3-D Finite Element Modeling and Construction Aspects for vertical shafts in Metro C Rome
G. Furlani, G. Guiducci & A. Lucarelli
Studio Sintesi, Rimini, Italy
A. Carrettucci & R. Sorge
Metro C S.c.p.A., Roma, Italy

ABSTRACT: On October 2009 the two TBM used to excavate T5 Stretch of new Rome Metro C subway line reached the reception shaft 5.4, where the machines have been uplifted for maintenance activity. After a brief description of general geotechnical site conditions, this paper discusses the main geotechnical aspects of the shafts project and illustrates some of the numerical analyses that were carried out during design activities. Moreover the main construction solutions adopted have been described as well as the related monitoring controls carried out during construction.

1 INTRODUCTION

Two circular vertical shafts for the extraction of Metro C TBM have been excavated in Piazza S.Felice, Rome. The inner shaft diameter is 15.7 m while the distance between shafts axis is 21.3 m. In the final layout the two structures will be connected by technical rooms and will work as ventilation chimneys.

As reported in Figure 1, the maximum depth excavation is about 45 m from the ground level, and, as the external water table is approximately 24 m above the bottom of the excavation. The excavation is protected by 56 m deep diaphragm walls, which are 1.0 m thick.

The shafts are located in a densely urbanized area; therefore specific design choices have been adopted in order to minimize the disturbance on the surrounding soil and existing structures.

The circular diaphragm walls have been excavated using hydraulic cutter in order to obtain waterproof joints and good static collaboration between the elements. The inner soil below the shaft bottom was treated by cement injections with the aim to reduce ground permeability and control the water seepage from the bottom; this method has been preferred to jet grouting because of soil marked heterogeneity as cemented and lithoid layers alternates loose silty sandy layers. The bottom plug is shaped as an inverted arch in order to optimize the reaction to uplift (and save materials and money).

2 GEOTECHNICAL CONDITIONS

The subsoil of the area was explored by geotechnical site investigations; Near the shaft two 65 m long boring have been carried out

The ground surface is almost flat with an altitude of +47.5 m a.s.l. Below the fill R, the subsoil profile, as shown in Figure 1, consists of typical roman volcanic formations which are characterized by altered tuffs (VS, TL, TT, TA), black-red silty sand pyroclastic materials (PN, PR) and lithoid tuff (T1, T2). The design properties of the various geotechnical units are summarized in Table 1. The most challenging site condition is the high variability of soil cementing: different degrees of cementation have been found along the excavation, from unbounded to well-cemented sand (unit PN, TT, PR) up to fissured soft rock (unit PRb, TA, T1 and T2). Figure 2 and 3 show an example of the marked variability of volcanic materials found at various depths.

Table 1. Soil properties used in design.

<table>
<thead>
<tr>
<th>Units</th>
<th>γ</th>
<th>c'</th>
<th>φ'</th>
<th>Go</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>kN/m³</td>
<td>kPa</td>
<td>deg</td>
<td>MPa</td>
</tr>
<tr>
<td>VS</td>
<td>17.0</td>
<td>10</td>
<td>33</td>
<td>390</td>
</tr>
<tr>
<td>TL</td>
<td>17.0</td>
<td>25</td>
<td>35</td>
<td>840</td>
</tr>
<tr>
<td>TT</td>
<td>17.0</td>
<td>15</td>
<td>34</td>
<td>450</td>
</tr>
<tr>
<td>PN</td>
<td>19.0</td>
<td>10</td>
<td>35</td>
<td>480</td>
</tr>
<tr>
<td>PR</td>
<td>18.0</td>
<td>10</td>
<td>35</td>
<td>570</td>
</tr>
<tr>
<td>PRb</td>
<td>20.0</td>
<td>300</td>
<td>35</td>
<td>660</td>
</tr>
<tr>
<td>TA</td>
<td>17.5</td>
<td>30</td>
<td>35</td>
<td>300</td>
</tr>
<tr>
<td>T1-T2</td>
<td>17.5</td>
<td>40-300</td>
<td>35</td>
<td>930</td>
</tr>
</tbody>
</table>

Piezometric measurements have shown that the water table maximum level is +26.5 m a.s.l., which means about 21 m from the ground level.
Permeability distribution is quite complex and in situ Lefranc tests data have shown great dispersion. In the cemented tufs the permeability is linked to the type and distribution of fractures, in the silty sand and altered tuffs it depends on the clayey content (see, Figure 1, 2 and 3). The total available experimental data and experiences in similar conditions leads to a medium-high permeability values (average $10^{-5}$ with minimum of $10^{-4}$ m/s).

![Figure 1. Vertical section: shafts geometry and soil profile.](image1)

![Figure 2. Red-pyroclastic soil (PR) from 25 m to 30 m depth.](image2)

![Figure 3. Cemented tuff from 55 m to 60 m depth.](image3)

### 3 THREE DIMENSIONAL ANALYSIS

#### 3.1 Critical aspects and objectives

The behavior of the structures and the effects of the excavation have been analyzed using a numerical finite element model. Since the two dimensional modeling (plane or axial-symmetry condition) is inadequate for the markedly three-dimensional geometry of the problem, a complete 3D fem model has been used to investigate all critical aspects of soil-structure interaction and construction phases and in particular:

- mutual interaction between the two shafts (the inter-axis distance is about 21 m and the minimum spacing are about 5 m);
- additional effects of asymmetrical earth pressure (i.e. treatment of the soil around the hole of TBM, loads of the surrounding buildings foundation);
- redistribution of the vertical and circumferential stresses on the wall after the local demolition to allow TBM entrance.
- analysis of seismic conditions using pseudo-static method.

The structural shell is made of adjacent panels without reinforcement passing through the vertical joints. For this reason, design prescription has fixed to to 4 MPa the maximum horizontal compressive stresses, calculated on the contact area of the joint reduced by the error of vertical construction of the diaphragms.
3.2 The numerical models

In order to predict the interaction between the shafts and the surrounding soil, a 3D finite element analysis has been performed, using the commercial code “Plaxis 3D Foundation”. Numerical model section dimensions are 150 m x 120 m x 180 m (L x B x H). Horizontal displacements on model’s vertical boundaries have been restrained, while all displacements have been restrained on the bottom boundary. The structures have been modeled with plate elements (vertical diaphragm walls) connected to the surrounding soil by interface elements having a frictional strength that is 66% of soil strength. Solid elements have been used to model the inner annular beams contrast, which have been activated during the excavation. All structural element have been modeled using linear elastic property of the concrete.

Varied mesh size have been applied to soil layers and structures of the shafts to save computation time. Mesh has been refined inside and around the shafts and then expanded by going to the boundary. The total number of elements is about 28000, with about 80600 nodes. A fully meshed Plaxis 3d is showed in Figure 4.

Figure 5 shows a close up on the structural plate and annular contrast beam. To ease the reading of the output solicitations, a low stiffness beam element has been inserted in the middle of the cluster representing shafts’ structures. Axial and Flexural stiffness of the beam element is about 1/1000 of panel’s stiffness in order to avoid excessive increase of total stiffness. Therefore the output internal forces of these elements had to be multiplied x 1·10^{-3}.

Figure 6. Meshed 2d Plaxis model (axial-symmetric).

The Hardening Soil small model’ for ‘Hardening Soil model with small-strain stiffness (Benz et al., 2009), has been selected for design. This constitutive model is a powerful tool especially for excavation problems. The main design geotechnical parameters are shown in Table 3.

The model involves frictional hardening characteristics to model plastic shear strain in deviatoric loading, and cap hardening characteristics to model plastic volumetric strain in primary compression. Failure is defined by means of the Mohr-Coulomb failure criterion (strength parameters φ’, c’).
Moreover, this model incorporates strain dependent stiffness moduli, simulating the different reaction of soils to small strains and large strains. Soil stiffness is therefore modeled with “large strain parameter” (Poisson modulus $\nu$, $E_{ur}$, $E_{50}$, $E_{oed}$, that describe the unloading–reloading stiffness and primary compression stiffness) and low strain parameters ($G_0$ and $\gamma_{0.7}$, that are shear modulus at small strains and the shear strain at which the secant shear modulus value is reduced to 70% of its initial value). The $\gamma_{0.7}$ was estimated following the suggestion in the Plaxis manual as:

$$\gamma_{0.7} = \frac{1}{9} \cdot \frac{G_o}{\nu} \left[2 \cdot c \left(1 + \cos 2\phi'\right) - \sigma'_1 \left(1 + ko\right) \cdot \sin 2\phi'\right]$$

Where $ko$ is the coefficient of lateral earth pressure at rest and $\sigma'_1$ is the effective vertical stress. Therefore the operative shear modulus $G$ is a function of shear strain $\gamma$ and stress level by the following expression:

$$G = \frac{G_o}{1 + a \cdot \gamma / \gamma_{0.7}}$$

Ssmall strains shear modulus at ($G_0$) has been calculated using shear wave velocity measures ($G_0=\rho \cdot V_s^2$). As said before, the HSsmall also requires the definition of the unloading/reloading Young’s modulus ($E_{ur}$). In this case $E_{ur}$ has been set equal to 1/5 of small strain Young modulus ($E_0\approx2.6 \cdot G_0$), that is a value that in previous experiences of the Authors has given good accordance between analyses and site monitoring measures. Finally, as suggested in the PLAXIS manual, $E_{50}=E_{ur}/3$ and $E_{oed}\approx E_{50}$ have been set.

The analysis have simulated the main stages of construction and excavation:
- geostatic initialization; activation of all the plate elements (diaphragms) and their interfaces;
- Simulation of soil improvement, modifying the geotechnical parameters in the volume of soil affected by the treatment (TBM breakthrough and bottom barrier for the seepage control);
- sequential excavation phases up to +7 m a.s.l. with construction of the two beams annular contrast at +21 m a.s.l. and +7.6 a.s.l. (above and under the tunnel crossing);
- demolition of the two openings for the TBM;
- excavation to maximum depth (+2.8 m a.s.l.).

3.3 Main results of the analysis

Figure 8 shows the 3D deformation of retaining walls after demolition of openings TBM (amplified 1500 times). The maximum displacement is about 2.4 mm; the deformed mesh highlights that the most stressed panel is located to the sides of the opening, due to the lack of mutual lateral support following the demolitions. It is also evident the confinement effect
offered by the annular beams, placed immediately above and below the TBM aperture.

The demolition of the opening leads to a redistribution of vertical and horizontal stresses with compression lines which tend to concentrate on the sides, above and below the hole. The following Figures 9 and 10 show the distribution of the vertical bending moment (M) and horizontal axial stresses (H), respectively. Figure 11 shows the axial stress obtained in the most stressed annular beam, which are all in compression.

As discussed above, the maximum vertical moment is reached in the panels located on the sides of the holes (about 300 kN/m, medium value determined on the width of the panel); therefore for these zones an integrative reinforced steel gage has been adopted.

The maximum horizontal compression is approximately 4000 kN/m, and is reached at the base of the TBM hole where the annular contrast beam is located. The stress distribution decreases far away from the openings where the distribution tends to be uniform. The maximum values reached in the diaphragm walls below the contrast beam is about 2800-3000 kN/m, in perfect agreement with the value obtained with the 2D axial-symmetric model (=2820 kN/m, reached around the bottom excavation).

In order to limit the stress on the contact vertical joints between the panels (<4.0 MPa), an executive vertical tolerance < 0.3% has been prescribed.

4 CONSTRUCTION ASPECTS AND MONITORING

4.1 Diaphragm walls

The circular diaphragm walls have been excavated using hydraulic cutter in order to obtain a good control of the verticality, waterproof joints and a good static collaboration between the panels forming eigh-
teen panels (nine primary and nine secondary panels) located with an inner diameter of 13.7 m. Each panel has been excavated with a nominal width of 2.8 m and an overlap of 30-380 mm between the primary and secondary panel (see Figure 12). This allowed a better control of panel verticality. Maximum deviations have therefore been very small (less than 0.1%, see Figure 13).

Figure 12. Plan disposition of the diaphragm panels.

Figure 13. Internal view after excavation.

4.2 Groundwater control

One of the most critical aspects of the project concerns the control of groundwater seepage from the base of the excavation, powered by a hydraulic head of about +24 m. Permeability distribution is quite complex and in situ Lefranc tests have shown great data dispersion. As it was expected, water flow analysis have shown that the water seepage phenomena is governed by the vertical permeability of soil located below the bottom of excavation: in the cemented tuffs (T2) the permeability is linked to the type and distribution of fractures, in the altered tuffs (TA) it depends on the clayey content (see, Figure 1, 2 and 3). The total available experimental data and experiences in similar conditions leads to a medium-high permeability values (average $10^{-5}$ with minimum of $10^{-4}$ m/s).

In order to reduce soil permeability and control the vertical water seepage from the bottom, the inner soil below the shaft bottom was treated by cement injections. The treated volume is 7 m thick and shaped like inverted arch in order to optimize the uplift global resistance (see Figure 1).

The adopted technique, named MPSP (Multi Packer Sleeveed Pipe, see also Bruce et al., 2009), owes much to the principle of the classical manchette system. Also in this case grout is injected in the surrounding rock through the ports of a plastic tube placed in a predrilled hole, but no sleeved grout is used.

The sleeved pvc tube (diameter 50 mm) has been retained and centered in each borehole (diameter 100 mm) by two collars-fabric bags inflated in situ with cement grout. These collars has been placed along each grout pipe in order to isolate two grouting stages, 3.5 m long each (see Figure 14). The holes are located on the vertices of an equilateral triangle mesh with 2.2 m long side (primaries holes). Secondary holes are located in intermediate position to intensify the treatment. In final layout hole spacing is 1.1 m.

Grouting has been executed in standard “tube a machette” procedure, starting from the bottom part via the double packer, using stable cement-bentonite mixes of water cement ratio 1 (by weight) and 2% bentonite. The grouting parameters have been chosen to respect target volumes (maximum 250-300 lt/m to prevent potentially waste of grout) and target pressures (maximum 25-30 bar).

The inside pumping tests performed before starting the excavations have allowed an experimental verification of the executed treatment. The tests have been performed pumping water from inside the shafts with continuous measurement of flow discharge and levels of internal and external electric piezometers. Two tests were carried out working separately on the two shafts; similar results have been obtained in both cases. In the graph of Figure 15 are showed the experimental data measured during the test conducted into the shaft B.D.. As shown in the figure the test is divided into two phases:
– in the first phase the internal pumping was activated in order to empty the shaft and maintain the internal water level constant around +0.0 m a.s.l. until stabilization of the discharge flow; the balance was achieved after about 500 hours by pumping approximately 0.3 l/s (same value for both wells);

– the second phase is a recovery test without pumping: there was a recovery of only 65-70% of the initial hydraulic head after 200-300 hours.

The external piezometers have shown a negligible drop (less than 0.1 m).

Assuming mono-dimensional seepage, the back-analysis lead to a equivalent permeability of the injected volume of $4 \times 10^{-7}$ m/s, which is a drastic reduction of the values of natural soils (average $10^{-5}$ m/s, with maximum of $10^{-4}$ m/s). The small discharge measured (25-30 m$^3$/day) was confirmed during the excavation, without perceptible effect on the external water table.

4.3 Monitoring system

A wide monitoring activity has been carried out in order to verify design assumptions, to optimize and adjust construction process and to control the stability and serviceability of supporting structures.

For these reasons, the monitoring system dealt with many aspects, and mainly:

– Water-table monitoring, by means piezometers installed outside and inside excavation (Casagrande and electric cells respectively): no relevant effects was observed on the external piezometers during shaft excavation (see also paragraph 4.2); a drop around -4.5 m was measured during TBM reception phase, followed by immediate recovery (see Figure 16).

– Diaphragm horizontal displacements monitoring, by means of inclinometers. As shown in Figure 17, the values measured are negligible and comparable to the instrument accuracy.

– Building movement monitoring, by means of automated total stations. The history of vertical settlements measured for the nearest building (that is about 10 m far from the shaft) is shown in Figure 18: the totality of the displacements (≈10 mm) were measured during ground improvement works that were needed for the TBM reception phase; no relevant movements was observed during shaft excavation and reception phases.
Analyses have outlined the most stressed panels and stress redistribution around TBM holes in the shafts. One of the most critical aspects of the project concerns the control of groundwater seepage from the base of the excavation. In order to reduce soil permeability, the inner soil below the shaft bottom was treated by cement injections. The adopted technique, named MPSP (Multi Packer Sleeved Pipe), led to a drastic reduction of the permeability of the natural soil: a small water discharge has been pumped during the excavation, without perceptible effect on the external water table.

A wide monitoring activity has been carried out in order to verify the behavior of the soil-structure system during construction and also to verify the design assumptions adopted. As predicted by design analyses, no relevant effects were observed during excavation of the shaft. The major effects observed were due to TBM work activities, ground improvement and reception phases.

6 REFERENCES


5 REMARKS

The design, construction and monitoring of 5.4 shaft of new Rome Metro C subway line have been described in this note.
Design analyses, needed to investigate all critical aspects of soil-structure interaction and construction phases, have been performed using a complete 3D Finite Element Model, since plane strain hypothesis is inadequate for the markedly three-dimensional geometry of the problem.