Deep excavation of Malatesta Station in Rome: Design, construction and measures

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ABSTRACT: Malatesta station is one of the new Rome urban subway line stations. Its construction required deep excavations in urban area, under high water pressures.

Site stratigraphy is characterized by the superimposition of pyroclastic soils, having different cementation degrees, and Tevere’s alluvional soils. The stratigraphic column base is composed by Monte Vaticano Pliocene Clays and Pleistocene marine Clays, which are typical Rome bedrock.

The excavation depth, the presence of nearby buildings and the high water thrust obliged to use top-down excavation method and to install a controlled dewatering system.

Numerical analysis using FEM were performed and the computation results were compared with the corresponding field measures results. A wide monitoring activity has been carried out: settlement control has been successful and wall movements have been typically small to moderate.

1 INTRODUCTION

The new urban subway line in Rome is called Metro Line C and it is realized by General Contractor Metro C S.c.p.a. This subway line connects the south-east suburb area to north-west suburb area and it has been developed with regards to the historic and urban importance of the area it crosses.

The Malatesta Station is located in a highly urbanized area along the Stretch T4, only few hundred meters far from the homonymous starting shaft of tunnel boring machines (TBM Malatesta Shaft). For this reason, the geometry of the station box has been performed both contemplating the “void” crossing and “full” crossing of tunnel boring machines.

The station has been constructed with top-down construction method and excavations have been realized between diaphragm walls. The geometrical characteristics of main structural elements are summarized in Table 1.

2 GROUND CONDITIONS

The ground conditions at Malatesta Station of Line C from the various phases of ground investigations are presented in section in Figure 1.

Table 1. Geometrical characteristics of main structural elements.

<table>
<thead>
<tr>
<th>Element</th>
<th>Dimensions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral Diaphragm walls</td>
<td>280 cm × 100 cm</td>
</tr>
<tr>
<td>Central Diaphragm walls</td>
<td>250 cm × 80 cm</td>
</tr>
<tr>
<td>Roof slab level (70 cm thick)</td>
<td>+30.40; +31.20 m ASL</td>
</tr>
<tr>
<td>Concourse level (50 cm thick)</td>
<td>+24.10; +24.60 m ASL</td>
</tr>
<tr>
<td>Ticket hall level (50 cm thick)</td>
<td>+20.00 m ASL</td>
</tr>
<tr>
<td>Base slab level (150 cm thick)</td>
<td>+11.90 m ASL</td>
</tr>
</tbody>
</table>

They indicate that the ground generally comprises made ground (R), composed by sandy–silty matrix mainly of pyroclastic nature up to 6 m thick, over recent alluvional deposits (LSO), which are predominantly silty-sandy material with organic content either dispersed or present in lenses, that have total thickness of 9 m (Fosso dell’Acqua Bullicante).

Above these strata, the Inferior Tuff Complex consists of a stratification of granular sized non cohesive pyroclastic material to high cementation layers (T1 and T2) and fine ash sized to granular sized pyroclastic material with very stiff layers and layers of yellow Tuff of variable cementation (TA).

At the bottom of pyroclastic strata, there is a sedimentary fluvio-lacustrine pre-volcanic complex...
(PaleoTevere Unit 2), including these geological units:

− clayey sandy silt (STa) with carbonate concretions and localised levels of travertine concretion, up to 7–12 m thick;
− medium to coarse sand with gravel and lenses of silty sand and clayey and sandy silt (SG) with maximum thickness of 7 m.

The bedrock of the Rome area is composed by the Monte Vaticano Pliocene clays and the Pleistocene marine (APL). At Malatesta Station, the top level of APL layer has been defined with some boreholes (S4-19, S4-20 and S4-21) at the level of −30 m ASL.

Only a brief summary of geotechnical parameters relating to the station design are presented below.

The design parameters have considered all the available data, including results from in-situ tests (CPT, SPT, Lefranc permeability tests and RE.MI seismic tests) as well as laboratory tests (including particle size distribution tests, Atterberg limit tests, unconsolidated undrained triaxial tests and quick undrained tests).

The observations about the groundwater level have considered all the available data, including the ancient ground water control (May 2000-Feb. 2001 and Gen-May 2002) and the systematic measurements started after the ground investigation campaign in 2006–2007, with installation of new piezometers.

The ground water measurements confirm the existing piezometric data along the stretch T4 and suggest that the water level in the area of Malatesta Station is at +26 m to +27 m ASL.

The Casagrande Cells located in sand and gravel layer (SG) indicate the water level at +21 m to +21.5 m ASL.

### 3 STATION EXCAVATIONS

In order to reduce the interferences with the surface road network and minimize soil movements, the station was built with top-down construction method.

Figures 3 through 4 show construction photographs from two top-down excavation phases.

The sequence construction begins with the realization of diaphragm walls, 100 cm thick, along the box perimeter with length of 119 m and width of 33 m. The diaphragm walls have a total length of

<table>
<thead>
<tr>
<th>Layer</th>
<th>γ (kN/m³)</th>
<th>Φ (°)</th>
<th>c’ (kPa)</th>
<th>c_u (kPa)</th>
<th>K (m/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>R</td>
<td>17</td>
<td>28</td>
<td>10</td>
<td></td>
<td>1 E-5</td>
</tr>
<tr>
<td>LSO</td>
<td>17</td>
<td>30</td>
<td>20</td>
<td></td>
<td>1 E-5</td>
</tr>
<tr>
<td>TA</td>
<td>17</td>
<td>32</td>
<td>30</td>
<td>60</td>
<td>1 E-6</td>
</tr>
<tr>
<td>T1/T2</td>
<td>17</td>
<td>35</td>
<td>25–40</td>
<td></td>
<td>3 E-5</td>
</tr>
<tr>
<td>STa</td>
<td>18.5</td>
<td>24</td>
<td>25</td>
<td>230</td>
<td>1 E-9</td>
</tr>
<tr>
<td>SG</td>
<td>19</td>
<td>35</td>
<td>5</td>
<td></td>
<td>1 E-5</td>
</tr>
</tbody>
</table>

Figure 1. Cross-section.

Figure 2. Plan.

Figure 3. Roof opening and first floor slab.
36 m and they extend more than 10 m into clayey layers of the almost impermeable stratum called STA, providing both the temporary and permanent earth pressure support for excavation. Then it is realized a double alignment of central diaphragm walls, 80 cm thick, with the aim to support the intermediate slab floors during provisional phase.

This is followed by excavation to just below the roof slab level of the underground structure, with the retaining walls supporting the soil at the sides. The roof slab is then constructed and connected to lateral diaphragm walls, providing a massive support across the excavation. Access openings on the roof slab are provided so that works thereafter could proceed downwards to the base slab level of the underground structure and the site area could reduce to the central part of Malatesta Square.

The depth of diaphragm walls was designed such that the full water pressure on the bottom of the soil between the diaphragm walls could be resisted by the weight of the soil (uplift), however the dewatering wells have to be installed in order to facilitate the operations of works machines.

A number of 6 drilled wells with diameter of 400 mm are installed along the double alignment of central diaphragm walls; they reach a depth of 6.5 m below the formation level, providing safety conditions to place the base slab, and during maximum digging they pump 7.22 l/sec.

Upon completion of second floor slab, provisional steel struts (see Fig. 5) are installed at level of +15.00 m ASL, providing safety conditions to reach the final excavation level, and a different procedure for excavating is adopted. Longitudinal sectors, 12 m width and 5 m deep, have been excavated with progressive construction of the base slab.

The advantage of trench sectioning is it allows to avoid uplift and instability of formation level before base slab placing.

Actually all these phases are fully completed, the base slab is placed and the intermediate steel struts are removed. Dewatering system is still active and it will be turned off when the interior lining will be cast. Figure 6 shows a construction photograph of Breakthrough TBM.

4 MONITORING SYSTEM

A wide monitoring activity has been carried out in order to verify the stability of supporting structures during construction and also the design assumptions to allow adjustment and optimization of the constructed processes.

For these reasons, the monitoring system has interested different topics:

- Water-table monitoring, by means piezometers installed outside the excavation (Casagrande cells) and inside excavation (electric piezometers under final excavation level);
- Ground movement monitoring, by means of several in-place inclinometers, multi-base assessimeters and precise levelling points;
- Building movement monitoring, by means of automated total stations to give real-time three dimensional displacements and remotely logged biaxial electrolevel tiltmeters;
- Structures monitoring, by means of inclinometers for measurement of horizontal displacements,
extensometers for the determination of the stress-strain of the structures and strain gauges load cells on steel struts.

The following Figure 7 shows the position of control sections (named section 1 and 2) and two inclinometers inside the diaphragm elements for horizontal displacement control (named IN03P and IN04P).

The Figure 8 shows the instrumented section 1. The monitoring design has fixed threshold and limiting values for all the used instruments, which may trigger responses ranging from more frequent readings to modifications of construction procedures and, in the extreme, to implementation of a specified contingency plan.

The Figure 9 shows water levels measured during excavation phases. The external piezometers (Casagrande cells) have been placed around the initial external water table (= +25 m ASL) with a drop around 1 m up to 2–3 m for the shallowest piezometers. The internal piezometers (electric type) have measured a level around final excavation level, thus with a pseudo-hydrostatic gradient.

The Figure 10 shows load profile of steel struts, measured by strain gauges load cells.

5 NUMERICAL ANALYSIS

5.1 General aspects and objectives

Several numerical analysis, using finite elements method, have been performed in order to model the behavior of the supporting structures and the effects of the excavation.

Initially, a simplified and conservative 3D-FEM model ("elasto-plastic springs") has been used to design the structural elements. Then, more complex 2D and 3D-FEM models have been performed in order to analyze the detailed aspects:

- verify predicted effects induced in the neighboring buildings and understand the observed behavior of the supporting structures and ground movement;
- sensitivity analysis of the results about geotechnical parameters variability and different hydraulic conditions;
- stability analysis;
- analysis of three-dimensional excavation of the last step (from +15 m ASL to +10 m ASL), that has been described before, and consists of trench sectioning.

The last aspect has requested a full three-dimensional FEM model in order to evaluate the advantage due to the excavation procedure adopted.
5.2 3D-FEM model: description and main results

The 3D finite element analysis has been performed using Plaxis 3D Foundation. A soil model section of 260 m $\times$ 180 m $\times$ 40 m (L $\times$ B $\times$ H) has been adopted. 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 (horizontal floor slabs and vertical diaphragm walls) connected to the surrounding soil by interface elements having a frictional strength that is 2/3 of soil strength. All structural elements have been modeled using linear elastic property of the concrete.

Variable mesh size has been applied to soil layers and structures of the station to save computation time. Mesh has been refined inside and around the excavation volume and then expanded by going to the boundary. The total number of triangular elements is about 11000, with about 33000 nodes. A fully meshed Plaxis 3D is showed in Figure 11.

The Hardening Soil small model, abbreviated as HSS small model, has been selected for design. This constitutive model is a powerful tool especially for excavation problems. The main design geotechnical parameters are shows 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 $\phi'$, $c'$).

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’s ratio $\nu; E_{ur}, E_{50}, E_{oed}$ 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 $\gamma_{0.7}$ has been estimated following the suggestion in the Plaxis manual as:

$$\gamma_{0.7} = \frac{1}{9} \cdot \frac{G_0}{\sigma_1'} \cdot \left[ 2 \cdot c' \cdot (1 + \cos 2\phi') - \sigma_1' \cdot (1 + k_o) \cdot \sin 2\phi' \right]$$

(1)

where $k_o$ 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:

$$G = \frac{G_0}{\left(1 + a \frac{\gamma}{\gamma_{0.7}} \right)}$$

(2)

Table 3. Main input parameters of HS small model.

<table>
<thead>
<tr>
<th>Layer</th>
<th>$\gamma$ (kN/m$^3$)</th>
<th>$\phi'$ (°)</th>
<th>$c'$ (kPa)</th>
<th>$G_0$ (MPa)</th>
<th>$E_{ur}$ (MPa)</th>
<th>$E_{50}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>R</td>
<td>17</td>
<td>28</td>
<td>10</td>
<td>85</td>
<td>45</td>
<td>15</td>
</tr>
<tr>
<td>LSO</td>
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<td>30</td>
<td>20</td>
<td>85</td>
<td>45</td>
<td>15</td>
</tr>
<tr>
<td>TA</td>
<td>17</td>
<td>32</td>
<td>30</td>
<td>280</td>
<td>145</td>
<td>48</td>
</tr>
<tr>
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<td>STA</td>
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<td>24</td>
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<td>SG</td>
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<td>35</td>
<td>5</td>
<td>750</td>
<td>390</td>
<td>130</td>
</tr>
</tbody>
</table>
The shear modulus at small strains ($G_0$) has been calculated using shear wave velocity measures ($G_0 = \rho \cdot V_s^2$). As said before, the HS small model 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’s modulus ($E_0 = 2.6 \cdot G_0$), 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, has been set $E_{50} = E_{ur}/3$ and $E_{oed} \approx E_{50}$.

The analysis have simulated the main stages of construction and excavation.

- Geostatic initialization using the ko procedure; activation of all the plate elements (diaphragms) and their interfaces;
- Sequential excavation phases up to +15 m ASL (subway platform level) with construction of the internal contrast floor at levels +31.0 m (roof slab), +24.1 m and +20.0 m ASL.
- Installation of provisional steel struts (+15 m ASL) and excavation up to formation level (+10 m ASL) proceeding by through partial longitudinal sectors, 12 m width; this excavation phase has been divided into 9 consecutive steps. Figure 12 shows the configuration of the model in an intermediate excavation stage. The model also considers the stabilizing effect of the base slab that is executed before of the next excavation stage.
- Completion of the excavation (and base slab) and removal of the provisional steel struts.

The FEM model led to quantify the stabilizing confinement effect due to the methodology excavation through partial longitudinal sectors. An medium increase of about 50% is observed for the effective vertical stress acting along the diaphragm wall below the final excavation level, where acting the passive resistance (Figure 13). Therefore, in order to take into account this phenomena in the simplified 1d-FEM analysis, an equivalent surcharge has been applied on the final excavation level during the last phase.

Figure 14 shows the 3D deformation of retaining walls after completion of excavation and base slab construction.

Figure 15 shows the distribution of the vertical bending moment (M): the maximum value is ±800 kNm/m, in good agreement with the simplified 1D and 2D FEM models.

As shown in Figure 16, the predicted ground settlements fit the field measurements quite well for instrumented section 1, but higher values were measured for section 2 without any damage.

Figure 17 shows the comparison of measured wall horizontal displacements and those predicted using the 3D FEM model. Measures and numerical analyses results agree well in the upper part of the wall, while in the lower (inside ground) part of the wall FEM analyses are still conservative and predict horizontal displacements are higher than the measured ones.

Figure 12. Intermediate longitudinal stage excavation (the view of internal structural floor is disabled).  
Figure 13. Effective vertical stress below final excavation level.
6 CONCLUSIONS AND MONITORING CONSIDERATIONS

Monitoring measures have been obtained over all the period of construction works, highlighting that:

− The perimeter diaphragm walls, embedded into a stratum with low permeability, allows a good separation between water level outside the station box (not significantly affected either by the excavation or by the dewatering process) and water level inside the station box (almost coincident with final excavation level);
− The measured deformations of the supporting structures are less than the predicted deformations, with shear state correlated to allowable stress.

It has to be underlined that building monitoring showed maximum settlement of 6+10 mm and maximum horizontal displacements of 10+13 mm, without any damage. During monitoring activity it has been observed a high dependence of measures with temperature variation, both in short time and seasonal fluctuation. Therefore measures have been corrected by the influence of external temperature.

Also in the case of Malatesta Station the good monitoring program and the critical comparison of monitoring measures with design predictions provided key information that lead to a safe and successful construction.
REFERENCES


