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Jacked piles as mitigation measure of surface settlements due to tunneling in clay

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ABSTRACT: Tunnelling in highly urbanized areas very often leads to ground movements and subsequent potential risks of damage to adjacent structures and infrastructures. Intensive soil strengthening interventions ahead of and on the excavation face are usually carried out, but they are not always sufficient to keep soil movements within acceptable limits. The paper describes a well documented case-history of a shallow tunnel in clay in which, after the execution of a preliminary support by a pipe-roofing technique, jacked piles were eventually installed in order to stiffen the foundation soil under the base of the ribs and to further reduce tunnel-induced surface settlements. The reported case-history is a novel application of jacked piles as settlement reducers. The pile installation sequence during the tunnel excavation is described herein. A set of installed piles were provided with extensometers; records of monitoring data show the evolution of the load transfer mechanism from the tunnel structure to the piled foundation during excavation as function of the subsequent construction steps.

1 INTRODUCTION

Mitigation measures for the reduction of surface settlements due to shallow tunnel excavation is a recurrent challenge for geotechnical engineers, especially for shallow tunnels in soft soils and highly urbanized areas (Attewell et al., 1986, Mair & Taylor, 1997). The final part of the access tunnel to the new underground high-speed railway station in Bologna (Italy), on the Milano-Napoli line has been excavated in such difficult conditions. Figure 1 shows the overall plan of the intervention area together with the indication of the tunnel excavation sectors (from 1 to 34) and the details of the location. A typical stratigraphic section of underground conditions along the tunnel axis is reported in Figure 2 (section A-A in Figure 1), which summarizes the results of extensive site investigations. In Figure 3 a more detailed view, transversal to the tunnel direction (section B-B in Figure 1), is also provided: the soil profile consists of four distinct units:

- Unit R: silty-sandy backfill with brick inclusions;
- Unit 1: clay, silty clay and clayey silt, with local sandy lenses, from brown to green-grey, deriving from alluvial fan and paleo-riverbed deposits.
- Unit 2: silty sand, with gravel and cobble and local silty and clayey lenses, from alluvial fan and paleo-riverbed deposits. From moderately to very compact ($q_t > 10$ MPa).
- Unit 3 and 3a: clay, silty clay and clayey silt, with local rare slightly sandy lenses, from brown to green-grey. Unit 3 is moderately compact ($q_t = 1 \div 4$ MPa), Unit 3a is from compact to stiff ($q_t = 4 \div 7$ MPa).

Note that a few piezocone tests have been carried out from the tunnel bottom level, at a depth of about 22 m below the ground surface. The tunnel excavation, around 140 m² wide and 23 m deep (Fig. 3), has been carried out in such heterogeneous formations made up of alternating layers of coarse-grained and fine-grained soils, with different local water levels. Standard soil improvement techniques, such as jet grouting ahead the face and umbrella jet columns, adopted along most of the tunnel length, were replaced here by special mitigation measures due to the very limited thickness of the overlying soil and the densely populated intervention area, close to the Bologna central railway station.
Figure 1. Plan view of the intervention area with location details and indication of excavation sectors.

Figure 2. Schematic geological section along the tunnel axis (section A-A in Figure 1).

Figure 3. Schematic stratigraphic section of the subsoil and typical tunnel cross section (section B-B in Figure 1).
In order to further reduce surface settlements in this area, a rather elaborate preliminary structural support, has been carried out by microtunnelling, an assisted pipe-roofing technique. Nevertheless, settlement monitoring showed that during the excavation of the initial sectors (from number 1 to number 3 in Figure 1) and before the construction of the permanent liner, the structure of the tunnel itself could settle in the lower clayey layer due to the load transferred from the microtunnels to the steel ribs (Broere, 2000; Duan, 2001). In order to prevent tunnel structure subsidence, an array of jacked piles was eventually installed under the steel ribs. In the case-history described the innovative application of jacked piles as steel rib support, and hence as settlement reducers, is presented and relevant data recorded during the excavation and the subsequent construction steps is discussed.

2 THE REINFORCEMENT BY JACKED PILES AS SETTLEMENT REDUCERS

2.1 The jacked piling technique

The jacked piling technique, used in this application, is relatively new (Fig. 4): piles are installed by means of special hydraulic jacks (Fig. 5) and cast in-situ. The schematic section of Figure 4 shows the main pile components. The hollow tube steel pile (Fig. 4b) is driven into the ground via a quasi-static jacking force applied by hydraulic jacks (Fig. 5). The foundation raft serves as counterweight and creates a connection between piles and superstructure by means of the driving assembly, previously positioned inside the concrete foundation. In this specific case, a beam has been created at the base of the steel ribs in order to house the driving assembly. The pile base is closed by two flanges, a base flange and a widening flange. During installation, the flanges create an annular space between pile and soil which is filled with micro-concrete, maintained under pressure during installation. Consequently, the pile-soil contact surface is quite rough and the shaft resistance improved. During the installation the jacking force can be recorded and provides a measure of the soil resistance very similar to the CPT tip resistance (Marchi et al. 2010). At the end of the installation process, the inner part of the pile is filled with concrete. This technique, together with the typical benefits of displacement piles, has been chosen because of its potential advantages over traditional dynamic pile driving:

- no vibrations are produced, which is clearly a crucial aspect in urban areas;
- the small-size installation equipment can be easily handled in restricted spaces, hence, it is compatible with the narrow space in front of the excavation face of the tunnel;
- due to the characteristic slenderness of the pile (typically L/D > 20), the shaft resistance is the most important contribution to the pile overall bearing capacity. On the other hand, the jacking installation force is only due to the base resistance because of the lateral fluid micro-concrete maintained under pressure during installation.

2.2 Tunnel excavation, pile installation sequence and monitoring

The double-track tunnel construction was preceded by the execution of the pre-support along the intervention stretch (272 m long). This ground reinforcement consisted of 10 large diameter (1.6 m) concrete pipes installed longitudinally on top of the tunnel section and excavated through micro-tunnelling, a remotely controlled, guided pipe jacking process. All operations were carried out from two shafts, an entrance and an exit shaft showed in Figures 1 and 2. The pipes were installed one adjacent to the other, along an ‘umbrella’ shape but without any reciprocal interconnection, to form a pipe-roof above the tunnel excavation (Fig. 6) and then filled with reinforced concrete. As the excavation progresses, the pipe-roof provides continuous support for to the excavation face, carrying the over-burden load by the longitudinal beam action.
Thus, this ground improvement technique helps to stabilise the face and reduces ground movements (Stein et al., 1989, Thompson, 1993). However, to this purpose in this case, as already mentioned a layout of jacked piles was also eventually installed. The subsequent tunnel construction (a typical cross section is shown in Figure 6) was executed by sequential excavation. For each excavation sector, 8 m long (Fig. 7), the sequence of operations was:

- execution of drainages and, at the same time, face and border reinforcement with fibreglass rods injected every meter;
- excavation and execution of the temporary liner using shotcrete and steel ribs;
- jacked piles installation (plan in Figure 7);
- proofing and casting of the invert;
- permanent liner installation.

Figure 7 shows the lay-out of piles installed in a typical sector, from number 4 to number 34 (Fig. 1). No piles were installed in sectors number 1 to number 3. Two rows of 8 piles, 9 m and 10 m long in turn, with a diameter of 320 mm and spaced at 1 m (about 3 diameters) were installed in each of the other sectors. In addition, two of them (pile number 3, close to the excavation face and pile number 11, in the middle of the sector, in Figure 7) were provided with two extensometers located at pile extremities. An additional pile was installed in sector 4 in order to perform a load test. Data from the load test, shown in the next section, has been used in the interpretation of instrumented piles monitoring data.

3 DATA ANALYSIS

3.1 Load test

A preliminary load test was carried out in order to investigate pile performance in relation to the existing soil conditions. For this purpose, a pile, 11 m long, was installed in the middle of sector 4 and instrumented with three extensometers (one in the middle and two at the ends). The test (Fig. 8) was performed according to maintained load test procedure i.e. by applying monotonic increasing constant load steps. The interpretation of measurements enabled the typical load vs head displacement curve and the load distribution along the shaft to be plotted, via the pile extrapolated elastic modulus (Figs. 9–11). Figure 9 shows the load vs displacement curves and provides the size of the pile ultimate load. Notice that the final load steps were not fully stabilized. In addition, Figure 10 shows the estimation of the composite pile elastic modulus obtained during the load test, as usual, from the comparison between the total applied load and the measurements of the extensometer located at the top of the pile.

Note that, according to the general rule, the elastic modulus tends to a constant value as deformations increase (i.e. with load increment): $E = 30,000 \text{ MPa}$ is therefore the reference value used in the monitoring data analysis. Figure 11, representing the
load transfer curves along the pile shaft, shows that load at the pile toe starts to increase when the head load is already about 300 kN; lower loads are almost completely taken by the upper shaft. Data in Figures from 9 to 11 are a useful tool for the interpretation of monitoring data.

3.2 Monitoring data analysis

Figure 12 shows a significant example of the recorded monitoring data. Every extensometer provided three data logs and their average has been elaborated in order to obtain the size of the load resting on piles. The calculations of loads, starting from the $\mu\varepsilon$ records, have been carried out assuming $E$ constant and equal to 30,000 MPa, as suggested by the load test (Fig. 11). Figures from 13 to 15 show the analysis results for the instrumented piles in sectors 5, 6 and 8. These sectors have been selected among the whole database because of their representativeness and their location below a very densely built area (see Fig. 1). The period covered by the records is very variable, from about 15 days (19 days in sector 8, shown in Figure 15) to about 2 months (60 days in sector 5, shown in Figure 13).

As a consequence, the load evolution on every single pile is shown for a variable number of subsequent excavated sectors (5 in Figure 13, 4 in Figure 14 and 2 in Figure 15). The superimposition of the works timetable and data enables the in situ interaction mechanisms between piles and the tunnel structure in relation to the various excavation and construction phases to be identified. The pile load patterns in sector 5 (Fig. 13) shows that load taken by piles 3 and 11 increases during excavations of the subsequent sectors and execution of temporary liner. In particular, the load increase due to the excavation of the closest sector (sector 6) is higher than the increase due to the excavation of the subsequent sectors (7, 8, 9 and 10). During face reinforcement no clear trend can be identified, while the installation of piles clearly causes unloading, particularly on pile 3, close to the excavation face. Comparing pile 3 and 11 a final head load, common to both piles, of about 500–600 kN can be identified. This
Figure 13. Evolution of the load applied to: (a) piles number 3 and (b) pile 11 in sector 5, during the tunnel construction.

Figure 14. Evolution of the load applied to: (a) piles number 3 and (b) pile 11 in sector 6, during the tunnel construction.
Figure 15. Evolution of the load applied to: (a) piles number 3 and (b) pile 11 in sector 8, during the tunnel construction.

4 CONCLUSIONS

This paper describes a well-documented case-history of jacked piles being used as provisional steel rib support during excavation of an urban tunnel in clay. A preliminary ground reinforcement intervention by a microtunnelling—an assisted piperoofing technique—was carried out before the tunnel excavation, but additional mitigation measures were needed in order to further reduce tunnel-induced ground settlements. Jacked piles were eventually installed under the steel ribs and several piles were instrumented in order to investigate the evolution of the load transfer mechanism from the tunnel structure to the pile foundation during excavation phases. Analysis of the data clearly shows the pile load pattern, characterized by loading during excavation phases and evident unloading during pile installation in subsequent excavation sectors. This novel application of jacked piles as settlement reducers has been therefore documented and the relevant monitoring data used to better understand their possible positive contribution to prevention of tunnel induced ground settlement.

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