight of the pile above the O-cell. Therefore, net load, which is defined as gross O cell load minus the buoyant weight of the pile above, is used to determine skin friction resistance above the O-cell and to construct the equivalent top-loaded load-settlement curve. For this test a pile buoyant weight of 1.47 MN above the O-cell was used.

5.1 *Ultimate resistance*

With a settlement value $S_0 = 85.94$ mm, using a failure criteria of 1/10th of the pile diameter as the total settlement, the ultimate resistance Rc of the pile was not

Figure 7. Pile Head with instrumentations.

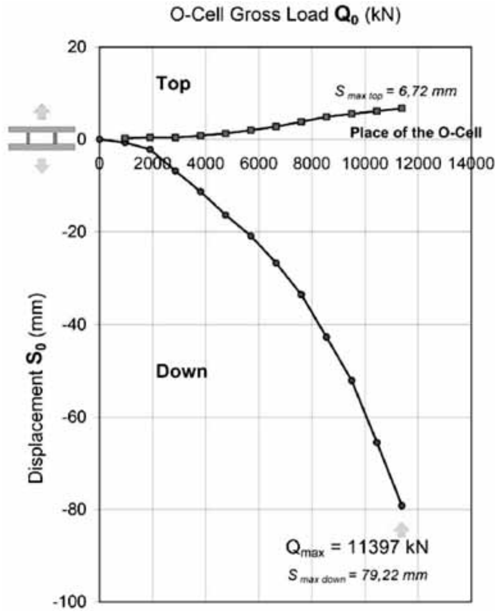

Figure 8. Load settlement curves.

achieved under the maximum load test of 22.8 MN (2280 t). By using the method of Chin (Chin 1970, Fleming 1992), we can estimate the ultimate resistance to be about 30 MN.

The maximum downward applied load was 11.7 MN which occurred at the last load interval. At this loading, the average downward movement of the O-cell base was 79.2 mm after correction due to reference beam movement.

The maximum upward applied gross load was 11.7 MN. At this loading, the upward movement of the top of the O-cell was 6.7 mm. Figure 8 shows the combined load settlement curves in both directions.

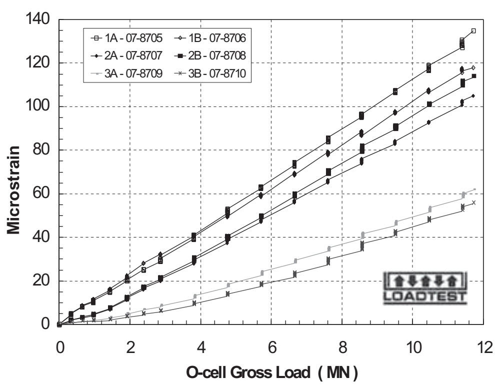

Figure 9. Strain guage micro-strain results (me) (SG 1a. 1b, 2a 2b, 3a & 3b).

5.2 *Skin friction resistance*

The different strain gauge systems employed during the O cell loading test gave similar results.

5.2.1 *Above the O-cell*

The upward movement of the pile (Fig. 8) and the strain gauge micro-strain results (Fig. 9) have been analyzed to determine the characteristic skin friction values.

The analysis of the two strain gauges (SG 2 & 3) shows a maximum mobilized unit skin friction of:

q_s = 33 kPa at 21.5 m depth in the clay.
q_s = 62 kPa at 34 m, in sand.

The upward movement of the pile (Fig. 8) and the removable extensometer micro-strain results (Fig. 10) have been analyzed to determine the skin friction distribution. The analysis of the stain gauges within the removable extensometer string give unit skin friction values of:

q_s = 80 kPa from 17 to 21.5 m depth, clay level,
q_s = 60 kPa from 21.5 to 24 m, it is for sand,
q_s = 65 kPa from 32.1 to 35.25 m, sand,
q_s = 110 kPa from 35.25 to 38.4 m, sand,
q_s = 60 kPa from 38.4 to 41.55 m, sandy clay,
q_s = 50 kPa from 41.55 to 44.7 m, sand.

This analysis is presented in Figure 11.

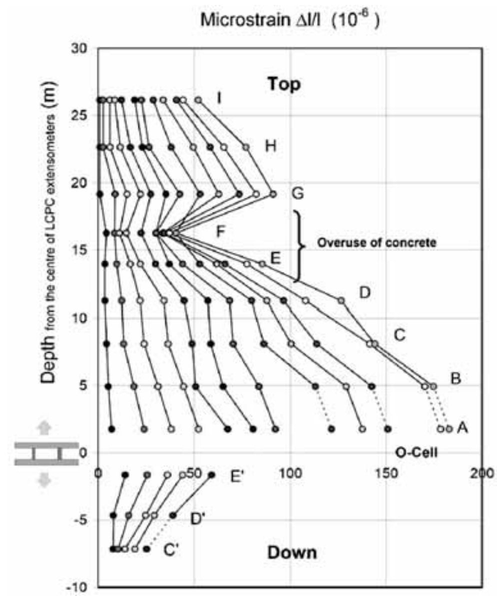

Figure 10. Removable extensometer micro-strain results.

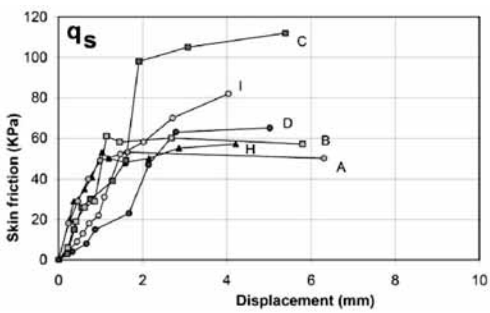

Figure 11. Mobilization of shaft resistance along the pile above the O-cell.

5.2.2 *Below the O-cell*

During the load increment of 4.76 MN, water caused an electrical problem and it was decide to stop the acquisition of the removable extensometer below the O-cell. If the results of the removable extensometer are extrapolated from the strain distribution previously determined, (SG level 1) it is possible to estimate the toe resistance of the pile as 22.8 MN and a total skin friction of approximately 7.2 MN.

The analysis of the strain gauge (SG 1) indicates a unit skin friction of: $q_s = 72$ kPa at 48.5 m depth in sand at the end of the test.

The interpretation of extensometer measurements made by the LCPC and MB Foundations identified unit skin friction q_s for different levels along the pile (Fig. 10). The comparison between the measured values and those taken from curves recommended by the French norms (Fascicle 62 Title V) shows the validity of the values taken in the foundations design notes (determination of the length of the piles) of Soares da Costa group.

6 CONCLUSION

The pile behaviour measured downwards allows the ultimate capacity to be determined and a total bearing capacity below the O-cell of 22.5 MN (asymptotic definition) is deduced.

The most interesting conclusion made from this test is that it has been proved possible to construct the pile with LCPC removable extensometer and use the O-cell methodology to implement the loading. Due to the low cost of implementing the O-cell test it has proved more economical than a traditional top-down axial static loading test.

The use of the removable extensometer in determining unit skin friction along the pile has been proved beneficial in determining whether to slightly modify the method of pile design analysis and also provided a nice insight into the effect of the oversize cross section.

The combined use of these two instrumentation systems is a world première and we hope to make further tests and create a database.

ACKNOWLEDGEMENT

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