Ground movements and strains in the lining of a tunnel in cohesionless soil

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ABSTRACT: A huge transport plan is being implemented in the city of Naples; in this framework, a new line extending eastwards the underground network is at present under construction. Its twin tunnels are being excavated by a couple of TBMs. A monitoring program has been set up, including several instrumented sections where surface settlements and deep movements are being observed using optical precision survey and down-hole instruments. The strains in a section of the pre-cast concrete lining are also being measured using embedded vibrating wire gauges. Instrumented segments of linings were calibrated before installation in tunnel: details on the instruments, on the installation techniques and on the calibration tests are reported in the paper. The results collected so far, their interpretation and some preliminary comments are also reported.

1 INTRODUCTION

Prediction of forces acting on tunnel linings is a rather complex task. They are the result of the interaction between soil and lining, which is influenced by the excavation procedures and by the details of lining installation. The lining structural behaviour also plays an important role, but is not easy to understand and model.

The analysis of the case histories reported in the literature shows that linings of the same type but with a wide range of thickness have been designed and constructed in similar ground conditions. This suggests that in many cases they have been over-dimensioned and that design could have been more efficient. Reducing uncertainties on prediction of loads acting on lining is therefore an important step towards a more efficient design, as it would make it easier to adopt less conservative and less arbitrary solutions: whoever has been engaged in the design of a tunnel is aware that sometimes the design decisions on parameters and models are hard to justify.

In recent years, our cities have widely experienced the construction of shallow tunnels, mainly aimed to develop railway transportation networks.

The excavation is usually performed by means of TBM (Tunnel Boring Machines) which allow for high advancement rates, around ten metres per day, and reduce the risk of damaging the above built environment.

The TBM itself installs pre-cast reinforced concrete segments to form a sequence of rings. This kind of lining permits economy of scale in production but, on the other hand, introduces complications in modelling when compared to the 'simple' monolithic cast-in-place linings. The segmental nature of lining is often neglected in design, thus achieving a conservative estimation of the loading level in the segments. However, recent studies (Lee *et al.*, 2001; Hefny *et al.*, 2004) show that the joints between segments produce a significant reduction in loading level, thus allowing savings in structural dimensions.

The Department of Geotechnical Engineering of the University of Napoli Federico II is at present engaged in a research on this topic, based on the monitoring of the works of the Line 1 of Naples subway. Among others monitoring activities, a lining ring has been instrumented and its behaviour is under observation.

2 INFLUENCE OF JOINTS ON THE LINING FLEXURAL STIFFNESS

One of the factors which noticeably affects the magnitude of loads arising in a tunnel lining is the presence of joints among the lining segments, which is often neglected in practice.

Muir Wood (1975) proposed to model a segmental ring as a monolithic ring having an equivalent reduced moment of inertia:

$$I_e = I_j + \left(\frac{4}{n}\right)^2 I \qquad (1)$$

where I_j is the moment of inertia of the joints, I the moment of inertia of the ring without joints, n the number of joints in the ring ($n > 4$).

Hefny *et al.* (2004) have recently published the results of numerical analyses showing that both the number and position of the joints between segments sensibly influence the magnitude of loads acting in the lining. Their analysis assumes uniformly spaced joints; this is obviously a limitation as most of the existing pre-cast segmental lining have a smaller key segment, which results in a non-uniform distribution of joints.

Lee *et al.* (2001) proposed to model the ring as a set of curved beams linked each other by rotational springs whose stiffness is K_θ [N · m/(rad · m)]. Lee & Gee (2001) analysed the possibility of introducing an equivalence between such a model and a monolithic ring characterised by a relative joint stiffness $\lambda = K_\theta l/EI$, where l is a calculation length usually taken as 1 m. According to the Authors, for most tunnels in soft ground the values of the parameter λ range between 0.03 and 0.3. This parameter significantly affects the ratio $\eta = EI_e/EI$, which represents a factor of reduction of bending moment: as λ decreases, both η and bending moment decrease. In the typical range of relative joint stiffness, λ, the factor η ranges between 0.1 and 0.6, thus allowing for very important reduction of bending moment. According to this finding, the Japanese Civil Engineering Society recommends preliminary values for η between 0.6 and 1, when loading tests on the whole segmented ring are not carried out.

3 EXPERIMENTAL WORK

The stretch Dante-Centro Direzionale of the extension of Line 1 of Napoli subway is to be constructed in a very densely urbanised area. Most of the running tunnels are completely excavated in the formation of Neapolitan Yellow Tuff. Only the upper part of the large station shafts and the final 400 m of the tunnels, are located in pyroclastic sandy soils and well below the water table. The risk of damage to buildings is mainly limited to such works. At the time of writing, surface and deep displacement measurements are being carried out in several transverse tunnel sections (Fig. 1). Strains in four rings of the pre-cast lining are being measured too. At the time of writing the data recorded in one ring only are available. The latter measurements are performed by means of vibrating wire strain gauges, embedded in the segments during concrete casting in plant.

Five strain gauges were embedded in each of the six segments of a ring (Fig. 2), thus allowing for 30 measurement points for each instrumented ring. Four strain gauges are installed with their axis perpendicular to the tunnel axis, while the fifth is parallel to the tunnel axis, and is used as a 'dummy' gauge.

In order to check that the strain gauges worked correctly after embedding and to calibrate the strain readings in terms of normal force, bending moment and shear acting on the segment, the single instrumented segments were tested before installation. In Figure 3 a segment can be seen inside the frame used for tests.

In Figure 4 a schematic plan view of the experimental layout with a segment in position to be tested is reported.

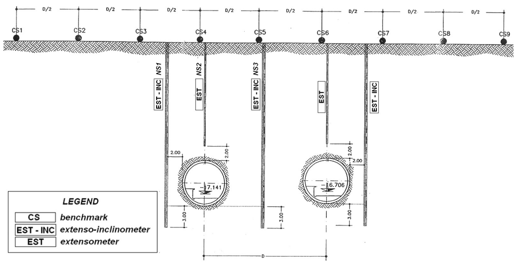

Figure 1. Typical layout of an instrumented cross section.

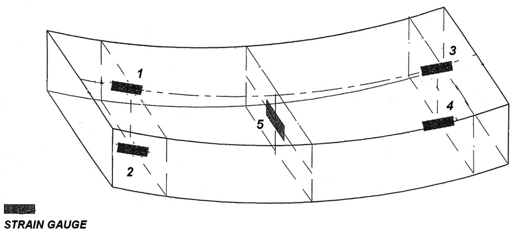

Figure 2. Location of the strain gauges in the instrumented segments.

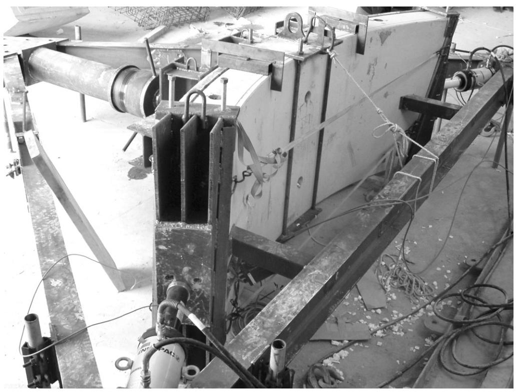

Figure 3. Loading test on a segment.

At the time of writing, calibration of 12 segments of two instrumented rings have been carried out. Each test was performed by applying two normal forces to the opposite side faces of a segment, by means of two hydraulic jacks. Such forces were balanced by two reactions on the convex surface of the segment. Loads were applied in steps (about 100 kN) up to a maximum of 700 kN. Sometimes the load was kept constant for about 1 h without observing significant creep. Due to the segment shape, by applying pure normal forces on the two opposite faces, both bending moment and thrust arise in the sections where the gauges are located.

It is worth noticing that all the installed gauges survived the concrete casting phase.

The results of three out of twelve tests are plotted in Figure 5.

Similar behaviour is exhibited by all the twelve tested segments.

Some minor differences can be justified by even small variations of the location and direction of the applied forces on the faces of the segment.

The behaviour appears noticeably linear on the side of the more compressed fibres, while a marked non-linearity arises on the other side. An explanation of this behaviour has not yet been found.

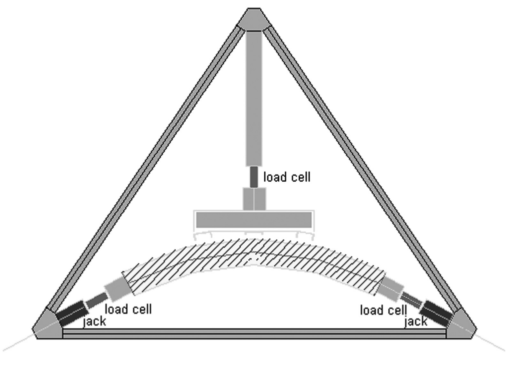

Figure 4. Schematic plan view of the experimental layout.

A further proof of the reliability of the measurements is provided by the 'dummy' gage (no. 5), which underwent a significant dilation by Poisson's effect, as it could be expected.

Subsequently all the tested segments have been installed in the tunnel. In the next section some measurements carried out in an instrumented ring are reported and commented.

4 SOME RESULTS

In the first stretch of the tunnel, crossing cohesionless soils, four instrumented 'greenfield' sections equipped with optical levelling targets, extenso-inclinometer tubes and instrumented lining rings, have been set up. Moreover, along the tunnel axis optical targets have been levelled with a frequency depending on the progression of works.

In Figure 6 the settlement measured along the transverse section 1 when the excavation front was well ahead of the instrumented section are reported. A Gaussian curve is fitted on the observed settlement trough imposing the coincidence on the maximum recorded settlement and searching for the distance of the inflection point i by means of a best fitting procedure. On the same diagram are also shown the settlement of the top of the extenso-inclinometer tubes, which were not used for curve-fitting.

The settlements plotted in Figure 6 were measured when the tunnel face was sufficiently far away from section 1, and no consolidation settlement was expected according to the soil type. Nevertheless, settlement measured in the same section at various following dates, showed some scatter. Therefore curve-fitting was repeated on a few series of data and average values of percentage trough volume $V_{T,\%}$ and $K = i/z_o$ were calculated. $V_{T,\%} = 0.47\%$ and $K = 0.38$ (tunnel axis depth $z_o = 14$ m) were hence determined. Such values are close to the expected ones for the kind of soil and excavation technology (O'Reilly & New, 1982; Oteo & Sagaseta, 1996; Bilotta *et al.*, 2002).

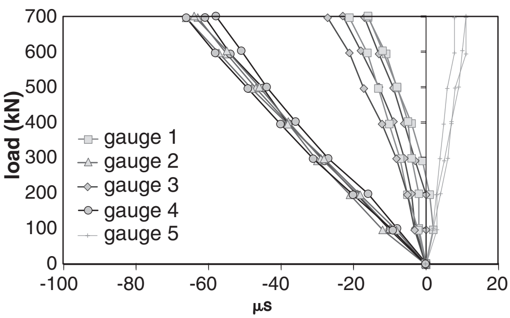

Figure 5. Load-deformation curves (tests 4, 6, 10).

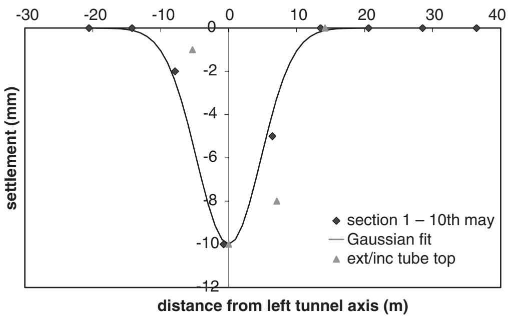

Figure 6. Measured settlement in section 1.

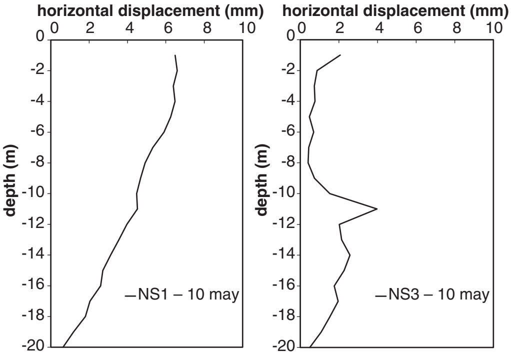

Figure 7. Horizontal displacements measured in section 1.

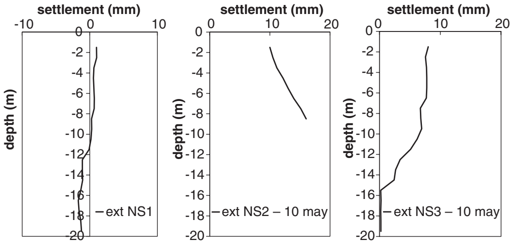

Figure 8. Deep settlement measured in section 1.

Section 1 was also equipped with inclinometers and extensometers, as shown in Figure 1. Horizontal displacements as measured along vertical inclinometer tubes NS1 and NS3 are reported in Figure 7. Vertical inclinometers NS1 and NS3 lay respectively at the left and at the right side of the southbound tunnel (left tunnel in Figure 1), which is the only one already excavated. The azimuthal values show that horizontal displacements along NS1 occurred substantially in the plane of section 1, whereas along NS3 a consistent out-of-plane component was observed in depth. When calculating horizontal displacements, it was assumed that the bottom of the inclinometer tubes, about 3 m below the tunnel invert, was fixed.

In Figure 8 settlement along vertical lines NS1, NS2 and NS3 as measured on the same date are shown. NS2 line corresponds very closely to the southbound tunnel axis. In this case, settlement of the tubes was elaborated assuming that the top of the tubes settled as reported in Figure 6.

An asymmetric field of deep settlements is evident from Figures 7–8. As the two vertical tubes NS1 and NS3 are almost symmetrical about the tunnel axis, the observed asymmetry remains to be investigated. In any case the settlements below the tunnel invert are negligible.

Between 23rd and 29th of April 2004 the TBM did not advance due to minor unforeseen operations. In this period the face of the tunnel, 99.3 m from the intake, was kept pressurised and measurements were repeated daily on the topographic targets L13 ÷ L23, all located in a greenfield area. In Figure 9 these settlements are plotted versus the distance of the target from the tunnel face. The dashed line is the longitudinal settlement profile obtained as a curve of cumulative normal probability in which the settlement at front is assumed equal to 0.5 w_{max} (Attewell & Woodman, 1982). The settlement w_{max}, in fact, should occur in plane strain conditions. The value of i which is necessary to define the normal probability curve has been chosen equal to the average value evaluated for the greenfield cross section 1, which is very close (89 m from the intake).

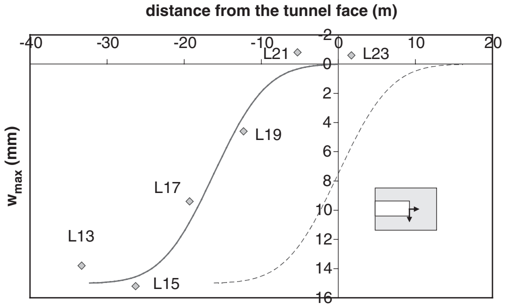

Figure 9. Longitudinal profile of settlement *vs* distance from face.

In granular soils is generally acknowledged that the main portion of the overall volume loss occurs at the shield tail, even if it is well known that its amount depends also on support techniques and grout injections in the gap between lining and ground. Craig (1975) suggested that in sand below the water table the ground loss at the face of the shield can range between 0% and 25% of the total, therefore it can make sense imposing equal to zero the settlement at front. In such a case, the computed settlement profile is translated back (continuous line in Figure 9) and it appears very close to measurements.

It is worth noticing that the ground surface settlements at L21 and L23 (i.e. close to the tunnel face) could also have been restrained by the stiffening effect of a building located about 10 m ahead.

The strains in the ring 117 of the lining were also automatically recorded.

First of all general trends are commented on the basis of the measurements obtained in one out of the six instrumented segments of the ring. In Figure 10 the strains in the segment D4 are plotted versus the time starting from the early stage of curing. Dilation is assumed as positive strain. The bold continuous line indicates the temperature as measured by a thermocouple embedded in the concrete close to one of the five strain gauges.

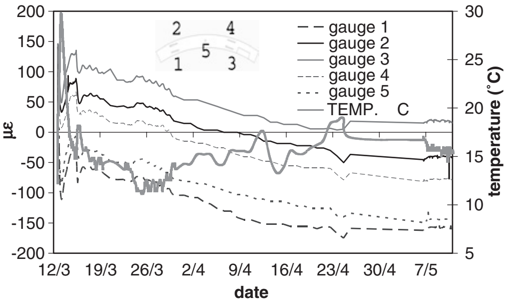

Figure 10. Measured strains in segment D4 during the curing stage.

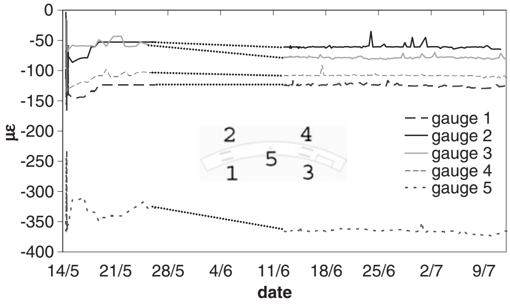

Figure 12. Measured strains in segment D4 after installation.

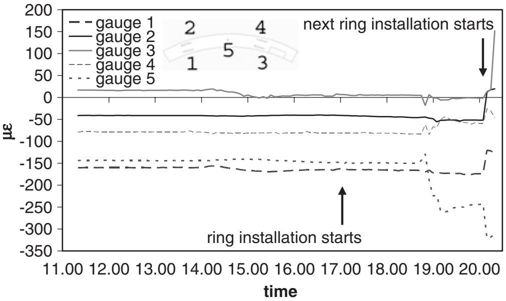

Figure 11. Measured strains in segment D4 during installation inside the shield tail.

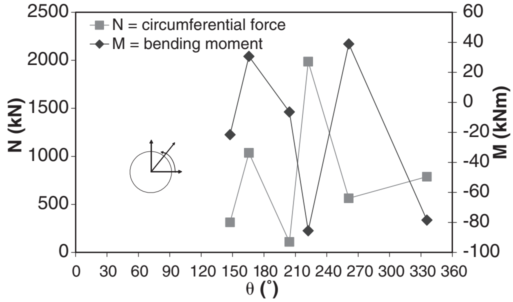

Figure 13. Circumferential force and bending moment in the ring.

In the chosen period a similar pattern of deformation is exhibited by all the gauges, apart from the very initial stage where random effects of the concreting predominate. Such a behaviour was expected as the observed strains were produced by the curing of the concrete and by rather uniform thermal changes.

The calibrations reported in Figure 5 were carried out at the end of this period.

In Figure 11 the strains measured during the complex installation procedure are reported. Typically the installation of a whole ring takes about three hours. At 19.00, the sudden increase in compression of the dummy gage no. 5, (i.e. the transversal one), indicates that the segment was erected and slightly compressed by the hydraulic jacks of the shield against the previous lining ring. After the ring was completed, at 20.00, the shield started to advance by pushing its jacks against the whole completed ring no. 117. Squeezed between the shield and the rest of the lining at its back, ring 117 deformed. Compression in the direction of gauge no. 5 and dilation by Poisson's effect on all the others gages is clearly shown by the last readings of the Figure 11.

Once the ring was installed and exposed out of the shield tail, the ground started to act on it. In Figure 12 the strains related to this step are then shown.

In the early stage strains changed quickly: the tunnel face advanced and the beneficial three-dimensional arching effect decreased, increasing the loads on the lining ring. Figure 12 shows that the strains stabilised about one week after the installation. Minor changes have occurred in the long term after a break in the observations due to bad functioning of the logging system.

Similar data have been collected and analysed for all the instrumented segments of the ring no. 117.

On the basis of the information gained by the calibration tests and of the long term strains measurements both circumferential force and bending moment in the ring have been deduced and plotted in Figure 13.

The values of the circumferential force and the bending moment were obtained using only the increments of strains measured after that the pushing stage was completed and the ring was fully exposed to the soil. This choice was made in order to highlight only the internal forces produced in the lining by the interaction with the surrounding ground. Unfortunately for three out of six segments these data were not available because it was not possible to get access to the loggers due to the TBM interference. That is the reason why for two out of the planned four instrumented rings radio wireless logger have been proposed. As it is shown in the plot of figure 13 the maximum value for the circumferential force is about 2000 kN while for the bending moment the maximum value is −80 kNm. This last value is rather low as it could be expected in a segmented lining. As a further preliminary comment the lack of symmetry is quite significant.

5 CONCLUDING REMARKS

The tunnelling works for the extension of Line 1 of Naples Underground have provided the chance to observe the behaviour of a segmental pre-cast lining embedded in a sandy soil below the groundwater table. Optical precision survey and down-hole instruments were also used to measure both surface and deep movements. Several instrumented cross sections have been planned.

Data coming from the first section have been presented and commented. The analysis of the observed behaviour is in progress and only some preliminary results have been presented in the paper. Curve-fitting of the settlement trough has been carried out to infer the parameters of the Gaussian curve: the obtained values confirm some of the most widely applied indications from the literature.

A calibration of the instrumented pre-cast segments of the concrete lining was carried out, allowing a more accurate calculation of the internal forces in the lining on the basis of strain measurements.

The strain measurements started from the curing stage at the manufacturer stock place and followed the main events until the instrumented ring was set in place in the tunnel. As shown with the support of the data of one out of the six instrumented segments every significant change in the strain was recorded and the main causes were identified. It is worth pointing out the fact that large values of strains occurred even before the ring was set in place in the tunnel.

Bending moments and circumferential forces in three out of six segments were calculated. As it could be expected in a segmented lining the maximum value of the bending moment was rather low.

A whole report of the data which are going to be collected in the other planned sections supported by a more refined and detailed interpretation will be published elsewhere.

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