Scale Laboratory Model for Studying the Behavior of Pipe Umbrella in Sandy Soil

D. Galetto, J.C.B.J. Silva, D. Peila, A. Assis

Abstract. Steel pipe umbrellas have been used to tunnel in difficult conditions or in weak rock masses and/or soils. Despite the large number of applications around the world, there are still some doubts concerning how pipe umbrellas behave under loading when the tunnel face advances. For this reason, a scale laboratory model of pipe umbrella reinforced tunnel heading in a sandy soil was set up with the aim of understanding tunnel boundary deformation mechanisms in order to understand and define which is the best pipe umbrella design. The laboratory tests were performed using a box (1.5 m x 2.0 m x 1.8 m) in which the excavation of a 50 cm diameter tunnel at a low depth was simulated. The behavior of the ground and of some pipes of the umbrella was monitored during the test and the results were compared with FLAC 3D program modeling results. The paper reports the preliminary results of the first set of tests which, however, demonstrated the great efficiency of the pipe umbrella system, even for heading of half of the tunnel diameter, that the behavior of the tunnel face is a key parameter in the deformation scheme and that this support technique can be modeled using a three-dimensional code.
Keywords: tunnel, pipe umbrella, numerical modeling, portal.

1. Introduction

When adverse geotechnical conditions are encountered in tunnelling and the free span and self supporting times are short, the technological possibilities for the designer are: to reduce the size of the excavation sections, and increases the number of working attacks; to improve the rock mass quality or reinforce it or to pre-support the excavation to apply a pressure to the tunnel face. Among the different ways of pre-supporting a tunnel ahead of the tunnel face, steel pipe umbrellas have been widely used (Anagnostou & Serafeimidis, 2007). The umbrella method, which consists of a closely spaced, usually grouted, canopy of steel tubes, installed at the tunnel extrados, is effective in controlling deformations and volume losses for a wide range of ground conditions as it improves face stability and increases the stand-up time (Fig. 1). Steel pipes are usually installed with a 5°-10° dip (with reference to the horizontal) in such a way as to form an umbrella of pipes, with a size that ranges between 80 mm and 220 mm. The umbrella has a truncated cone shape which allows two adjacent umbrellas to overlapped thus covering advancement lengths of 12-15 m of which 9-12 m are of excavation. The diffusion of this method has been facilitated by the technological improvements on the installation machines.

This technique has been used for: the construction of shallow tunnels in soft ground where a good control of the ground displacements is necessary to reduce surface subsidence, the construction of tunnel portals because of low overburdens; the construction of tunnels through weak ground with high overburdens or through fault zones, and the crossing zones where the tunnel has already collapsed (Barisone *et al.*, 1982; Pelizza & Peila, 1993; Shirakawa *et al.*, 1999; Carrieri *et al.*, 2004; Volkmann *et al.*, 2007). Despite the large number of applications there are still no generally accepted methods or reliable means for designing a steel pipe umbrella. Among the other factors it is difficult to take into account: the high number of the involved geotechnical parameters, the three dimensional shape of the tunnel near the face, the effect of the overlapping of the umbrella pipes and their connection to the steel sets, the stiffness of the steel sets in comparison to the vertical loads and the geotechnical characteristics of the ground at the excavation face and finally the influence of the position of the face during the excavation process. Some researches have supposed that the pipes act by forming a shell around the tunnel boundary that reduces the stresses acting on the rock core ahead of the advancing tunnel face and that a two-dimension numerical model can also "model the physical behaviour of the reinforced tunnel in a realistic manner" (Hoek, 2001). Other authors believe that there is no significant mutual interaction between the singles pipes, thus each must be individually designed taking into account their longitudinal direction (Max & Mattle, 2002; Oreste & Peila, 1998; Peila & Pelizza, 2003), while others authors have used three-dimension numerical models to take into account the three dimensional shape of the tunnel face and the presence of the pipes and face reinforcement (Peila, 1994; Uhtsu *et al.*, 1995; Eclaircy-Caudron *et al.*, 2005) and finally some

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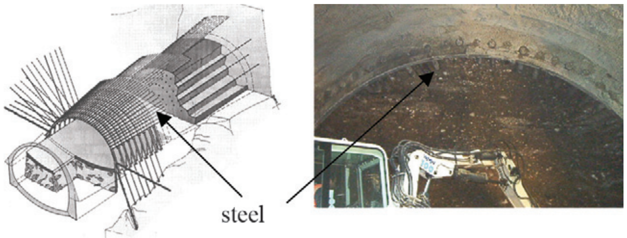

Figure 1 - Scheme of a steel pipe umbrella intervention for an excavation head and bench with face reinforcement and underpinning of the steel stets (courtesy Geodata SpA, Turin) (left) and photo of an application in a granular soil (Turin metro excavation) (right).

authors have carried out physical modelling to understand pipe behaviour more clearly or measurements in real tunnels (Volkmann *et al.*, 2007; Ocak, 2008).

A scale laboratory test can be an useful tool to better understand the global behavior of this pre-support during tunnel advancement as already developed by some researchers (Kim *et al.*, 2004; Takechi *et al.*, 2000; Yoo & Yang, 2001; Shin *et al.*, 2008) since measurements in real tunnels (Volkmann *et al.*, 2007; Ocak, 2008; Shirakawa *et al.*, 1999; Shin *et al.*, 2007) are particularly difficult to interpret due to the fact that the pipes are often overdesigned (because of the adopted safety factors) and therefore no significant deformations can be recorded and if collapse occurs, it is often very difficult to understand its real cause and its development.

2. Materials and equipment to simulate a tunnel excavation

A half-tunnel with 0.5 m diameter was excavated in a wooden 1.5 m wide, 2.0 m high and 1.8 m long box (Fig. 2). The box was filled with silty-sand, whose geotechnical parameters are summarized in Table 1. This paper presents the preliminary results obtained by the first set of experiments.

The sand was poured into 15 cm layers with its natural water content and the box filling was limited to one tunnel diameter of overburden, that is, 0.50 m over the tunnel crown.

The pipe umbrella was made with 18 fiber-glass (E = 27 GPa) 1.2 m long bars (Fig. 3) with a 10 mm x 3 mm section (Inertia modulus of 2.3×10^{-11} m^4). This type of elements, with different shape from the real ones, was chosen to have a more flexible support structure in order to permit

Figure 2 - Photograph of the test box where the position of the half-tunnel and holes for reinforcing elements can be observed.

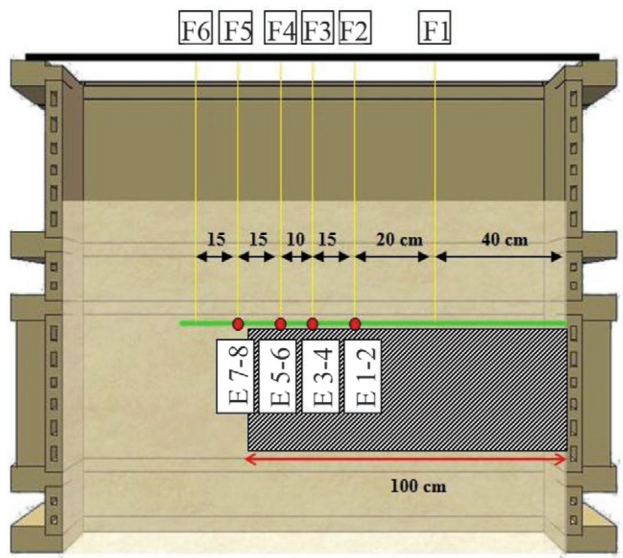

Figure 3 - Photograph of the fiber glass pipe umbrellas during installation.

deflection, even at low level of applied loads, so that the model could be used to understand the deformation shape expected ahead of the tunnel face.

The system has been monitored with (Fig. 4):

- Inspection windows located above the tunnel crown, where a 3 cm thick layer of sand, with different color

Table 1 - Geotechnical parameters of the silty-sand used for the test.

Sand content [%]	Silt content [%]	Water content (w) [%]	Dry unit weight (δ_d) [kN/m^3]	Total unit weight (δ_t) [kN/m^3]	Cohesion (c) [kN/m^2]	Friction angle (ø) [°]	Deformation modulus (E) [MPa]	Poisson modulus (ν)
90	10	4.6	16	17	1	32	25	0.40

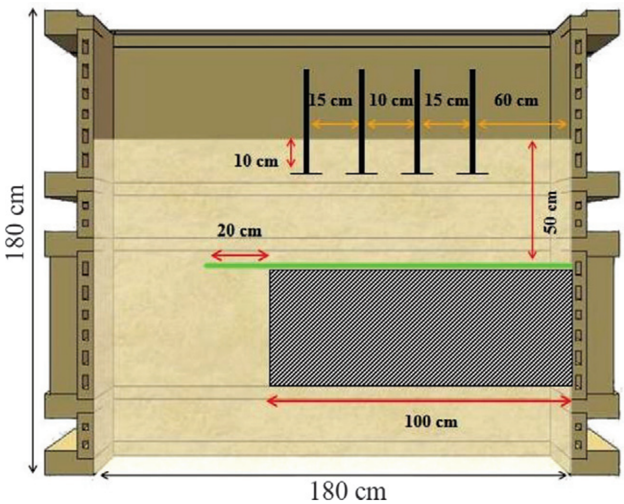

Figure 4 - Instrumentation scheme used in the test (upper figure E = strain-gauge; F= flexible wires that transmit the displacement outside the box) and topographic fixed points on ground level (lower figure).

Figure 5 - Tunnel excavation by hand.

Figure 6 - Photograph of the inner part of the tunnel after excavation and application of the overload on the surface (step 5). It is possible to observe that a local arch is created between adjacent reinforcing elements, preventing material flowing.

from the sand used for filling, was created. The deformation of the layer was registered optically;

- A bar instrumented using 8 strain-gauges that was installed as a third beam from the tunnel axis;
- A bar instrumented using 6 flexible wires to measure its displacements that was installed as a fourth beam from the tunnel axis;
- 8 topographically fixed points situated about 10 cm below the ground level.

The excavation was done by hand in four stages (Fig. 5) and the advancement steps are reported in Table 2. An overload equal to the overburden (8.4 kPa) was applied to the surface at the end of the last excavation step. In the first preliminary test, during the excavation, the tunnel was kept unsupported.

3. Results of the Test

Vertical displacements measured on ground level were practically nil without the application of the overload, since the pipe umbrella was able to absorb completely the ground load. When the overload was applied it was necessary to withdraw the fixed measurement points thus making it impossible to obtain any further ground subsidence data (Fig. 6). Although it has not been possible to precisely quantify the displacement values observed through the observation windows above the tunnel crown, they permitted some qualitative evaluations that is that displacements have the same shape and similar values as those measured on the instrumented pipes. Therefore, it can be said that, in soft soils, the instrumented pipe displacements are closely

Table 2 - Excavation steps.

Step	Excavation length (mm)	Position of the face from the entrance (mm)	Note
1	350	350	Excavation
2	200	550	Excavation
3	200	750	Excavation
4	200	950	Excavation
5	-	950	Overload application

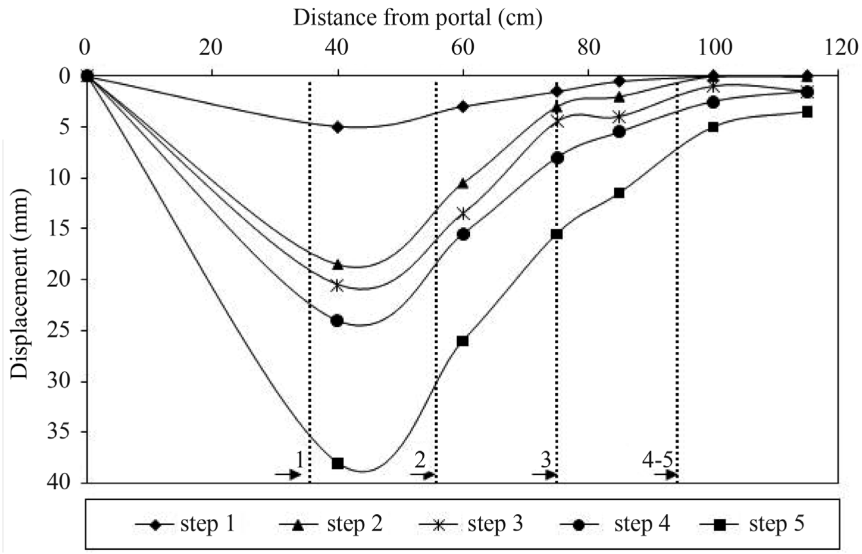

Figure 7 - Displacements observed in the fourth pipe from the tunnel axis for each excavation step.

linked to the surrounding soil displacements and are summarized in Fig. 7.

From Fig. 7 it can also be observed that the displacement of the reinforcing element ahead of the face, in this type of soil, is not negligible and reaches about the 30% of the excavated length (values similar to those found by Shin *et al.*, 2008).

Considering an advancing working face of 35 cm (0.7D), the value of the maximum displacement on the pipe umbrellas is 5 mm, which is 1% of the tunnel diameter. However, when the advancing working face reaches 1 D, the maximum displacement rises to 18.5 mm, (3.8% of the tunnel diameter). The test demonstrates a high efficiency of the pile umbrella system until an advancing working face of half of the tunnel diameter was reached. For an advancing work face of 95 cm, the observed maximum displacement in the pile umbrella is 26.5 mm (5.3% of the tunnel diameter) and when the overload equivalent was applied, the maximum displacement of the beam reached 40 mm.

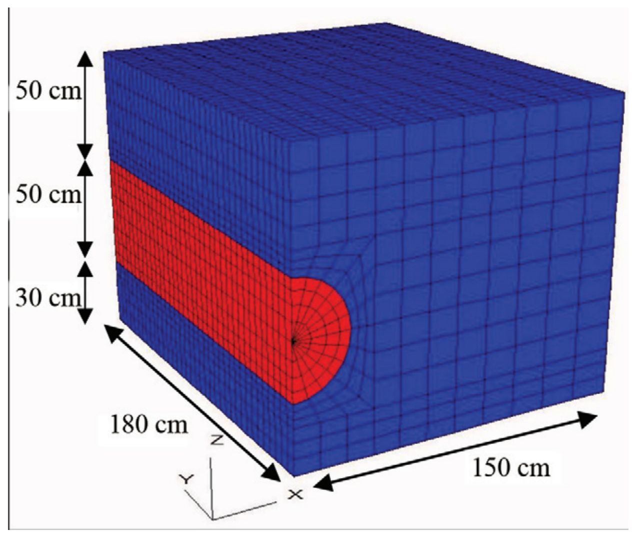

Figure 8 - Geometry of the Flac 3D model.

4. Numerical Modeling

In order to verify the test obtained results during test, a numerical model of the small scale tunnel has been set up. The model was implemented using the Flac 3D code (Ver. 3.1) and all the individual reinforcing elements were singularly modeled using beam element. The whole model was described by a mesh with 8584 nodes and 7524 elements (Fig. 8).

The comparison between the displacements values measured in laboratory test on the monitored reinforcing element and the FLAC 3D results (Fig. 9) have show a very good agreement (Figs. 10 and 11).

From an analysis of the tunnel boundary displacements it emerges that the 3D codes give that the vertical displacement is mainly concentrated in the free span length and that the reinforcing elements ahead of the face control the stability of the soil since few displacements are observed (Fig. 12).

5. Conclusion

Many different approaches have been used in the literature for steel pipe umbrella design and this problem is more relevant near the tunnel portal where low tunnel depths are faced and taken into account. For this reason, a simple scale laboratory test was developed aiming to better understand how this type of pre-support behaves during tunnel excavation, particularly ahead of the face.

The physical model allowed to observe that pre-supports installed ahead of the face permitted a stress transfer in the longitudinal direction until the region beyond the tunnel heading is reached (not yet excavated), thus mini-

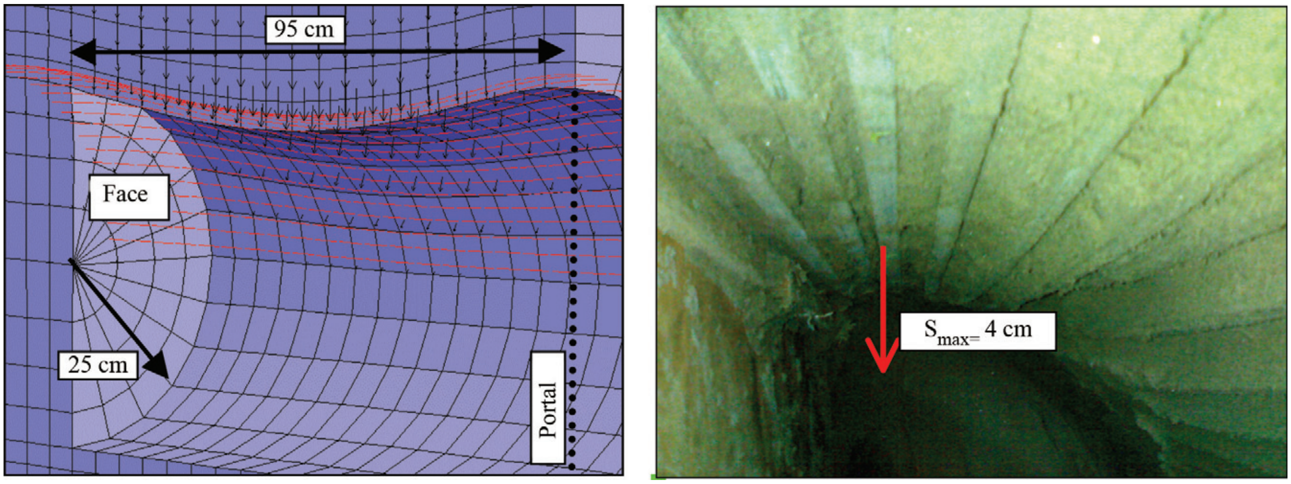

Figure 9 - Computed displacement using FLAC 3D code compared with the physical model results after step 5 (the arrow in the left figure shows the maximum measured displacement while the FLAC 3D shows the computed deformed mesh with deformation amplification factor equal to 1).

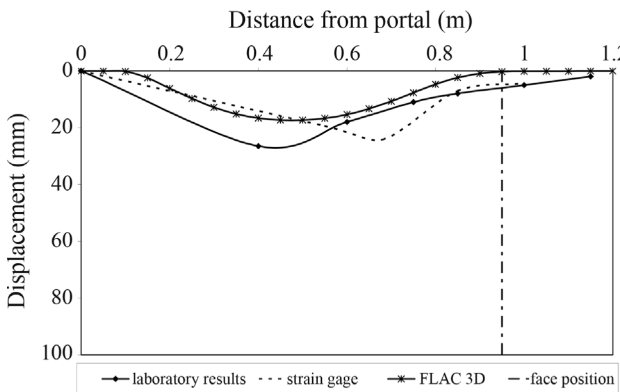

Figure 10 - Displacement for stage 4 (95 cm).

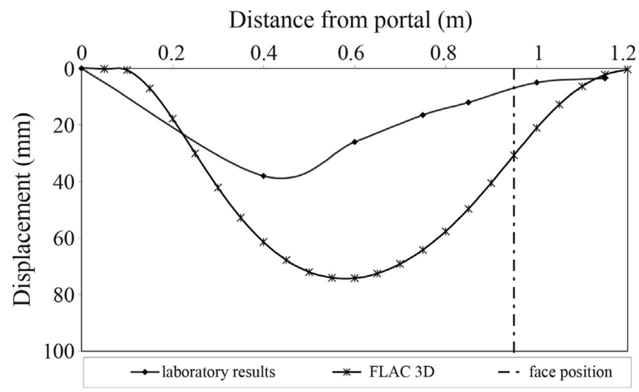

Figure 11 - Displacement after the overload (strain gauges damaged).

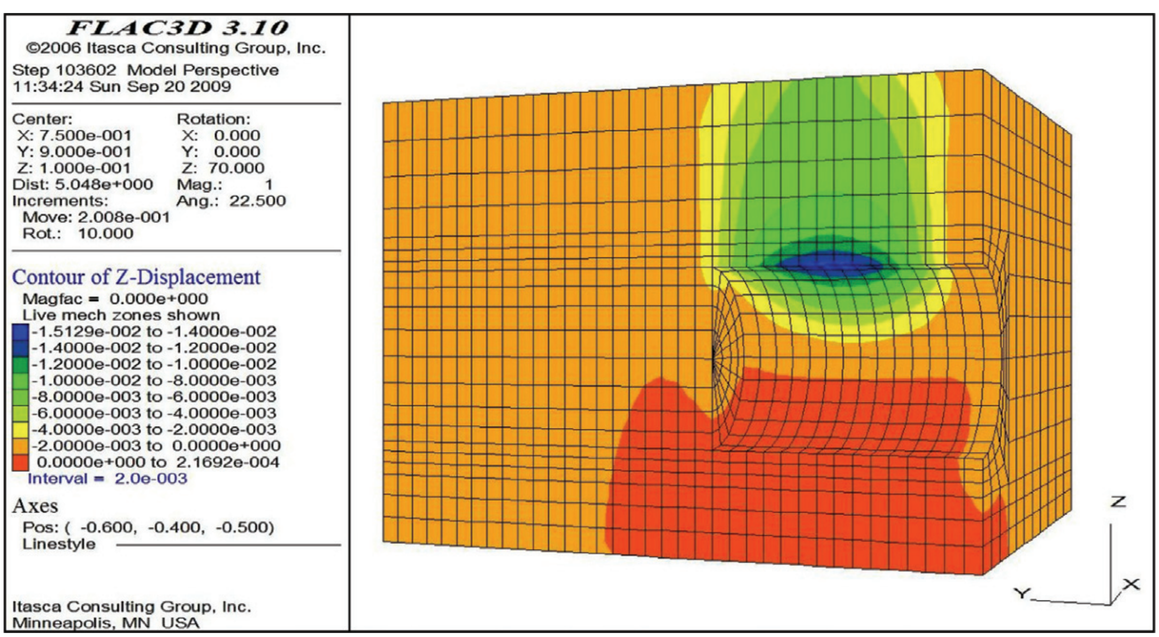

Figure 12 - Computed vertical displacements after step 3 (it is possible to see that the vertical displacement is mainly concentrated in the free span length and that the reinforcing elements ahead of the face control the stability of the soil since few displacements are observed).

mizing the global deformations of the tunnel boundary. It was also been highlighted that the displacement of the support element starts ahead of the face with a distance that, in the physical model case, was about 0.5D, which is also confirmed by the numerical 3D computation.

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