Analysis of the Importance of the Filling Material Characteristics Injected Around the Precast Concrete Lining in the Microtunnelling Technology

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ORIGINAL PAPER

Analysis of the Importance of the Filling Material Characteristics Injected Around the Precast Concrete Lining in the Microtunnelling Technology

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Abstract Microtunnelling is an excavation technique capable of positioning underground pipes with considerable advantages from the point of view of the environment, safety, construction times and also total costs, when the lengths are significant. The technology used is now reliable and is suitable for use in different types of soils and rocks without particular problems. The installation of the concrete lining of the micro-tunnel takes place behind the excavation machine starting from a thrust shaft. A filling material must be injected between the outer surface of the lining and the tunnel wall. This filling material plays a very important role, because it conditions the overall stiffness of the support system and, therefore, influences the radial load applied by the ground to the support system. Therefore, it is necessary to take into account the presence of the filling material in order to correctly size the support system of the micro-tunnel. This paper illustrates a new procedure for analyzing the behavior of the micro-tunnel support system, based on widely used and tested analytical calculation methods. This procedure was applied to a real case, underway in Northern Italy. The results obtained were then compared with those of numerical modeling, which made it possible to simulate the micro-tunnel in detail. From the comparison it was possible to note how the analytical calculation procedure leads to stress states similar (slightly higher) to those of numerical modeling; the first, therefore, can be effectively used in the preliminary stages of the design of micro-tunnels to obtain useful information on the characteristics of the support systems to be adopted to ensure the stability.

Keywords Microtunnelling · Concrete pipes · Support system · Grout filling material · Stress–strain state · Numerical modelling

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1 Introduction

Pipe jacking and microtunneling are becoming increasingly crucial for the installation of sewer pipes and service facilities all over the world. In fact, microtunneling represents a trenchless elective a method for laying underground pipes (Fig. 1), which has the benefit of creating the least amount of disruption along the construction route (Sterling 2020; Shen et al. 2011). Behind a remote-controlled tunneling shield known as the micro-tunnel machine, taylored hydraulic jacks are employed to force specially engineered pipes through the soil (Fig. 2). Once one pipe has been jacked, excavation is stopped, the jacks are pulled back, and another pipe ring is put behind the previous one and pushed

Fig. 1 Example of the micro-tunnel machine used for the construction of small diameter tunnels

Fig. 2 The system of pushing the elements of the reinforced concrete pipeline through hydraulic jacks that contrast on the side walls of the thrust shaft

forward. This operation is repeated until all the pipe elements have been inserted, establishing the permanent tunnel lining (Chapman and Ichioka 1999).

The face of the machine excavates and a circulating slurry system pumps the excavated materials till to reach the surface. The residual slurry is instead transferred to a dewatering plant after the slurry has been separated using a separation plant.

The microtunneling procedure starts with meticulous planning: all available material is analyzed, and a comprehensive site assessment is carried out to find out the characteristics of the soils that will likely be encountered, as well as information about the water table. In order to meet the project requirements, the pipe diameter and tunnel grade must be properly determined. Design grades and tolerances must be considered by the designer because they are frequently specified for gravity systems but are less important for conduits, dry utilities, and pressure pipes. Also, the diameter has a big impact on the project's cost and drive distance because bigger MTBMs often have greater power, thrust, and torque and can travel farther.

With the exception of the construction of launch and reception shafts, the depth at which the microtunnel is accomplished has little bearing on the expenses.

In comparison to conventional methods, this construction project is an environmentally friendly method and helps to conserve protected landscape areas. As a result, there is practically no negative effects on the traffic flow or any impact on the local economy. The benefits of this process are particularly apparent in urban areas. Additionally, since the micro-tunnel machine is remotely controlled, it boosts job site safety when crossing pipes or any other sort of infrastructure.

Without using an expensive stabilization or dewatering processes, microtunneling devices enable excavating in virtually any type of ground while ensuring the highest level of protection against ground movements (de Rienzo et al. 2009).

A material capable of filling the gap between the radius of the excavation and the external radius of the pipe (concrete lining elements) is injected. The injection occurs when this material is initially fluid; then it solidifies quickly. Subsequently it shows mechanical characteristics that evolve over time until reaching a stabilization. The average stiffness of this material during the loading phase of the support system has effects on the entity of the loads transmitted from the ground to the lining and, therefore, on the stresses state inside the concrete (Oggeri et al. 2021, 2022).

The studies available in the scientific literature on the mechanical behavior of the microtunnelling support system have never investigated the role of the filling material on the stress state induced in the concrete lining. Furthermore, comparisons between the results of the calculation with numerical modeling and those obtained through the use of tested and widespread analytical calculation methods are not available today.

In this paper, some mechanical tests developed in the laboratories of Politecnico di Torino on the filling material are presented and a calculation procedure is proposed on the basis of reliable analytical methods for the evaluation of the stress state induced within the lining of the micro-tunnel. The results of the study are applied to a real case in Italy and then compared with the results obtained by the numerical modeling considering both the presence of the concrete lining and of the injected filling material around it.

2 The Studied Case of a Micro-Tunnel Excavated in Northern Italy

The studied case refers to the realization of a new methane pipeline underpassing a river in Northern Italy. This new pipeline is constructed by means of the microtunneling technique, with the aim of replacing the old line, that was subjected to erosion due to the underground circulation of water and the river proximity. The micro-tunnel total length is of 546 m. The pipe inner diameter is 2.0 m while the outer diameter is 2.4 m; the excavation diameter is instead equal to 2.6 m and so an outer gap of 10 cm has to be filled in order to install the precast concrete lining. The depth of the tunnel axis is between 4–5 m (close to the ends of the path) and 21 m (in the middle zone of the path).

The cutting head of the machine has both cutters for the disintegration of the rock (pebbles and gravel grain) and scrapers to allow the penetration in the softer soil (sandy-silty).

The micro-tunnel is excavated in a large geomorphological paleocoinoid originated by progressively more recent deposits along the current path of the river. From a granulometric point of view, it is a soil with an alternation of sandy silt and silty sand weakly clayey, rich in gravel and pebbles of a centimetric size. The degree of compaction is very high, so that the relative density should be between 85 and 100%. The friction angle of the encountered soil can be assumed to be in a range between 32° and 35° and the elastic modulus between 30 and 45 MPa.

The adopted filling material is a mix of water and cement and its mix design is shown in Table 1.

Table 1 Filling material mix design

Volume (m^3)	1
Cement (kg)	1085
Water (kg)	638
Density ($tons/m^3$)	1.72
Water/Cement ratio	1/1.72

A detailed experimental phase in the laboratory allowed to evaluate the main mechanical characteristics of the filling material.

In the first step six specimens were prepared by using a Portland cement with the strength class of 42.5 (Portland cement with limestone type II/A-LL). The obtained mix has been then inserted in 52 mm diameter cylinders and left there to rest for 9 or 14 days.

The load-deformation tests (uniaxial compression tests) were performed using a Galdabini machine, equipped with a HBK load cell with precision of 10 N and two specular HBK LVDT transducers with a maximum vertical range of 50 mm and resolution of 0.001 mm. For each specimen (MG1, MG2, MG3, MG4, MG5, MG6) the load applied in time and the displacement at the two transducers were measured and, in real time, the mean value of them was evaluated. With the first three samples (MG1, MG2 and MG3), tested after 9 days from their preparation, a limit load around 20 kN was reached; for the specimens tested at 14 days (MG4, MG5 and MG6) the maximum load reached is around 35 kN except for sample MG5, for which the maximum value is around 20 kN again.

A first view of the raw data is shown in Fig. 3.

It can be noticed that the initial trend of the curves is not really linear: the cause can be related to the mix design, which provides a high amount of water and the total absence of aggregates, such as sand. These are also the reasons why the evaluated elastic modulus value is very low as well as variable in a fairly wide range (Table 2).

A picture of each broken specimen is shown in Fig. 4. As shown by the figure, for specimens MG1 and MG3 a little cone, which represent the frictional behavior of the material, is visible near the base. For specimen MG2 this cone is quite visible again but this time near the top of the cylinder. The same happens for the other three specimens that

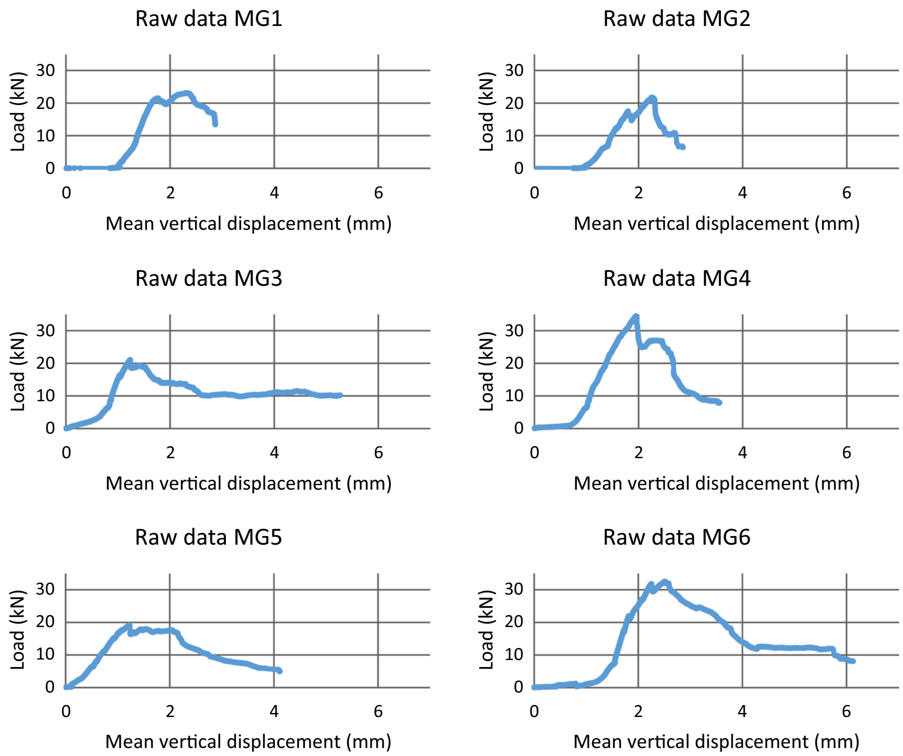

Fig. 3 Values of the applied vertical load (kN) and of the measured vertical displacements (mm) on the cylindrical samples in uniaxial compression tests

Table 2 Main test results on the six prepared specimens

	Volume (cm^3)	Apparent unit weight (g/cm^3)	Failure stress (MPa)	Elastic modulus (MPa)
MG1	171.77	1.86	11.54	1676
MG2	172.44	1.85	10.20	634
MG3	178.76	1.81	9.94	1314
MG4	163.92	1.90	16.61	1041
MG5	183.34	1.73	9.06	732
MG6	159.83	1.87	15.13	876

were tested after 14 days from the preparation: the cone is visible, for MG4 at the top of the specimen while for MG5 and MG6 at the base.

To get a representative elastic modulus of the filling material, the values obtained for the individual specimens were averaged. However, the sample with the lowest apparent unit weight was excluded in order to eliminate the specimen that has undergone greater bleeding. The final result is an elastic modulus approximately equal to 1100 MPa in a period of 9–14 days after the preparation of the specimen (grout mix).

The evolution of the elastic modulus of the filling material over time can be thought of as negative exponential starting from the time of the mixture preparation, with the achievement of the final asymptotic value, after an approximate time of 28–30 days. The estimate of the elastic modulus after 9–14 days therefore constitutes a representative value of the entire curing phase and can be adopted in subsequent calculations when it is necessary to indicate only one stiffness value of the filling material.

3 The Analysis of the Stress State in the Precast Concrete Lining by Using Analytical Methods

The convergence-confinement approach can be used to determine the relationship between the internal radial pressure of the microtunnel acting on the tunnel wall and the radial displacement of the tunnel wall (Oggeri et al. 2021; Spagnoli et al. 2016). The convergence-confinement curve is then coupled with the reaction line of the selected support system to yield

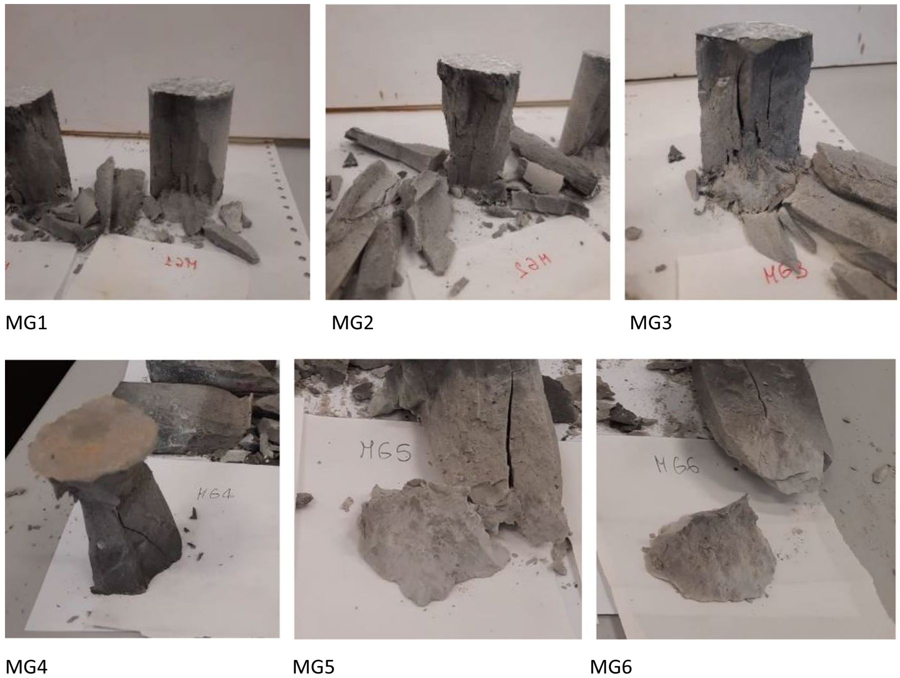

Fig. 4 Details of the six specimens at the end of the uniaxial compression test, after their breakage

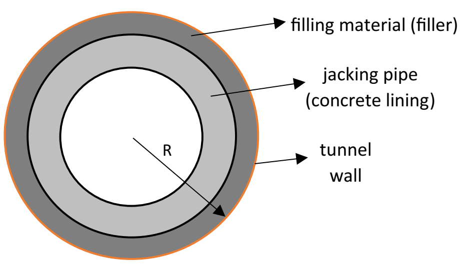

Fig. 5 Tunnel support geometric scheme (not in scale)

the equilibrium inner pressure and the maximum internal displacement. In this instance, the jacking pipe (precast concrete lining) and the filler that is put into the gap make up the support system (Fig. 5).

Some assumptions are needed:

- Circular excavation and high depth/void diameter ratio
- Continuous, homogeneous and isotropic ground and related properties
- Hydrostatic type in situ stress
- Bi-dimensional approach (a vertical and transversal section is studied)

In the case of an elastic medium, the convergence-confinement curve has a linear trend which means that as the inner pressure p decreases the radial displacement u linearly increases:

$$u = -\frac{1+\nu}{E} \cdot \left(p_0 - p\right) \cdot R \tag{1}$$

where p_0 is the lithostatic stress of the ground at the center of the tunnel, R is the tunnel radius, E is the elastic modulus and ν is the Poisson ratio of the ground surrounding the tunnel.

If instead the medium has an elastic–plastic behavior, the trend of the convergence-confinement curve is linear until the inner pressure is higher than a certain value, called critical inner pressure (p_{cr}); starting from it and decreasing the inner pressure the trend becomes curvilinear and a plastic zone develops around the void. The critical pressure is the value of the inner pressure when the elastic limit of the ground, dictated by the strength criterion, is reached in the soil around the tunnel.

On the same graph of the convergence-confinement curve (CCC), the reaction line of a support system (RLSS) can be added; this last element is a straight line that represents the load acting on the support system and which is characterized by:

- An inclination equal to the stiffness of the support system k_{sys} (Eq. 2) and

- A starting point equal to the value of the radial displacement of the tunnel wall where the support system is installed (u_0); for a microtunnelling system it is defined through the convergence-confinement curve by generally considering a value of the inner pressure equal to 50% of the lithostatic pressure p_0.

$$k_{sys} = \frac{2 \cdot E_{fm} \cdot (1 - \nu_{fm}) \cdot R \cdot \left[\frac{E_{fm}}{(1+\nu_{fm})} + (R - t_{fm}) \cdot k_{cl}\right]}{E_{fm} \cdot (1 - 2 \cdot \nu_{fm}) \cdot R^2 + (R - t_{fm})^2 \cdot \left[E_{fm} + (1 - 2 \cdot \nu_{fm}) \cdot (1 + \nu_{fm}) \cdot k_{cl} \cdot t_{fm} \cdot \left(1 + \frac{R}{(R-t_{fm})}\right)\right]} - \frac{E_{fm}}{(1 + \nu_{fm}) \cdot R} \quad (2)$$

where:

$$k_{cl} = \frac{E_{cl}}{(1 + \nu_{cl})} \cdot \frac{(R - t_{fm})^2 - (R - t_{fm} - t_{cl})^2}{(1 - 2 \cdot \nu_{cl}) \cdot (R - t_{fm})^2 + (R - t_{fm} - t_{cl})^2} \cdot \frac{1}{(R - t_{fm})} \quad (3)$$

where E is the elastic modulus, ν is the Poisson ratio, t is the thickness; all these parameters are defined for both the filling material (with the subscript 'fm') and the concrete lining (with the subscript 'cl').

In the studied case the jacking pipes (concrete lining) are made of concrete C50/60, with an elastic modulus of about 37,300 MPa. The typical value of the Poisson's ratio equal to 0.15 is considered for both the filling material and the concrete. The remaining main input parameters adopted in the calculation are:

- Soil unit weight of the ground: 18.5 kN/m.[3]
- Soil cohesion: 1 kPa
- Soil friction angle: 38°
- Soil elastic modulus: 60,000 kPa
- Soil Poisson ratio: 0.3
- Initial displacement of the tunnel wall at the installation of the support system u_0: 6.8 mm
- Concrete lining stiffness k_{cl}: $5.9 \cdot 10^6$ kN/m.[3]
- Support system stiffness k_{sys}: $3.8 \cdot 10^6$ kN/m.[3]

The convergence-confinement curve and the reaction line of the support system obtained for the studied case are shown in Fig. 6. The resulting equilibrium inner radial pressure (p_{eq}) is then equal to 0.24 MPa.

The Einstein and Schwartz (1979) method was also used in order to evaluate normal forces and bending moments acting inside the support system. The main assumptions of the method are:

- Circular opening in an infinite, homogeneous, isotropic, linear elastic ground

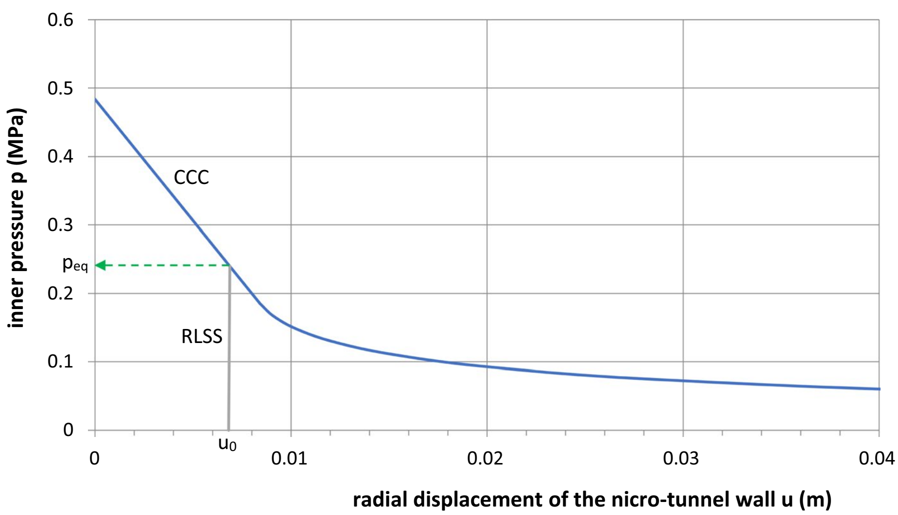

Fig. 6 The convergence-confinement curve (CCC) and the reaction line of the support system (RLSS) obtained for the studied case. Key: p_{eq}: final radial load acting on the support system (equilibrium inner radial pressure)

- Bi-dimensional geometry (vertical and transversal section of the tunnel)
- Elastic thick-walled shell simulating the support system

By placing the radial contact pressure between the surrounding ground and the support system equal to the p_{eq} obtained by the convergence-confinement method, it is possible to determine the maximum moment M_{max} that develops in the support system (the maximum bending moment assumes the same value at the crown and at the sidewalls but with opposite sign) and the axial force N, at the center of the crown and on the sidewalls (Oggeri et al. 2021):

$$M_{max} = \frac{p_{eq} \cdot R^2 \cdot \left(1 - K_0\right)}{\left(1 + K_0\right) \cdot \left(1 - a_0^*\right) + \left(1 - K_0\right) \cdot \left(3 - 6 \cdot a_2^*\right)} \cdot \left(1 - 2 \cdot a_2^*\right) \tag{4}$$

$$N_{crown} = \frac{p_{eq} \cdot R \cdot \left(1 + K_0\right)}{\left(1 + K_0\right) \cdot \left(1 - a_0^*\right) + \left(1 - K_0\right) \cdot \left(3 - 6 \cdot a_2^*\right)} \cdot \left(2 \cdot a_2^* - a_0^*\right) \tag{5}$$

$$N_{sidewall} = \frac{p_{eq} \cdot R \cdot \left(1 + K_0\right)}{\left(1 + K_0\right) \cdot \left(1 - a_0^*\right) + \left(1 - K_0\right) \cdot \left(3 - 6 \cdot a_2^*\right)} \cdot \left(2 - a_0^* - 2 \cdot a_2^*\right) \tag{6}$$

where the auxiliary parameters can be expressed as:

$$a_0^* = \frac{C^* \cdot F^* \cdot (1 - \nu)}{C^* + F^* + C^* \cdot F^* \cdot (1 - \nu)} \tag{7}$$

$$a_2^* = \frac{(F^* + 6) \cdot (1 - \nu)}{2 \cdot F^* \cdot (1 - \nu) + 6 \cdot (5 - 6 \cdot \nu)} \tag{8}$$

$$C^* = \frac{E \cdot R \cdot (1 - \nu_{cl}^2)}{E_{s,eq(C^*)} \cdot \left(t_{cl} + t_{fm}\right) \cdot (1 - \nu^2)} \tag{9}$$

$$F^* = \frac{12 \cdot E \cdot R^3 \cdot (1 - \nu_{cl}^2)}{E_{s,eq(F^*)} \cdot \left(t_{cl} + t_{fm}\right)^3 \cdot (1 - \nu^2)} \tag{10}$$

where: K_0 is the horizontal thrust coefficient at rest (0.38 in the studied case); $E_{s,eq}$ is the equivalent elastic modulus of the support system evaluated for Eq. 9, 10, 11 and 12; C^* (compressibility ratio) and F^* (flexibility ratio) are two dimensionless parameter that take into account the relative stiffness of the ground mass with respect to the tunnel support system.

$$E_{s,eq(C^*)} = E_{sl} + E_{fm} \cdot \frac{t_{fm}}{t_{sl}} \tag{11}$$

$$E_{s,eq(F^*)} = \frac{4}{t_{cl}^3} \cdot \left\{ E_{cl} \cdot \left[y_0^3 + \left(t_{cl} - y_0\right)^3 \right] + E_{fm} \cdot \left[\left(t_{cl} + t_{fm} - y_0\right)^3 - \left(t_{cl} - y_0\right)^3 \right] \right\} \tag{12}$$

where y_0 is the distance of the neutral axis of the support system section from the intrados of the lining and it is defined as:

$$y_0 = \frac{E_{cl} \cdot t_{cl}^2 + E_{fm} \cdot (t_{fm}^2 + 2 \cdot t_{cl} \cdot t_{fm})}{2 \cdot (E_{cl} \cdot t_{cl} + E_{fm} \cdot t_{fm})} \tag{13}$$

The values obtained for the maximum bending moment M_{max} and the normal force N, acting at the crown and on the sidewall, can be used to verify the proposed cross section of the support system.

Developing the calculations for the studied case a maximum moment value M_{max} equal to 48.07 kN•m and axial forces of 86.49 and 285.12 kN were obtained respectively at the center of the crown and at the sidewalls.

4 Numerical modeling

Itasca Consulting Group's FLAC (Fast Lagrangian Analysis of Continua) software was used to carry out a 2D numerical modeling analysis of the stress state induced in the filling material and in the concrete lining of the jacking pipes (Itasca Counsulting Group, 2022). By offering a sophisticated geotechnical analysis of the stress state of the medium and taking into consideration the presence of support and reinforcing systems (Zaheri et al. 2020), this code uses the explicit finite difference formulation to simulate the behavior of structures realized in rocks or soils.

A grid of tiny numerical elements connected by nodes discretizes the ground that surrounds the void. Each component replicates, in an approximate way, the physical behavior of the associated area as it responds to the applied forces and the boundary conditions in accordance with the prescribed stress–strain law.

The main steps that were adopted in the numerical model of the studied case are:

- Grid generation
- Definition of the constitutive law and of the geomechanical properties of the ground
- Application of the boundary conditions to the model
- Initiation of lithostatic stresses
- Simulation of the excavation phases, including the placement of the support system
- Numerical calculation

In the studied problem the adopted grid represents a quarter of the microtunnel; it is characterized by a total of 5000 elements. The minimum radius of the model is the internal radius of the pipes (1 m); the maximum radius produces the outer border of the model where the boundary conditions are applied. The mesh of the model is shown in the Fig. 7. 4 rows of elements were dedicated to represent the concrete lining along the tunnel profile and two rows to simulate the presence of the filling material.

The chosen constitutive law is that of an elastic medium for all the materials: the ground, the concrete lining and the filling material. A pressure acting on the nodes placed along the tunnel profile equal to 50% of the lithostatic in-situ stresses was initially applied in order to simulate the condition where the support system is installed at a certain distance from the excavation face. Only later the support system was activated within the model.

As shown in the Fig. 8, at the crown the maximum compressive stress obtained by the calculation, which is around 3 MPa, is reached at the extrados of the concrete pipe, while its intrados is completely unloaded; the layer of filling material is instead characterized by a compressive stress of around 1 MPa. For the sidewall, instead, the maximum compressive stress is equal to 6 MPa and acts at the intrados of the concrete pipes; moving toward the filling material the maximum principal stress tends to decrease.

Concerning minimum principal stresses (Fig. 9), at the crown the intrados of the pipe undergoes the maximum tensile stress of 2.7 MPa while at the extrados a

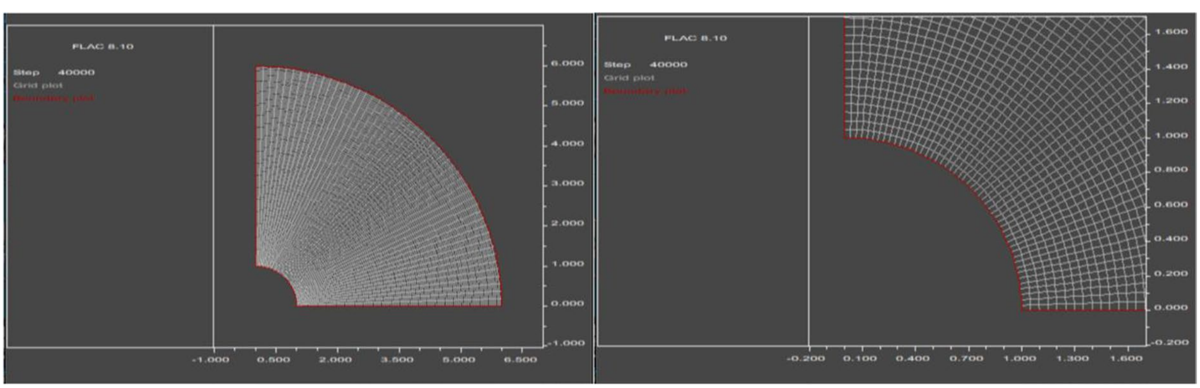

Fig. 7 General (left) and detail (right) view of the mesh of the numerical model developed to analyze the stress state in the support system and in the ground for the studied case. A total of 5000 quadrilateral elements were used to represent a quarter of the micro-tunnel geometry, together with the concrete lining and the filling material

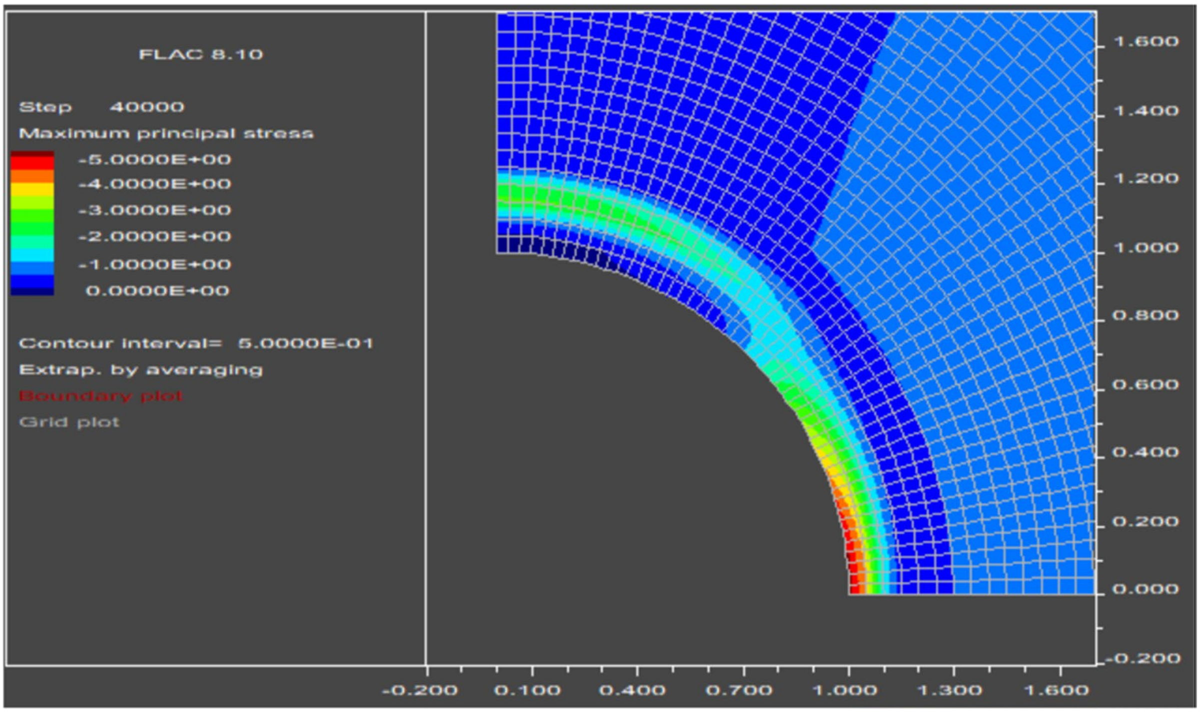

Fig. 8 Maximum principal stresses developed in the model after the completion of the micro-tunnel construction. The first 4 rows of elements starting by the inner profile of the model simulate the concrete pipe, the next two rows of elements represent the filling material; the remaining elements refer to the surrounding ground. Key: negative values: compression; positive values: traction

compressive stress of 0.25 MPa acts. At the sidewalls just the extrados zone is subjected to a tensile stress: 0.25 MPa.

All these stress values are used to determine the maximum bending moments and the normal (circumferential) forces acting in the two locations of interest (centre of the crown and sidewalls) along the tunnel profile. A maximum moment value equal to 29.52 kN•m (crown) and 32.27 kN•m (sidewalls) and axial forces of 75.21 and 338.75 kN were obtained in the support system respectively for the center of the crown and the sidewalls.

5 Discussion and conclusions

Microtunneling is a valid substitute for open tunneling methods because it offers the best means of minimizing surface alteration, avoiding utility conflicts, reducing risks to workers, traffic, and pedestrians, as well as lowering the likelihood of ground movements, especially in areas with high water tables. Additionally, it is the favored option for pipeline and deep sewer projects. Since geological and geotechnical assessments are crucial to determining the path of the microtunnel and all of the equipment that will be employed, the first step in the microtunnel process, planning and design, is the most crucial.

During the use of a microtunnelling technique, a concrete pipe is inserted to ensure the advancement of the machine and the support of the micro-tunnel. A filling material is injected around it, able to connect the concrete lining to the surrounding ground. This filling material plays a very important role in the transmission of loads from the ground to the support system and, therefore, also on the stress levels induced in the concrete. It is therefore very important to be able to evaluate in detail the interaction between the micro-tunnel and the support system in the design phase in order to correctly choose the geometric and physical characteristics of the involved materials. Properties of grout depend on several conditions,

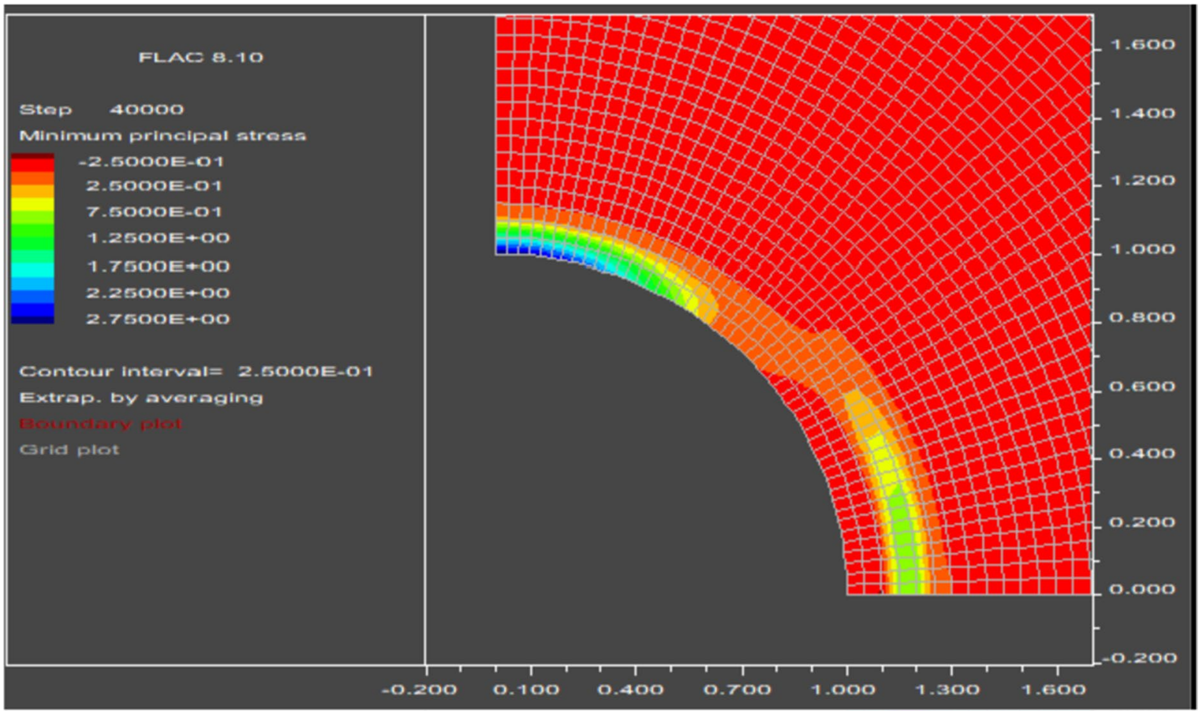

Fig. 9 Minimum principal stresses developed in the model after the completion of the micro-tunnel construction. The first 4 rows of elements starting by the inner profile of the model simulate the concrete pipe, the next two rows of elements represent the filling material; the remaining elements refer to the surrounding ground. Key: negative values: compression; positive values: traction

such as mix design, mixing and injection procedures, time dependant convergence of the ground, gelling time, local water flow in the ground: for these reasons a preliminary laboratory investigation can reveal suitable modes to improve efficiency of the supporting system (Oggeri et al. 2021, 2022).

In the present paper a calculation procedure has been illustrated able to evaluate the stresses induced in the support system by adopting analytical calculation methods, widespread in the scientific literature. In particular, the convergence-confinement method combined with the Einstein and Schwartz method allow to determine not only the radial pressure transmitted from the ground to the support system, but also the maximum bending moments and the normal (circumferential) forces that develop in the concrete lining at the two critical points: centre of the crown and on sidewalls. The calculation procedure was used for a studied case of a 2.6 m diameter micro-tunnel excavated in Northern Italy in medium-thickened loose soils with known mechanical characteristics.

In the calculation it is necessary to insert the mechanical characteristics of the filling material and in particular its elastic modulus. This parameter has direct effects on the stiffness parameters of the support system which condition the stresses that develop inside it. For this reason, a specific laboratory investigation made it possible to evaluate the elastic modulus of the filling material used for the studied case.

The proposed procedure is reliable and fast and is able to produce all the calculation results that are necessary for dimensioning the support system. In particular, it is possible to evaluate the stress state in the concrete and in the filling material for the dimensions of the lining assumed in the calculation. The particular speed obtainable from the calculation is useful in the dimensioning phase, when the dimensions of the support system must be varied and the maximum state of tension to which it is subjected must be continuously checked.

Numerical modeling through a finite difference computation code has allowed to analyze the same problem, using a more complex approach. The model developed covered only a quarter of the micro-tunnel and required the adoption of 5000 quadrilateral numerical elements. The study made it possible to detect the stress state that develops in the concrete lining and in the filling material and, from these, to subsequently determine the bending moments and normal forces in the same two critical points along the tunnel profile.

The values of the bending moments and normal forces obtained by the numerical modelling results to be slightly lower compared to those evaluated with the proposed analytical approach, with the only exception for the normal force acting at the sidewall, which is instead greater. This little discrepancy is justified by the fact that the simplified analytical approach is based on some simplified assumptions that are not present in the numerical modelling.

The use of simplified calculation procedures, as in the case of this paper, when they allow to obtain precautionary results, capable of slightly overestimating the stress level in the supports, allow to quickly reach the correct sizing of the structures, taking into account some aspects that show a great influence on the results, as in the problem under examination the presence of the filling material around the concrete lining. Then, only a final refinement or verification of the geometric and physical characteristics of the materials to be used can be delegated to a more complex numerical modeling.

The proposed procedure is particularly suitable for complex support systems, such as in the case of microtunnelling, when two different materials are present: the filling material which has mechanical characteristics that vary over time and which reach the stabilization after about 28–30 days from the preparation of the mixture, and precast concrete of high strength and stiffness. It can also be used in the case of segmental lining (when the tunnel is excavated with a TBM) and definitive concrete linings made in the tunnel shortly after the installation of the shotcrete lining.

Author contribution All authors contributed to the study conception and design, material preparation, data collection and analysis, first draft of the manuscript and all authors commented on previous versions of the manuscript. All authors read and approved the final manuscript.

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Data availability No datasets were generated during the current study.

Declarations

Conflict of interest The authors have no relevant financial or non-financial interests to disclose.

References

Chapman DN, Ichioka Y (1999) Prediction of jacking forces for microtunnelling operations. Tunn Undergr Space Technol 14(Supplement 1):31–41

De Rienzo F, Oreste P, Pelizza S (2009) 3D GIS supporting underground urbanisation in the City of Turin (Italy). Geotech Geol Eng 27(4):539–547

Einstein H, Schwartz W (1979) Simplified analysis for tunnel supports. J Geotech Eng Div 14541(GT4):499–518

Itasca Counsulting Group, https://www.itascacg.com/software/flac-8-basics, (2022)

Oggeri C, Oreste P, Spagnoli G (2021) The influence of the two-component grout on the behaviour of a segmental lining in tunnelling. Tunn Undergr Space Technol 109:103750

Oggeri C, Oreste P, Spagnoli G (2022) Creep behaviour of two-component grout and interaction with segmental lining in tunnelling. Tunn Undergr Space Technol 119:104216

Shen X, Lu M, Chen W (2011) Tunnel-boring machine positioning during microtunneling operations through integrating automated data collection with real-time computing. J Constr Eng Manag 137(1):72–85

Spagnoli G, Oreste P, Lo BL (2016) New equations for estimating radial loads on deep shaft linings in weak rocks. Int J Geomech 16(6):06016006

Sterling RL (2020) Developments and research directions in pipe jacking and microtunneling. Undergr Space 5(1):1–19

Zaheri M, Ranjbarnia M, Dias D, Oreste P (2020) Performance of segmental and shotcrete linings in shallow tunnels crossing a transverse strike-slip faulting. Transp Geotech 23:100333