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Lining structural monitoring in the new underground service of Naples (Italy)

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ABSTRACT

A new underground stretch connecting two existing lines of the urban railway underground system was built in Naples. The tunnel was excavated by NATM through unsaturated pyroclastic silty sand at relatively large depths, ranging between thirty to eighty metres. The geometry of the section is rather typical with a top part shaped as half a circle and a rectangular bottom. The tunnel was conventionally built with a staged construction: after the full face excavation, a temporary lining composed by horse-shoe steel ribs and sprayed concrete was put in place. Two steel ribs, spaced about one hundred metres apart and located about sixty metres below the ground surface, were instrumented with twenty-four vibrating wire strain gauges, installed at six symmetric locations. Nearly two years of measurements were collected. In the paper the measured strains and the back-calculated internal forces are reported. The results of FE 3D and 2D back-analyses are also shown allowing to throw a light on the observed behaviour.

1. Introduction

A tunnelling method in which excavation and support procedures can be continuously adapted to the encountered conditions, depending on the observed deformation, is nowadays widely defined as New Austrian Tunnelling Method (NATM). This is a somewhat generic definition suitable for any tunnelling procedure which does not use automated machines. The design of tunnels excavated using NATM is a rather challenging operation. In such a case the stability of the face and of the whole cavity must be ensured relying upon the available natural soil strength or methods to improve it. Generally the lining of tunnels excavated via NATM are installed at a convenient distance from the front determined at the design stage as the best compromise between an adequate relaxation of the in situ soil stress and a sufficient safety factor against collapse of the tunnel heading. The adopted compromise plays a major role in determining the stress level in the lining. Furthermore minor details of the construction procedure adopted on site are often not perfectly defined at the design stage. These details may sometimes be not negligible mainly because influencing the strain and the stress level in the temporary lining. Monitoring activities carried out during construction provide evidence of this.

Standard equipment for monitoring NATM tunnels consists of optical systems aimed at recording displacement of a number of targets along several cross sections of the tunnel. In most cases contact stresses between the lining and the ground are not measured, since proper installation of the pressure cells is difficult and discrete measurements of contact stresses, non-continuous along the cross section, are often insufficient to calculate internal forces acting in the lining. However, monitored displacements can be used to estimate forces and stresses in the shotcrete lining of NATM tunnels to assess its degree of loading (Sercombe et al., 2000; Hellmich et al., 2001; Lackner et al., 2002; Ullah et al., 2010). Ullah et al. (2013) recently proposed a shell-theory based conversion of measured three-dimensional displacement vectors into in-plane stresses of shotcrete tunnel shells.

When steel ribs are used as principal structural support for the large section and thin shotcrete layer is only an integration measure to prevent minor and local soil mechanisms in silty sand, a straightforward alternative is the installation of strain gauges welded or bolted on the steel profile. Such measurements can be used to check the degree of loading of the temporary lining during tunnel construction, as shown in this work.

In the city of Naples a huge development of the underground railway network is going on since 10 years at least (Bilotta et al., 2006; Bilotta and Russo, 2012a,b; L'Amante et al., 2012; Russo et al., 2012). In the west part of the city of Naples, the Line 7 of the underground urban railways system is currently under

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List of symbols

$\sigma - u_a$	net stress
ψ	dilatancy angle in Hardening Soil model
λ	stress reduction factor
γ	unit weight of soil
ε, $\mu\varepsilon$	strain, microstrain
ϕ', ϕ	angle of shear resistance or friction angle (in terms of effective and total stresses)
σ', τ	effective normal and shear stresses
A	cross section area of the steel rib
A_t	area of the equivalent arch of reinforced soil
$c*$	cohesion of the homogenized canopy of pipes
c', c	cohesion (in terms of effective and total stresses)
e	void ratio
E	Young's modulus
$E_{50,\text{ref}}$, $E_{\text{oed,ref}}$, $E_{\text{ur,ref}}$	reference stiffness moduli in Hardening Soil model
E_{oed}	one-dimensional (oedometer) compression modulus
G_P	shear modulus from pressuremeter test (PMT)
I	second moment of inertia of the steel rib
K_0	earth's coefficient at rest
M	hoop force in the steel rib
M_l	longitudinal bending moment in the steel rib
M_t	transverse bending moment in the steel rib
$N_{\max}$	limiting tensile axial force (fiberglass pipes)
n_{pipes}	number of pipes
N_{SPT}	blows count in a Standard Penetration Test (SPT)
$p - u_a$	net mean stress
q	stress deviator
S	degree of saturation
W_x	elastic section modulus of the steel rib about the main strong axis
ν	Poisson's ratio
ν_{ur}	unloading–reloading Poisson's ratio in Hardening Soil model

construction. The line has a total length of 5.4 km with 4 new stations and represents a closing connection to an existing open ring.

The stretch between Parco San Paolo and Monte Sant'Angelo stations was built by using NATM. It is a single, rather large and deep tunnel where two railway tracks are lodged. The tunnel runs for a total length of approximately 750 m at a depth ranging between 35 m and 80 m crossing partially cemented and unsaturated pyroclastic sandy soils.

Two ribs of the provisional lining were heavily instrumented using vibrating wire gages for strain monitoring with the aim to finally obtain the internal forces in the lining (Marino, 2011). The monitoring data were manually logged during the construction stages following the erection of the steel ribs. Once installed the rib in the final position, automatic logging was set up and the data were regularly recorded and downloaded for a relatively long period of about two years. During this period, the final lining was installed by concreting around the temporary steel ribs, thus forming a larger composite steel–concrete structural section.

In the paper the measurements are first presented, showing and remarking the positive quality check carried out on the recorded data. In the second part of the paper the results of both 3D and 2D back analyses via FEM are presented, allowing interesting findings and comments on the monitoring data.

2. Underground works

As stated in the introduction this paper focus on a small stretch of the Line 7 of Napoli Underground Network and precisely the one comprised between the stations of Monte Sant'Angelo, close to facilities of the University of Napoli Federico II and the residential area of Parco San Paolo (see Fig. 1).

The tunnel runs at depths ranging between 35 and 80 m. The subsoil profile along the route is sketched in Fig. 2. Starting from the rather uneven ground surface a few metres of made ground and a relatively thick layer of silty sand are first found. The top of the pyroclastic sandy layer through which the tunnel is excavated is between 10 and 15 m deep. The thickness of this last layer is very large and such that the same soil layer is still found at depths larger than 1D below the invert of the tunnel section. The groundwater level is well below the invert of the tunnel.

In Fig. 3 a plan view of the stretch is superimposed to the urban map. Only a few buildings are underpassed by the tunnel being the area a mostly rural one and consequently not densely urbanised. Furthermore absolutely negligible effects on the ground surface and thus on the existing buildings were predicted at the design stage, due to the large cover above the tunnel. In this case the most relevant design issues are those related to the correct prediction of the stresses in the lining and of the external relaxation of the in situ stress due to the excavation and the consequent soil removal.

The excavation procedure as defined at the design stage is simply based on a full face heading relying upon the good quality of the surrounding soil mass and the apparent cohesion due to the suction present in the unsaturated pyroclastic soil above the groundwater table.

To increase the safety factor against face instability, 12 m long grouted fibreglass rods were installed in the top heading, according to the layout shown in the sketch of Fig. 4.

In longitudinal direction the nails had a relatively short superimposition length, generally ranging between 4 and 5 m.

A temporary support consisting of horse-shoe (10 m large and 8.25 m high) HEB160 steel ribs, a $\Phi 8/20 \times 20$ welded steel mesh and shotcrete about 10 cm thick was adopted during the excavation, the maximum unprotected length of excavation being equal to 1 m.

The reinforced concrete invert was generally cast in place about 25–30 m behind the front face. The side walls and the crown of the final reinforced concrete lining were cast in place generally between 40 and 50 m behind the front face. Consequently, only the temporary lining described above, consisting of steel ribs, welded steel mesh and shotcrete, was supporting the excavation heading while a relatively long stretch was fully excavated.

3. Site and laboratory investigations

As usual in such a deep tunnel, with the additional difficulty of the substantial inaccessibility of the ground surface along a large portion of the route, the investigations carried out at the design stage were not so accurate as it could be expected considering the importance of the whole contract. The subsoil profile shown in Fig. 2 was defined on the basis of the information from 3 deep boreholes carried out along the route together with further details recovered by existing site investigations nearby. Down along the boreholes several standard penetration tests and a few pressuremeter tests were executed to investigate the mechanical behaviour of the main soil layers.

In Fig. 5 the plot of the N_{SPT} blows count along the depth is reported. The values of N_{SPT} show a strong trend to increase with