The Italian approach for the design and the excavation of conventional tunnels: the case of the “Fabriano” tunnel

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ABSTRACT: This paper shows the results of the analysis of the data acquired during the excavation of the new “Fabriano” Tunnel along the railway line from Rome to Ancona in Italy. The comparison between the results of the analyses and the consolidation system used for the excavation, gives an indication about the relationship which links the method of excavation commonly used today in Italy for conventional Tunnels, and the responses of the excavated rock mass. The application of the method spans, from competent rock mass to "not self supporting" soils and provides an industrial cyclic process for the execution of tunnels. It defines different typologies of consolidation systems that shall be used along the tunnel, based on a full face excavation and on the construction of a final lining generally close to the tunnel face. The approach has been used to excavate the “Fabriano” tunnel, which is here taken as an example for the application of the methodology.

1 INTRODUCTION

The concept of excavation of a tunnel or, in general, of an underground structure, usually follows the experience acquired by a certain country. Often these experiences are implicitly implemented inside rules and/or regulations of the country itself, providing a codified system for the realization of this kind of works.

This is the case, for example, of the so called "New Austrian Tunneling Method" (NATM). This method considers different approaches to the excavation using, mainly, rock bolts and shotcrete as a system for stabilizing and supporting the loads from the rock mass.

According to this method, the approach to the excavation and both, quantity of rock bolts and thickness of the shotcrete are related to the expected rock mass behaviour. During the excavation, the support system can be adapted, both in quantity and type, monitoring the real response of the excavated rock mass. The latter is measured by using an apposite monitoring system installed during the excavation phase.

The shotcrete can absolve either the function of a temporary support or a final lining depending on the external loads and the expectations of the client.

The method has a strong observational component. In fact, it is implicit in the design process that the modification of the consolidation and support system can be changed during the excavation.

In spite of the codification of the NATM method, which is often wrongly identified as a general method for the excavation of conventional tunnels, its application suffers the risks related to the concept of a typical observational method (Kovari, 1993).

The rock mass can reach its resistance limit, that is very difficult to define properly especially in complex rock masses, increasing the risks for working people and for the excavation equipments, especially for masses with limited self supporting capability.

In the Italian approach to design and excavation of conventional tunnels, this concept is reversed. The latter follows three main steps:

- First, the rock mass is characterized, by using geological and geotechnical data, and different zones where the rock mass tends to behave homogeneously are defined. The behavior before mentioned, particularly concerns the excavation face, which also influences the final equilibrium;

- In the second step, for each identified homogeneous zone, an adequate consolidation and support system for the rock mass is defined. This is summarized in the so called "Section Types". Contrary to the NATM method, in this case, the consolidation system is strongly based on the tunnel face stability conditions and on the concept that the execution of the tunnel must be based on a full face excavation (Figure 1). The latter is partialized only for very special situations. If the tunnel face is not stable, its stability condition is guaranteed by using artificial consolidation elements. These procedures permit to industrialize the excavation providing a cyclic process. It is part of the design the definition of a guideline for changing the “section type” during the excavation, based on the monitored response and on the real encountered condition of the rock mass compared with the design assumptions;
- Finally, the third step is the feedback during the construction. This is provided by organizing a monitoring system to be installed during the excavation which permits to check out the real responses of the rock mass to the excavation. As the concept is based on the tunnel face stability conditions, when there are problems related to its stability, the monitoring system includes also equipments to check the extrusion of the tunnel face. It must be noted that the use of the monitoring system is conceptually employed to check the initial design hypothesis, rather than an instrument to design during the construction. If the response is different from the expected, the section type is modified according to the above mentioned guideline.

The described approach, very often used today in Italy, (that only with the purpose to identify the methodology it is here called ISET-Italian System for Excavation of Tunnels), ranges in its use from the competent rock mass to "not self supporting" soils.

Even it is not the aim of the present paper, the major differences between the Italian approach and NATM, are described in the following few rows. The Italian approach for design and construction of tunnels in all geotechnical condition was developed from the observation and interpretation of the deformation produced during the excavation of tunnels, especially the deformation of the tunnel face. This provided the basic ideas from which systematic study of tunnel failure is in relation to the stability of the advance core. This led to reconsider the contents of the deformation response of the ground to excavation, no longer in conventional terms of convergence of the cavity (NATM approach), but also in terms of extrusion of the core-face, (pre-convergence) and finally convergence of the cavity. In other words, it is considered that the plastic zones behind the tunnel face, influences also the extension of the plastic zones around the cavity and so the behavior of the excavated rock mass. The approach, focus on how to guarantee the stability of the core-face (and therefore of the cavity itself) by means of conservative action to reinforce and/or protect it. This allow to excavate tunnel also in bad soil condition, in full section excavation and to build tunnel final lining close to tunnel face so to take in account the benefit effect towards strength and strain control of closing the tunnel section with invert.

To illustrate a real example of application of the approach, this paper summarizes the results of the analysis of the monitoring data acquired during the excavation of the “Fabriano” tunnel.

This is only an example of application of the method. Tunnels in Italy are currently excavated according to the ISET principles.

These results are compared to the defined consolidation and primary support system, used for the excavation.

The comparison between the results of the analyses and the acquired data gives an indication about the relationship which links the method of excavation, commonly used today in Italy, and the responses of the excavated rock mass.

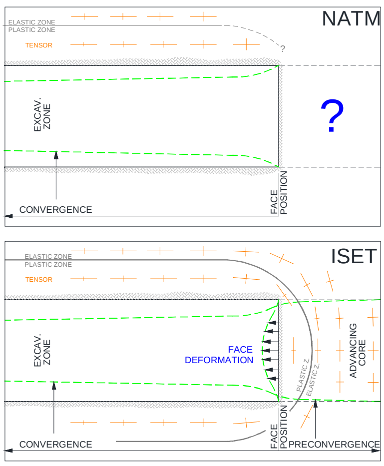


Figure 1.Influence of the tunnel face.

2 ITALIAN APPROACH FOR TUNNELS

In recent decades, the evolution of the techniques of excavation associated with the new available technologies together with an increased sensitivity on the safety aspects, addressed the approach to the excavations of tunnels toward a more industrialized process.

To achieve these objectives, different systems to excavate underground structures, which connect design, safety and production, have been developed. Starting from these concepts, a new approach to design tunnels based on the principle that the equilibrium of the cavity is strictly connected to the stability's condition of the tunnel face, took place in Europe.

At this regard, Lombardi (1974) pointed out that:

- The static problem of the cavity is three-dimensional in stresses and strains; a radical transformation of the state of tension takes place at the tunnel face;
- The problem is statically indeterminate; the reaction of the support which is opposed to the pressures depends both on its characteristics (strength, stiffness, etc.) and on the realization's techniques of the element and on the deformation path of the rock;
- The time factor is essential before and after the construction work.

The above mentioned author defined four stability conditions, considering both the cavity and the tunnel face, on which to orientate the definition of the excavation's method and the choice of support system.

These concepts have been developed in Italy, also thanks to the improvement of available technologies and new technical solutions.

This was possible using consolidation systems to improve the characteristics of the soil/rock mass.

The approach was firstly introduced by Lunardi (1988). The definition of the behaviour of the excavation is related to the relationship between the strength capacity of the mass and the stress generated from the excavation of the tunnel and the consequent deformation. Depending on the stability conditions of the tunnel face, Lunardi (1988) defined three different classes of behaviour of the excavation:

1. Behaviour where the tunnel face is stable;
2. Behaviour where the tunnel face is stable at "short-terms";
3. Behaviour where the tunnel face is unstable.

In the first case the reaction of the surrounding mass is substantially elastic. In the second case not negligible plastic zones around the tunnel and the into the tunnel face area appear. In the third case important plastic zones are present ahead the tunnel face and around the cavity.

For each of these classes, a so called "Section Type" to allow the excavation controlling deformations and potential instability, has been defined. The consolidation of the rock mass and the support system of the cavity are strictly related to the expected behaviour of the excavated rock mass. In particular:

- Where the state of stress in the ground at the tunnel face and around the cavity, is close to the boundary defining the strength limits of the rock mass, (deformation nearly to the elastic limit) the stabilization measures are aimed to assure the safety of the excavation (typically rock bolts associated to a certain thickness of shotcrete);
- Where the tunnel face is defined "stable at short-term"; the consolidation system is aimed to reduce the plastic zone ahead the tunnel face, so that the reaction of the rock mass around the cavity far from the tunnel face is less influenced by the disturbed material surrounding the tunnel face itself;
- Where the tunnel face is unstable the consolidation system is aimed to increase the characteristic of the rock/soil mass so that it can be excavated without stability problems. In these cases the deformation of the cavity and of the tunnel face shall be kept under predefined limits.

By using this concept, a cyclic process for consolidating, supporting, and excavating the soil/rock mass, can be organized. The level of stability can be always guaranteed by using different intensity of the consolidation elements.

In addition to the stability of the tunnel face, the deformations of the cavity far from the tunnel face shall not pass certain limits. If important deformations of the cavity are expected, typically in soils, the final lining is placed close to the tunnel face.

The cycle allows for a rapid reaction to changes in the characteristics of rock mass, by only changing the section type or number of consolidation elements, depending of the characteristics of the encountered material to be excavated and/or to the response of the rock mass to the excavation monitored by an appropriate monitoring system.

Thus, the project includes the preparation of appropriate guidelines to define how to manage changes to the initial defined typical sections.

3 THE FABRIANO TUNNEL: PROJECT

3.1 *Description*

The tunnel is a double track tunnel placed along the railway line from Rome to Ancona, running from West to Est in the middle of Italy. It has a circular shape with an outer radius of 6.20 m and an inner radius of 5.25 m.

The Rome tunnel portal is placed at the chainage km 2+448, while the Ancona tunnel portal is at the chainage km 4+130. Total length is 1682 m.

The tunnel may be divided into two sections: a first section with an average cover of 40 m and a max cover of 50m, for a length of about 700 m from the Rome side; and a second section towards Ancona, where the average cover is about 15 m.

The construction took place between the years 2006-2008.

3.2 *Geological conditions*

The geological structure represents a typical example of geology of the Apennine valleys in the middle of Italy and it consists of the following formations, from the ancient to the most recent:

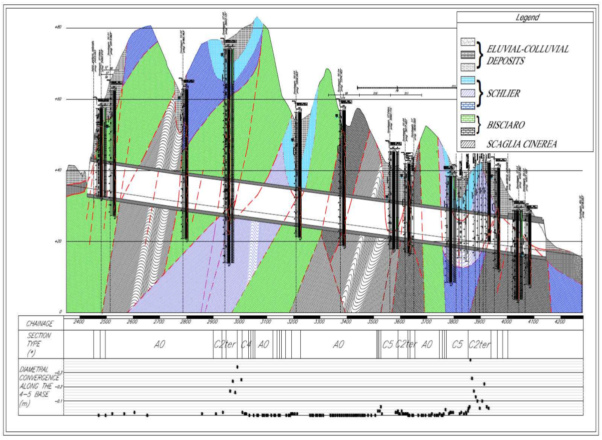
- Alternation of calcareous marl, argillaceous marl and marly limestones (“Scaglia Cinerea”);
- Alternation of marly limestones, calcareous marl and argillaceous marl (“Bisciaro”);
- Alternation of argillaceous marl gray in color, gray-blue marly clays with calcareous intercalated layers (“Schlier”);
- Eluvial-colluvial deposits: originated from the weathering of the bedrock, they are placed at the bottom of the valleys. This formation consists mainly of clayey and sandy silts with a variable thickness from a few meters to a maximum of 20 m.

The development of this geological structure allows to divide the tunnel in two “homogeneous” sectors, corresponding to the same zones divided by the cover:

- The first sector for a length of 700 m from the Rome side, consists of an alternation of the Bisciaro formation (slightly fractured) mixed with the Scaglia Cinerea and Schlier formations (more fractured);
- The other section to the Ancona side, it consists of a succession of argillaceous marl and siltstones belonging to the Scaglia Cinerea and Schlier formations, often as an alteration of the rock mass in terms of clays and silts.

During the excavation of the tunnel, groundwater has never been intercepted.

The distribution of the geological formation encountered during the excavation is shown in Figure 2.



* Blanc spaces correspond to a different section type
Figure 2.Geomechanical profile and max convergences

3.3 *Geotechnical characterization*

An intense investigation program has allowed to determine the mechanical properties of the rock masses and soils crossed by the tunnel.

In Table 1 are summarized the average mechanical properties from laboratory tests.

Table 1.Main properties of the intact rocks (lab. tests).

Material	UCS [σ_{ci}] (MPa)	Secant E [E_{50}] (MPa)
Bisciaro	18-22	4000-4500
Schlier/Scaglia	2-3*	40-50*

* poor quality of the samples

The laboratory tests has been integrated with the in situ tests and both have allowed to define the geotechnical parameters for the rock masses and soils. These parameters are summarized in Table 2.

They were estimated according to a linear-elastic perfectly plastic constitutive law with the Mohr-Coulomb strength criterion and are divided for two sections of the tunnel, as previously reported.

Table 2.Characteristics of the rock and soil masses.

Material	Cover [H] (m)	Unit weight [γ] (kN/m^3)	Coh. [c'] (kPa)	Fric. angle [ϕ'] (°)	Elast. mod. [E'] (MPa)
Bisciaro	0-25	24	54	42	1840
	25-50	24	95	36	1950
Schlier/ Scaglia	0-25	24	33	35	890
	25-50	24	68	30	1100
Weather. Schlier	0-25	21	156	26	230
Eluvial-colluvial deposits	-	20	10	26	16

3.4 *Support sections type*

According to the ISET concept, three conceptual excavation behavior and consequent "Section Types" for consolidation and support (named A, B and C) have been adopted (see Figure 3). The name of the section type follows the terminology suggested by Lunardi (1988).

Section type A was correlated to a self supporting rock mass (elastic or pseudo-elastic response-behavior "A"), section type C to an unstable materials (definitively elasto-plastic responses- behavior "C"), while the section type B was applied in the transient conditions (behavior "B").

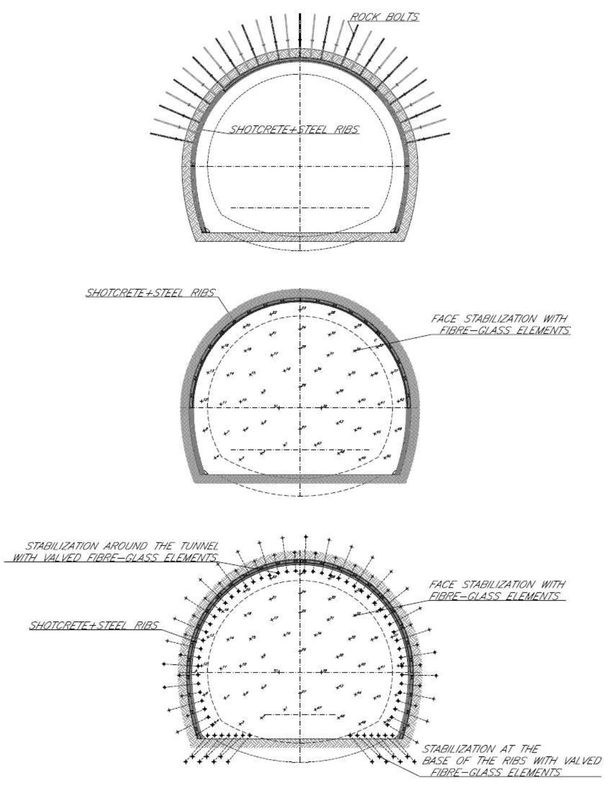


Figure 3.Sections type (base) adopted (A at the top, B in the middle and C at the bottom).

The "sections type" bases have been modified during the execution, leading to generate of various "subcategories", maintaining anyway, the basic concept. The modification were done according to the real condition of the material encountered during the excavation and on the monitoring measurements.

3.5 *Monitoring system*

The monitoring system was based mainly on the convergence measurement stations. A total of 120 convergence stations with 5 points set were installed to measure the 3D displacements on the primary support.

The frequency of the installation of the convergence stations was strongly dependent on the section types adopted:

- Sections type A: 1 station/40 m;
- Sections type B: 1 station/16 m;
- Sections type C: 1 station/8 m.

The geological survey of the tunnel face during the excavation completed the data acquisition in the convergence sections.

Deformation measurements of the tunnel face were carried out by a sliding micrometer along 30 m long, installed from the face in some special sections.

Some sections were installed to control the subsidence settlements in the low cover's zones, specifically where the clayey superficial deposits were encountered.

These solutions have been accompanied by the development of models and methods of calculation (Analytical and Numerical analyses).

A guideline concerning the procedure for the change of the sections type according to the encountered soil/rock masses and to the response of the installed monitoring system, completed the design.

4 THE FABRIANO TUNNEL: MONITORING RESPONSES

This paragraph summarizes some results in terms of convergences measured during the excavation's process. It must be noted that the maximum convergences have been measured almost always between points "4" and "5" (see, for example, Figure 6 for the position of the measurement points). Vertical displacements were extremely modest (maximum some centimeters even if the maximum convergences were of the order of some decimeters).

Figure 2 shows the maximum convergences measured along the profile of the tunnel. In the same figure the consolidation system locally applied is also summarized (section type). Figure 4 shows the same convergences as a function of the local cover.

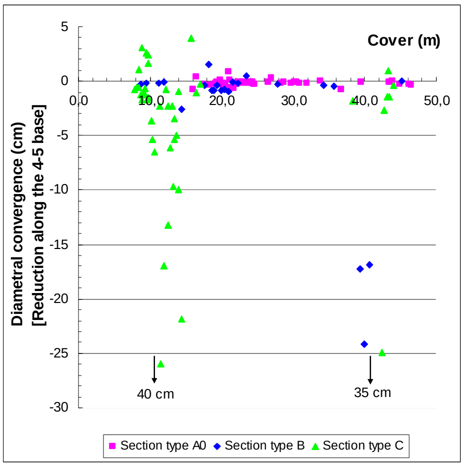


Figure 4.Convergence vs cover and sections type.

The maximum convergences measured on the rock masses where section type "A" was applied (mainly limestone), were on the order of a few centimetres independently of the cover. The reaction was always close to the elastic boundary.

For the sections "C" type, the behaviour always results in the plastic range with a very substantial convergence even at low cover. This is certainly due to the soil characteristics but also to uncertainty concerning the success of the preconsolidation in the context of extremely complex local geological situation.

In Figure 4 the majority of displacements registered in sections where the section type "C" was applied. In this case the convergences ranging between 27 and 35 cm for 40 m cover, were due to geological complexity which occurred in the transition zone between two different consistency rock masses.

It should be noted that stopping the progress triggered an increase of convergence under constant load that, given the type of material, is attributable to the development of the process of consolidation of cohesive material (as shown, for example, in Figure 5).

The reaction of the rock masses where the section type B has been applied is placed in the intermediate field at two points mentioned above (see Figure 4).

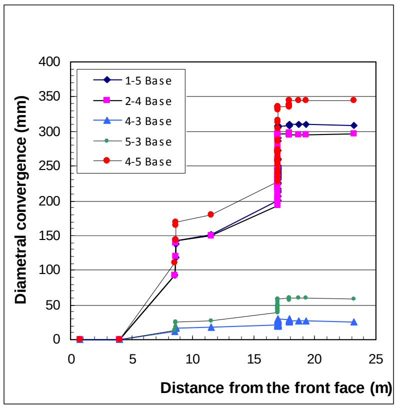


Figure 5.Typical convergences for the sections type C.

The convergences measured are typically small in absolute terms, but definetely always in the plastic field. During the application of this section type, movements (between 17 and 25 cm) were registered at around 40 m cover. These convergences are related to the presence of the transition zones between the stratified marly limestones and soft claystones. Even in these zones, the evolution of the response to the excavation has not been adequately controlled in its evolution and no further action was taken immediately to change the type of section using a more suitable section type (C).

It should be considered that the geological structural complexity where the excavation was made, led a sudden change in the section type in order to keep the convergences within the limits set by the project.

This change of excavation section is evident in Figure 2 between km 2+900 and km 3 +200, where section type has been changed 15 times. Many modifications have also been applied from km 3+600 to about 4+000.

Nevertheless, as mentioned, in some occasion it was not possible to keep deformation levels within fixed limits.

5 BACK ANALYSIS AND LINK WITH THE SECTION TYPE

5.1 *Introduction*

The definition of the support and consolidation system passes through the identification of geotechnical homogeneous rock mass zones along the underground alignment. For each homogenous zone, the analysis of the response of the excavated rock mass, as a function of the stress conditions, defines areas where these responses can be supposed to be similar and the same support section is associated.

The type of behaviour defines the consolidation of the rock mass and the support of the excavation, i.e. the "Section Type".

The evaluation of potential risk situations (asymmetric state of stress, dimension of the pillar between twin tunnels, limited covers, flowing, raveling, gas presence, etc.) complete the definition of the necessary measures to ensure the stability and excavation in conditions of safety.

The following points summarize the results of some DFM (Difference Finite Method) analyses with the aim of:

- To show the expected typical behavior for the homogeneous rock mass zones in different conditions, and to compare the main responses in terms of convergences and plastic zones for the different type of behavior (point 5.2);
- To compare the results of the FEM analyses with the registered data from the site (point 5.3).

Finally, point 5.4 illustrates an interesting comparison between the results obtained from a complete DFM model where each consolidation element was considered as a single structural element, with a model where the effect of these consolidation elements was simulated in a simplified way, by assuming a layer with improved characteristics. This simulation has been used to have a handy tool which can also give a certain sensitivity of the reaction of the rock mass to the excavation especially during the construction.

5.2 *Typical behavior for the homogeneous rock mass*

To illustrate the meaning of the defined typical behaviors for the homogeneous rock mass zones, the results of some DFM analyses, in terms of radial convergence, movements of the tunnel face and extension of plastic zones, for each defined behaviour of the rock masses (Type "A", "B" and "C"), are shown in this point.

The analyses were repeated introducing also the corresponding "Section Type" for consolidation and support system, to see their contribution to the equilibrium of the cavity.

For these analyses the geotechnical parameters summarized in Table 2 were taken into account. An elasto-plastic law, associated with the Mohr-Coulomb strength criterion, has been used.

For all cases a cover of 45 m, corresponding to the maximum cover above Fabriano Tunnel, has been considered.

The radial convergence is referred to the point "5", while the maximum longitudinal displacement of the tunnel face, defined as "extrusion", is referred to its centre.

Finally, based on literature data for similar conditions, a K_0 coefficient equal to 0.8 was assumed. Anyway, as at lower cover this value can be significantly lower, the DFM simulation for the all three typical behaviour using a K_0 equal to 0.5 and the same previous cover, were repeated to see its influence on the results.

Figure 6 summarizes the movements (radial convergences) obtained for an unsupported cavity, which indicate the "behaviour types". In this figure the same values calculated using the consolidation and support associate to the typical behaviour are also shown.

Similarly, Figure 7 summarizes the extrusion of the tunnel face with and without the corresponding consolidation.

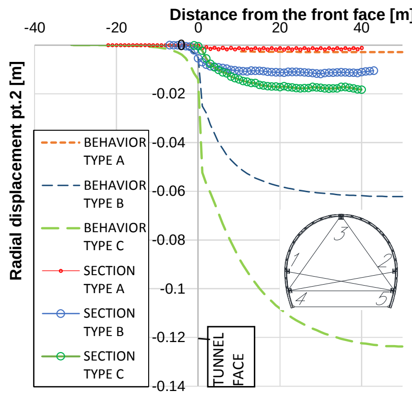


Figure 6.Radial convergence (free and supported cavity).

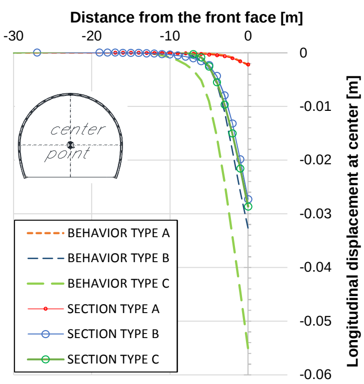


Figure 7.Extrusion with and without interventions.

For behaviour type "A" the effect of the consolidation system is negligible in terms of contribution for reducing movements (self-supporting rock mass).

On the contrary, as expected, behavior type “C” is characterized by important convergence phenomena and the contribution of the consolidation system is fundamental for reducing movements.

Behavior type “B” gives intermediate results in term of convergence, and also in this case the consolidation system is essential especially for the tunnel face.

The corresponding extension of the plastic zones is illustrated in Figure 8. In particular, this figure shows the evolution of the plastic zones for the three defined typical behaviour, starting from the tunnel face to the final equilibrium point. In the same figure the thickness of the plastic zone ahead the tunnel face is also shown.

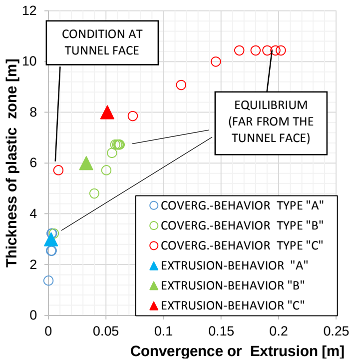


Figure 8.Convergence vs thickness of the plastic zone.

As anticipated, DFM simulation for the all three typical behaviour using a K_0 equal to 0.5, were repeated to see its influence on the results.

Figure 9 summarizes the point “5” movements (radial convergences) obtained for an unsupported cavity, which indicate the Behavior Types in case of K_0 equal to 0.8 and 0.5.

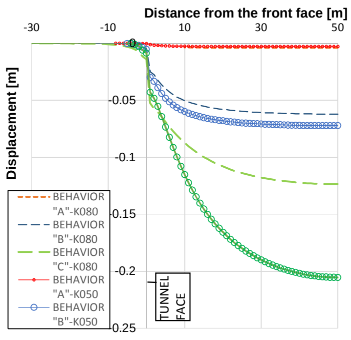


Figure 9.Excavation’s response for K_0=0.8 and K_0=0.5.

As expected, lower confinement gives more extended plastic zones (Figures 10, 11 and 12) and consequently greater movements of the cavity. It must be noted that in the examined case referred to the behavior type "C", the plastic zones even reach the surface which is indicative of a general potential collapse of the excavation (Figure 12).

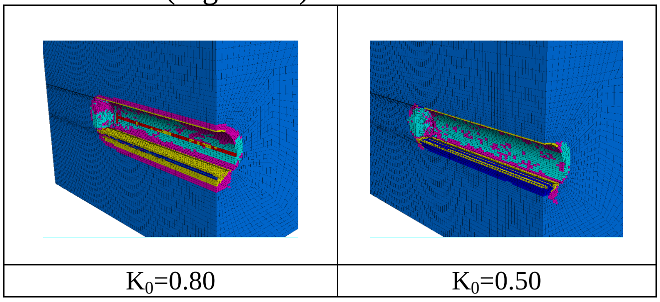


Figure 10.Behavior type “A”: Plastic zone’s extension.

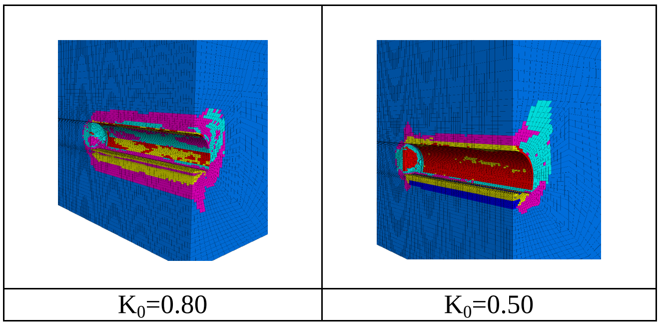


Figure 11.Behavior type “B”: Plastic zone’s extension.

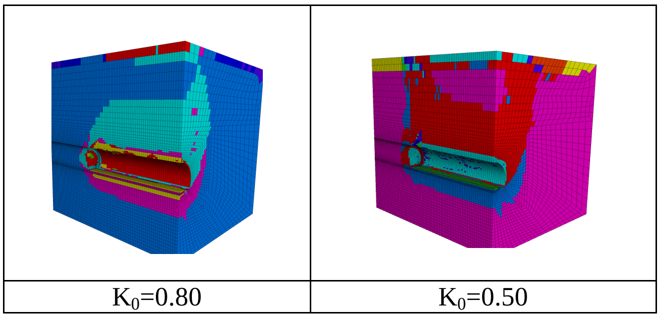


Figure 12.Behavior type “C”: Plastic zone’s extension.

5.3 *Expected and findings*

This point illustrates the comparison between the results of DFM models and the data collected during the excavation. In particular, the measured convergence data recorded in one Section Type taken as a reference section for each Behavior type, has been simulated using the 3D DFM model, with the aim to reconstruct the condition of the rock masses after applying the consolidation and support system. The measured data are referred to the reduction of the representative bases, i.e. reduction of the distance between points 4 and 5 (see Figure 6).

5.3.1 *Section type “A”*

A section with cover of 30 m has been chosen inside the Marly limestones. In general, for these sections type, the maximum measured convergences were of the order of a couple of cm.

The comparison between the measured in situ convergence and the result of simulation is shown in Figure 13. This comparison shows a good correspondence for the base between points 1 and 2.

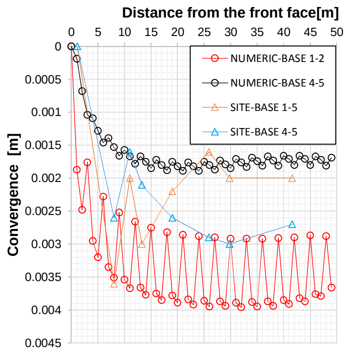


Figure 13.Section Type “A”. In situ typical convergences vs numerical results.

The extension of the plastic zone obtained by using 3D model is very limited giving an idea on the good state of stability of the cavity and of the tunnel face (Figure 14).

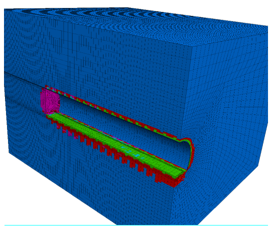


Figure 14.Section Type “A”. Plastic zone’s extension.

In situ, this section was definitely stable, and it was confirmed by the numerical analysis. The support system had the purpose to preserve the working site from loosening of blocks or from the decay of its properties.

5.3.2 *Section type “B”*

The observed in situ behaviour places this section between section “A” and “C”. This is normal as the behaviour can be considered as transition behaviour between the two related to section “A” and “C”.

For this reason, two different simulations have been done: ideally, one where the measurements place the "Section Type" close to the boundary of section type “A”, and one where the measured convergences place the Section close to the boundary for section “C”.

The results of the modelling are shown in Figures 15 and 16, respectively for the Section Type “B” with low and high values of measured convergences. In the first case the cover was 30 m, in the second case the cover was 45 m and the section was applied in a fault zone.

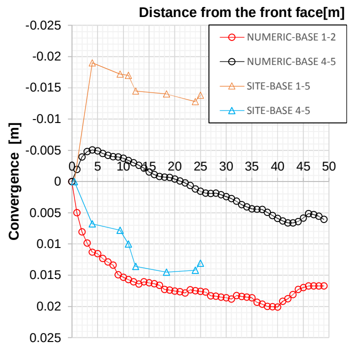


Figure 15. Section Type “B”. In situ typical convergences vs numerical results (first case).

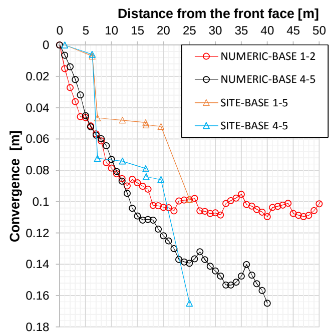


Figure 16. Section Type “B”. In situ typical convergences vs numerical results (second case).

The model has shown a good correspondence with the measured convergences. The excavation with behaviour of type B is associated with materials characterized by a certain degree of competence and stiffness and it is used also with high convergences, where a general stability condition is not compromised by the characteristics of the materials in relation to the state of stress.

The calculated extension of the plastic zones corresponding to the measured convergences shows that the maximum extension is on the order of 3.5 m, where the behaviour is comparable to the behaviour type “A” (Figure 17 on the left). Where the behavior is close to the one corresponding to a class “C”, i.e. where the measured convergences were of the order of more than a decimeter, the plastic zone extended up to 13.0 m (Figure 17 on the right). Anyway, results from the modeling, show that the use a consolidation elements at the tunnel face, can control the increasing of the plastic zone allowing to contain the development of the convergences.

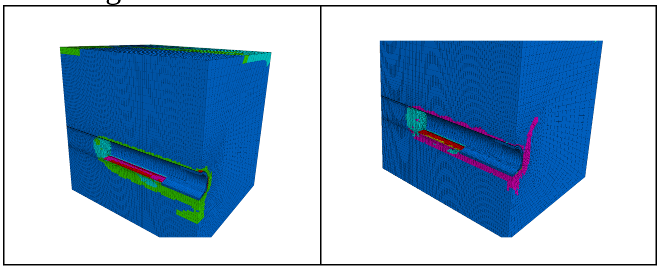


Figure 17.Section type “B” plastic zone’s extension (on the left the typical section, on the right the faulted zone).

5.3.3 *Section type “C”*

The application of the sections “C” type is associated with materials having very poor geotechnical characteristics. In the case of Fabriano tunnel, these characteristics were associated to a very low cover and consequent low level of confinement.

The back analysis examines a particularly critical condition with the presence of very weak and soft soil in the upper portion of the model, to simulate the real condition detected in site. The results in terms of convergences are shown in the Figure 18.

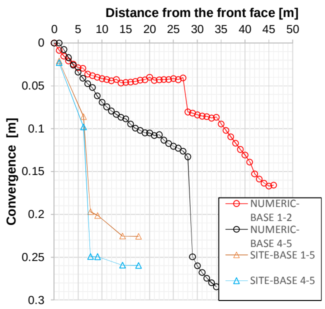


Figure 18.Section Type “C”. In situ typical convergences vs. numerical results.

The results of the model show that the plasticity of the rock mass is significantly wide, reaching the surface (see Figure 19). It must be noted that in this case the cavity is not stable without consolidation. These conditions indicate a collapse situation without interventions. The presence of face improvement allows the creation of longitudinal confinement limiting the deformations.

6 CONCLUSION

Some conclusion can be summarized as follows:

- The wide experience acquired in Italy so far, indicates that the approach for the design and the excavation of tunnel in Italy, is a good compromise between design, safety and production;
- Even in a very complex geological rock mass condition, like the condition encountered during the excavation of the Fabriano Tunnel, the system can be used conveniently;
- The final price of the tunnel can be defined as the system indicates since the design stage, the typology of the possible consolidation systems and its applicability range, according to the expected rock / soil mass conditions. A risk analysis due to possible geological variability, completes the terms of the possible variation of the costs.

Finally, each method is to be checked during the excavation phases. For this, the design must provide a procedure to manage the possible changes of the consolidation system if the reaction of the excavated rock/soil mass does not correspond to the expected one.

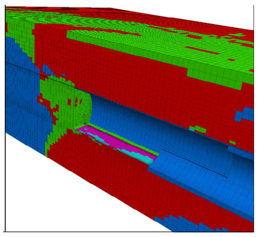


Figure 19.Section Type “C”. Plastic zone extension.

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