Special foundations for an urban viaduct in Lisbon

Fondations spéciaux pour un viaduct urbain à Lisbonne

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ABSTRACT

The aim of this paper is to present the main design and execution criteria adopted for the foundation of an urban viaduct with a deck of 32m wide and 770m of overall length, correspondent to 11 spans ranging from 50 to 105m, flying over the Padre Cruz Avenue, and being part of the last construction phase of the North – South Lisbon Expressway.

RÉSUMÉ

L'objective de cet article c'est la présentation des principaux critères de conception et d'exécution adoptés pour les fondations d'un viaduct urbain avec un tablier de 32m de large et 770m de longueur, correspondent a 11 travées variant de 50 a 105m sur la Avenue Padre Cruz, faisant partie de la dernière phase constructif de l´axe routier Nord – Sud de Lisbonne.

Keywords: soil improvement, foundations, jet grouting, micropiles

1 INTRODUCTION

The viaduct over the Padre Cruz Avenue with 770m of overall length allows the conclusion of the North - South Lisbon Expressway (IP7) at the Lumiar residential quarter, connecting the 25 April bridge, at south over the Tagus river, to the CRIL (regional internal ring expressway of Lisbon), at the North side of the Lisbon international airport, as shown in Figure 1.

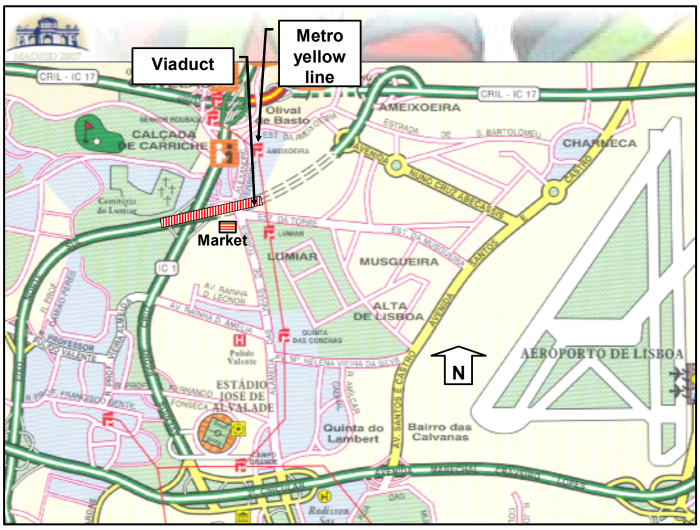

Figure 1. Plan of Lisbon with the viaduct location

2 MAIN RESTRAINTS

2.1 *Traffic, Metro and Structural Restraints*

The viaduct has only one single caisson deck with the following span distribution, from East: 50m + 75m + 105m + 87m + 105m + 72m + 54m + 57m + 57m + 57m + 51m. The length of those spans was induced by the location of the Lisbon Underground Metro, yellow line, under operation and the red line, to be built in the future, as well as the demanding structural minimum height of the deck caisson.

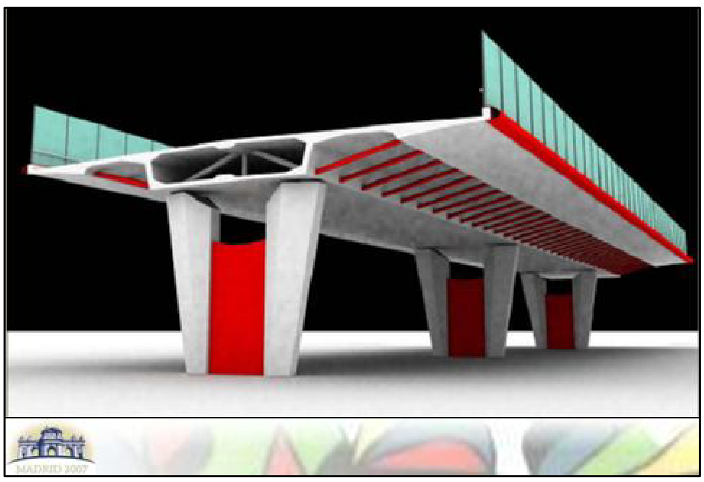

Figure 2. Viaduct cross section 3 dimensional view

The viaduct deck with 32,4m wide allows 3x2 lanes of traffic. From the structural point of view the deck is located over a reinforced concrete caisson with internal and external steel trusses, the last ones supporting the deck external cantilevers, as illustrated in figures 2 and 3. The existence of a 2,0m diameter water supply pipe, as well as the location of the Lumiar Market under the viaduct deck, lead also to the adopted spans distribution.

2.2 *Geotechnical and Geological Restraints*

On a summarized way the geotechnical conditions can be described from the surface as following:

- Heterogeneous fills: in general sandy clays with small stones, and an overall depth ranging from 5m to 0,5m.
- Alluvium and soft Miocene soils: sandy silt and clays with an average depth of about 5,5m. The number of blows N_{SPT}/0,3m are lesser than 30, with the average ranging from 8 to 14.
- Miocene bed rock: sandy silts with intercalations of calcareous boulders. The N_{SPT}/0,3m blows are always bigger than 60.

3 ADOPTED SOLUTIONS

Taking to account the existent main restraints it was adopted for the soil improvement a solution of jet grouting type 3 columns with 1500mm of diameter at the fills, alluvium and soft Miocene soils.

This solution had the double function of soil improvement and foundation of the viaduct superstructure. In order to better accommodate the tension loads due to the seismic action, micropiles hollow steel tubes TM-80 ∅127x9mm (yield stress of 560MPa) with external ring couplers at the joints were installed between the peripheral columns, as illustrated in figures 3, 4 and 5.

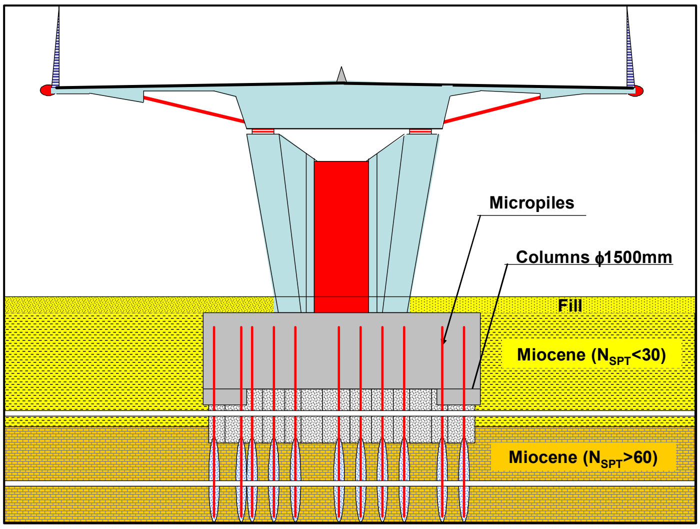

Figure 3. Viaduct cross section over the pier P8

In order to obtain the designed jet grounting columns with 1500mm diameter, axial compression stress and young modulus at failure of 4,0MPa and 3,0GPa, the following parameters were adopted: 910kg/m of cement consumption, air pressure of 10bar, water pressure of 450bar and cement pressure of 75bar. During the execution all the main parameters were permanently checked through a computerized system.

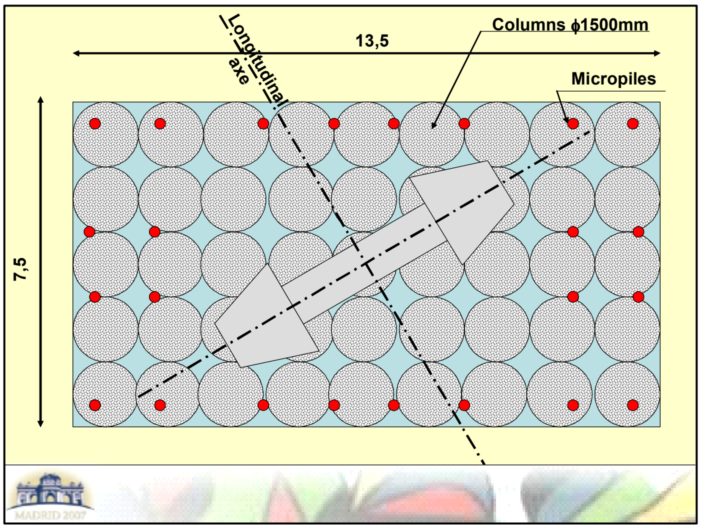

Figure 4. Plan of the pier P8 foundation

All the jet grouting columns had 1m of minimum deliver length at the Miocene bed rock. The micropiles had their 10m bond length also located at the same Miocene bed rock.

The jet grouting columns were design in order to accommodate a maximum value of compression service stresses ranging from 1,5MPa, for static loads, to 2,0MPa, for seismic loads. The micropiles were designed to accommodate axial tension service loads no bigger than 930kN, considering a shaft stress (qs) of 400kPa at the bond length for a drill diameter no lesser than 15cm. Those values were previously confirmed by a full scale pull out load test presented in section 5.

Figure 5. View of the jet grouting columns head after the excavation for the pier P4 foundation

The main design criteria were evaluated taking into account that the bending moments due to the seismic horizontal loads should be transformed by binary effect in axial loads, and the tension axial loads should be accommodated by the micropiles. The shear loads should be accommodated by compression struts formed at and between the jet grouting columns, as shown in figure 6.

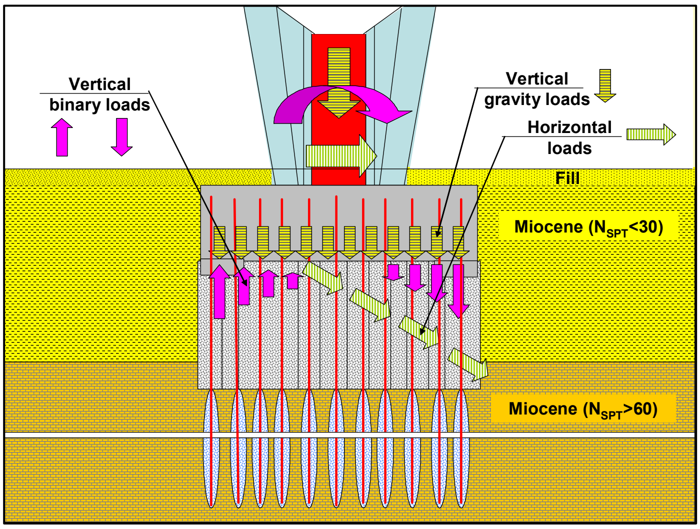

Figure 6. Main design criteria: distribution of loads from the cap to Miocene bed rock

The option for this solution and not for a more traditional one, using large diameter bored piles (1500 or 2000mm) was based on both economical and constructive criteria. The piles should have an overall length not lesser than 15 to 20m (10 diameters), demanding the intersection of several meters of calcareous boulders with negative consequences on the drilling rate. Other important issue was also the dimensions of the reinforced concrete caps for the traditional piles solution, much bigger than the adopted for the micropiles and jet grouting columns solution, as illustrated in figures 5, 6 and 7.

Figure 7. Installation of the reinforcement bars at the micropiles and jet grouting columns cap

4 INTERACTION WITH THE METRO TUNNEL

The location of the pier P8 close to the Lisbon Metro tunnel (under operation yellow line) demanded the analysis of the consequences related to the increment of stresses at the tunnel level, taking into account both the structural and the geological scenarios, as illustrated in figures 8 and 9.

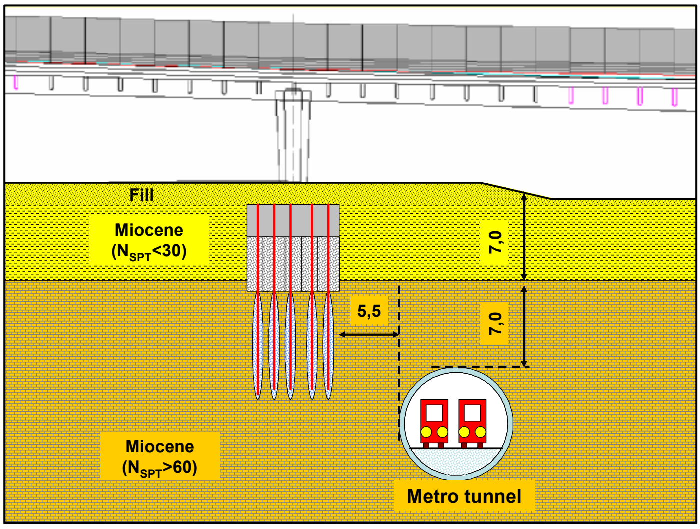

Figure 8. Cross section of the Metro tunnel and pier P8

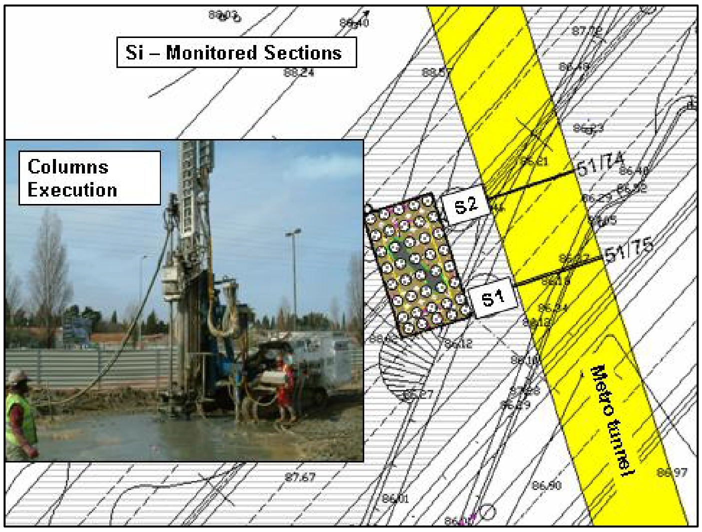

Figure 9. Plan of the Metro tunnel and pier P8

Considering both the location and the solution adopted for the foundation of the pier P8: 24 TM-80 ∅127x9mm micropiles and 45 jet grouting columns with 1500mm of diameter as improvement of both the fill and the soft Miocene soils, it was considered as very important the analysis of the stress field spread induced by the viaduct foundation in order to evaluate the behavior of the Metro tunnel structure formed by independent reinforced concrete segments. For this purpose a FEM analysis was performed, allowing the definition of the alert and alarm criteria used for the monitoring and survey plan, as shown in figures 10 and 11.

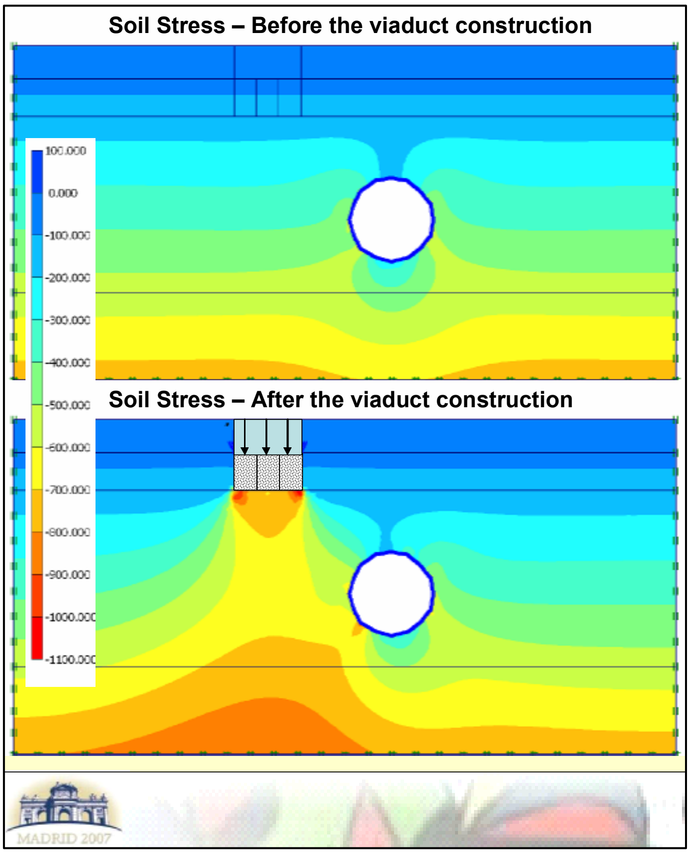

Figure 10. Stress field around the Metro tunnel (kN/m^2)

It is important to point out that the adopted solution for the foundation of the pier P8 allows the transmission to the bed rock of a compression and uniform stress not bigger to 550kN/m^2. Comparing with the traditional solution of large diameter bored piles, resting some meters on the bed rock, the adopted one has the advantage of minimizing the perturbation of the soil located around the Metro tunnel, allowing also a better spread and a better leveling of the viaduct induced stresses.

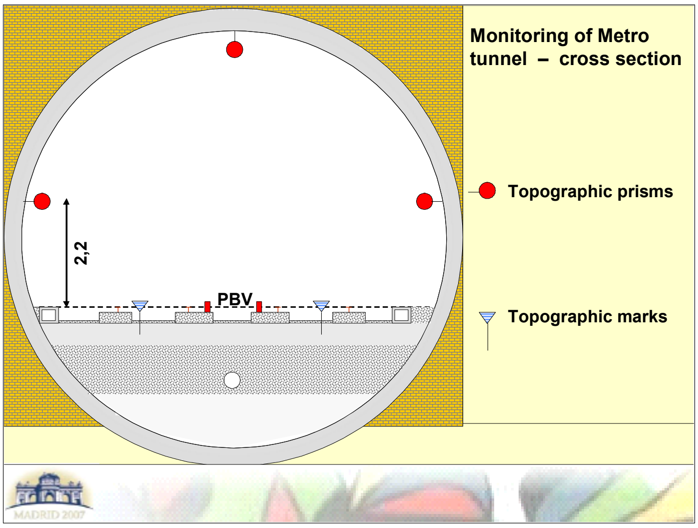

Figure 11. Metro tunnel: cross section monitoring

5 FULL SCALE LOAD TEST

5.1 *Scenario*

In order to previously simulate the tension axial load applied to the microplies and to the jet grouting columns a pull out full scale load test was performed on a micropile steel hollow tube TM-80 ∅127x9mm with external couplers and an overall length of 16m.

The tested micropile was sealed inside a jet grouting column with a diameter of 1500mm in the first 6m, simulating the improvement of the soft soils by the jet grouting columns.

The reaction structure was a triangle shape steel truss with HEB profiles and was fixed to the soil also by one single micropile and jet grouting column in each edge with the same characteristics of the tested elements.

In order to simulate as better as possible the viaduct foundations the pull out test was performed just a few meters from the foundations of the pier P4, at one of the worst geological scenarios regarding the characteristics of the soft soils.

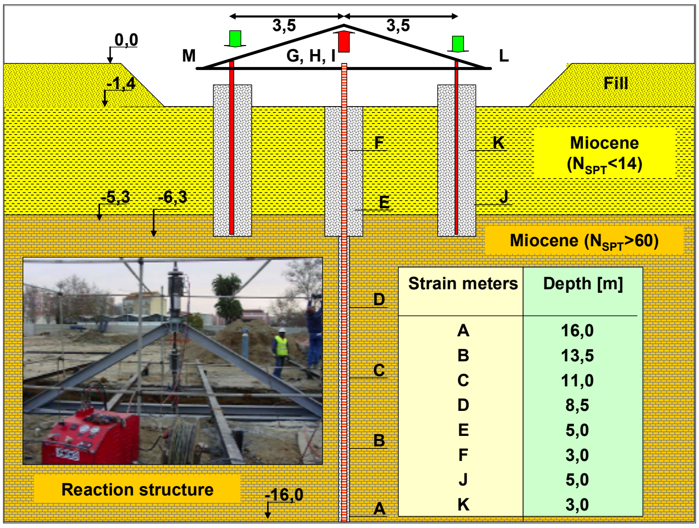

Figure 12. Pull out test: geology and monitoring

5.2 *Loading System*

The test program included a total of three cycles of charge and discharge, correspondent approximately to 1,0, 1,5 and 2,0 times the maximum service load. At the third cycle a failure occurred at the welding connection between the top of the micropile steel hollow tube and the loading device. This device consisted on steel plates and two ∅63mm Gewi bars (A500/550), allowing the transmission of the load from one hydraulic jack, located over the reaction structure centre, as illustrated in figures 12 and 13.

The reaction loads were also transmitted to the jet grouting reaction columns with 1500mm of diameter through one single ∅63mm Gewi bar, sealed inside each column.

Figure 13. Pull out test: applying load and reaction system

5.3 *Monitoring*

The adopted monitoring system included the loaded and the reaction columns, as well as the hydraulic jack and the reaction structure. The main quantities measured were the applied loads and the deformations at several depths. For this purpose the following devices were installed: 1 load cell at the base of the hydraulic jack, 13 deflectometers at the reaction structure and at the top of both the loaded and reaction columns and 8 rod strain meters installed inside the columns at several depths according to the local geology, as shown in figures 12, 13 and 15.

5.4 *Test Program*

The test program was defined in order to reach a maximum pull out load of 1860kN, about two times the maximum tension axial service load, as illustrated in Figure 14.

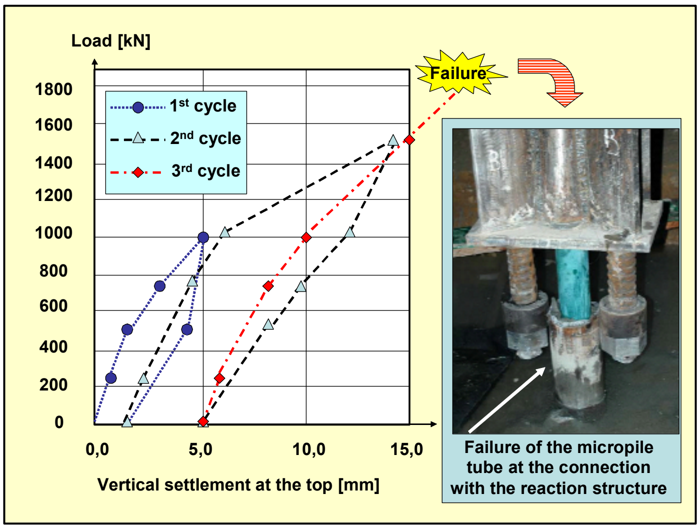

Figure 14. Pull out test: load – deformation curve

5.5 *Main Results*

From the analyses of the main results it was possible to point out the elastic behavior of the loaded micropile under service loads, as well as the fact of the majority of the tension load had been transferred to the soil through the jet grouting column. This situation can be explained by the value of the shaft stress (qs) mobilized at the interface between the jet grouting column and the soft soil.

Those results confirm the versatility and the suitability of this solution, mainly the use of jet grouting columns as both soil improvement and foundations, due to its geometry and to the value of the shaft stresses mobilized, as shown in Figure 15, together with the confinement of the soil located between the jet grouting columns.

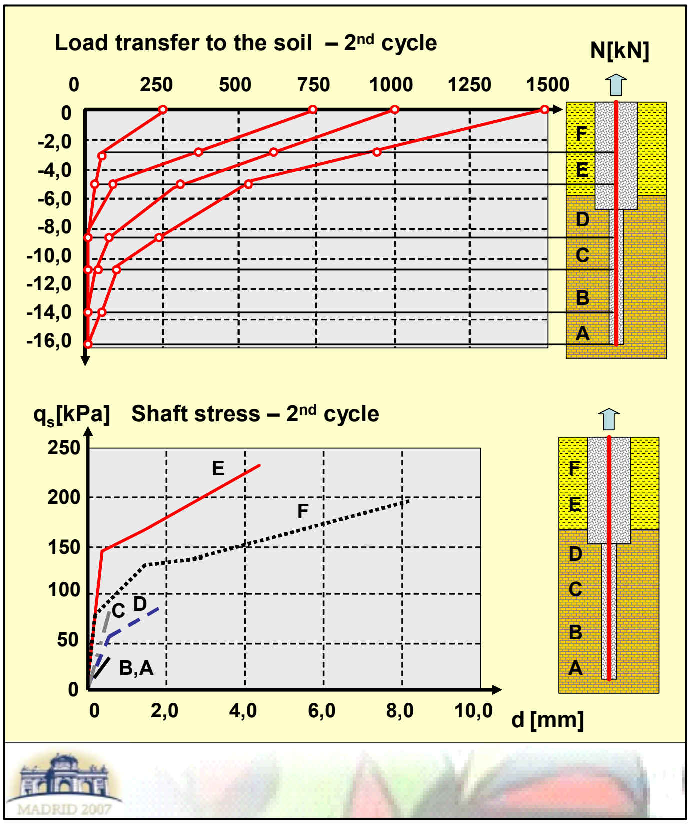

Figure 15. Pull out test: load transmitted to the soil

6 JET GROUTING EXECUTION CONTROL

Previously to the execution of the pull out test and the jet grouting final columns a field of trial columns was built in order to calibrate the columns geometry and resistance. Excavations around the columns were performed for the analysis of the geometry and the collection of cores for axial compression tests, performed with measurement of the Young's modulus. Those tests allowed the selection of the main execution parameters, which were permanently monitored through a computerized system during the execution of the final columns.

During the excavation for the execution of the reinforced concrete caps at the level of the columns head it was possible to observe some columns with a diameter lesser than 1500mm. When this situation occurred CPT tests were performed in the soil located between the columns confirming that the geometry reduction was due to the improvement of the soil characteristics when confined between the columns, leading to an average more resistant material formed by the columns and the confined soil.

In all the foundations for the piers and the abutments cores were also collected inside the final columns in order to check its resistance through unconfined compression tests, as illustrated in Figure 16.

Figure 16. Execution of cores on the final columns in order to check the geometry and the resistance

7 MAIN QUANTITIES

It is possible to point out the following main quantities regarding the foundations of all piers and abutments:

- 3.900m of jet grouting columns ∅1500 (triple);
- 5.820m of micropiles steel hollow tubes TM-80 ∅127x9mm with external ring couplers.

8 MAIN CONCLUSIONS

As main conclusion already stressed by Pinto et al (2003) it is possible to state the versatility and suitability of the adopted solutions in this kind of structural and geotechnical scenarios, mainly its versatility when compared with the traditional solutions of large diameter bored piles. The monitoring of the viaduct deck over the piers shows until now (see Figure 17) just a few millimeters of vertical deformations due to the soil loading.

Figure 17. Overview of the site in January 2007

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