Observational approaches in tunneling: some thoughts

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Topic: Planning and Designing Tunnels and Underground

SUMMARY

Doubtless, tunnel design involves much higher uncertainties than any other civil engineering work. There is no other field where construction and design are so inseparable as in the underground structures. Often, the knowledge of the geological structure, the hydro-geological conditions, the mechanical properties and the behavior of the soil/rock is only fulfilled during the tunnel excavation. The need to draw up a solution based only on the knowledge available at the time of the design phase can easily be unsafe or uneconomic. In tunnelling the reduction of uncertainties progresses during excavation. This information can be used to modify the original design and construction procedures to comply with the actual conditions following an "observational approach".
This paper focuses on the key issues of the observational approaches and describes two case histories of application consistent with Peck's "best way out" approach.

***Keywords**: back analysis, observational method, Eurocode 7, monitoring system*

1. Introduction

In the period between the years 1940 and 1970, Terzaghi and Peck worked to give an alternative to the approach of that time that, to tackle geotechnical uncertainties, was carried out either by adopting an excessive and costly safety factor or by making generalised and unsafe assumptions. Noting that using the in situ observations it was possible to construct a safe and economic structure, they laid the basis for most up-to-date design approaches that make extensive use of observations and monitoring. Although the application of the observational approach dates back from the 1960's and it was often used intuitively by geotechnical engineers, Peck was the first in 1969 who tried to formalize the use and introduced the term "Observational Method".
Although there is an agreement largely unanimous on the concept that an "observational" approach could be advantageous, a full agreement has not yet been reached regarding its status and definition and there are no completely definite rules for its application.
Since the publication of the Peck's Rankine Lecture, several interesting contributions were published and there is an increasing interest for its application. However no decisive steps have been taken forward. Even existing national and international regulations lend themselves to different interpretations. That's why, so that the method can develop its full potential, it is necessary a further step leading to the agreement, among the insiders, of a set of clear and sufficiently detailed rules.

2. Possible design approaches

The design of an underground structure, also according to Eurocode 7 (EN-1997-1), can be addressed through the following approaches:
1. design by prescriptive measures
2. design by calculations
3. observational method

Design by prescriptive measures provides design solutions to problems based on experience from similar cases. Even today, experience-based systems, such as empirical rock mass classification systems, are common in rock engineering design. They should, however, be limited to the stages of preliminary design for rock masses of good geomechanical properties

Design by calculations is the first and prevailing approach; it includes the partial coefficient method and probability-based calculation methods. The final design is determined in advance of construction, based on conservative ground parameters that take account of uncertainties inherent in natural soil/rock. Monitoring is carried out only to verify assumptions regarding soil/rock conditions and to confirm that system behavior is within acceptable limits.

Observational method is presented as a suitable design approach for situations where soil/rock properties and geotechnical behavior are difficult to predict. It uses observations and measurements carried out during construction to actively adapt the final design to suit actual site conditions. The monitoring plays a very much active role in both the design and construction. It was developed in response to the need to avoid highly conservative design assumptions when faced with unavoidable uncertainties. Instead of relying on one single solution that is fully developed before the construction, work starts monitoring and follow up the actual conditions that can be used to modify and optimize the design.

The observational method should not be used where there is a possibility of a "brittle" behavior (in the structure which does not allow sufficient warning to implement any planned modifications) and in situations where a conservative design would imply a lower cost, such as in homogeneous rock.

3. Historical background and major contributions

The philosophy of the observational approaches can be traced back through history and the importance of making observations, in case of geological and geotechnical uncertainties, was documented in several texts. The oldest, often cited as an antique example, comes from the Phoenicians, who improved the design of a canal after observing that its sides were prone to fail when cut to steeply (Herodotus c. 430 B.C.). Peck (1969) was the first who rationalized this approach and coined the name "observational method" also formalizing guidelines. The origin of the observational method is often credited to Peck, although his paper actually is more of a synthesis and an attempt to formalize of a practical work approach called the "learn-as-you-go method", which was previously developed by Terzaghi. Nowdays the term "observational method" is loosely applied. It is often implemented in a subjective manner without refering neither to Peck's rules nor to specific code. Peck identified that it was necessary to have two designs compared with the traditional single design approach. A range of foreseeable conditions needed to be considered, which Peck associated with the "most probable" condition. He suggested a design starting with the "most probable" condition and varying the design if the observed behavior is worst than those predicted. This approach is fundamentally different to the approach described in later work (CIRIA 185) and in codes (EC7). The CIRIA 185 approach advocates starting with an initial moderately conservative design, to be relaxed to a "most probable" condition during construction, should the observed behavior warrant it. This approach is also known as "Progressive Modification" to the design and was first suggested by Powderham (1994) as a safer method.

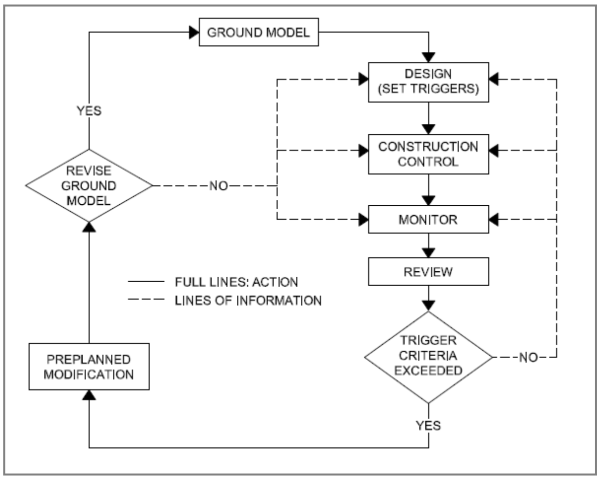

Fig. 1 - *Muir Wood 1987*

Muir Wood (1987) noted that the procedures for the Observational Method defined by Peck are cumbersome, and often impossible to achieve, for tunnelling. As a result of such considerations, recommended a simpler set of rules to apply to tunnelling, for a condition in which the need to modify the design might be expected to be exceptional (Fig.1):

1. Devise conceptual model
2. Predict expected features for observation.
3. Observe and compare against 2.
4. Are differences between 2 and 3 explained by values of parameters, inadequacy of 1 or inappropriateness of 1?
5. Devise revised conceptual model.
6. Repeat 2, 3, 4 and 5 as appropriate.

Such a procedure assumes that the design of the relevant feature will have conformed to the conceptual model (1), with predesigned supplementary work undertaken where the differences between (2) and (3) so require. He introduces the term Observational Design to define simplified approaches compared to Observational Method.

Stage	Phase	Description
Design	– Survey	– Analysis of existing natural equilibriums
	– Diagnosis	– Analysis and prediction of deformation phenomena (*) in the absence of stabilisation measures
	– Therapy	– Control of deformation phenomena (*) in term of stabilisation systems chosen
Construction	– Operational	– Application of the stabilisation instruments for controlling deformation phenomena (*)
	– Monitoring	– Control and measurement of deformation phenomena (*) as the response of the rock mass during tunnel advance (measurement of extrusion at the core-face and of convergence at the contour of the cavity and at varying distance from it, inside the mass of the ground)
	– Final design adjustments	– Interpretation of deformation phenomena (*) – Balancing of stabilisation systems between the core-face and the perimeter of the cavity

(*) Deformation phenomena in terms of extrusion at the core-face and of convergence at varying distance from it, inside the mass of the ground

Fig. 2 - *Lunardi 2008*

Lunardi (2008) and his ADECO-RS approach ascribe the utmost importance to monitoring. He was the first, since the early 1990s, to understand the primary role of the excavation face as a tool to constrain the behavior of the cavity and, accordingly, attributed to the face extrusion measurement a primary role for the understanding of the excavation behavior (before him reserved solely to convergence). He stressed that comparing predicted behavior with actual response measured is possible to perfect the construction methods specified in project.
He observes that during construction a monitoring system is essential: 1) to verify the hypotheses made in the diagnosis and therapy phases, 2) to deal with particular conditions not identified in the survey phase and therefore not specified in the therapy phase.
Numerous publications related to NATM make extensive reference to the observational method. However, the principles that enable its application are never specified in a scientific manner. Even according to Kovari Lunardi (2000), the NATM does not bring significant contributions to the topic under discussion.

In conclusion, there are other contributions that have dignity not less than that proposed by Peck and so we have to choose the most suitable for the specific case.

4. International codes

The European standard for geotechnical designs, Eurocode 7 (EN-1997-1) is one of the first design codes that presents the "observational method" as an appropriate design approach for situations where ground properties and geotechnical behavior are difficult to predict. When applying the observational method, the following requirements should be met before construction starts:

- *acceptable limits of behaviour shall be established;*
- *the range of possible behaviour shall be assessed and it shall be shown that there is an acceptable probability that the actual behaviour will be within acceptable limits;*
- *a plan of monitoring shall be devised, which will reveal whether the actual behavior lies within the acceptable limits. The monitoring shall make this clear at a sufficiently early stage, and with sufficiently short intervals to allow contingency actions to be undertaken successfully;*
- *the response time of the instruments and the procedures for analysing the results shall be sufficiently rapid in relation to the possible evolution of the system;*
- *a plan of contingency actions shall be devised, which may be adopted if the monitoring reveals behaviour outside acceptable limits.*

However, there are some drawbacks which if strengthened may result in a wider use of the method. The main drawbacks include the following:

a) it does not define the framework to be followed to manage the method within a contract
b) there is no indication of the need to perform computations
c) it refers to "acceptable limits of behavior" but it does not define how these may be derived.

Similarly to it, several other countries have stipulated in their regulations the possibility of employing the observational method. The regulations, which we have examined, have, however, indeterminate issues quite similar to those mentioned above. The Observational Method, as formal procedure, is still significantly underused in geothecnical practice and in particular in tunneling design. It is likely that, the cause of the underuse and the overall preference for predefined designs is due also to the the space left to the interpretation by the codes.

5. Handling of uncertainties

The predicted behavior of an underground construction depends not only on data, but to a larger extent on how we model different influencing factors. So, referring to project phase, the uncertainties can basically be classified into two categories: a) data uncertainty, b) model uncertainty.

Data uncertainty: represents both the natural variability existing in the data, the lack of knowledge about their exact values and the difficulty of evaluating them. On projects where there are complex geological and hydrological conditions, there may be unexpected variations in the ground conditions between boreholes. Uncertainties exist in the knowledge of the soil/rock characteristics, in the modelling of its behaviour and in the determination of the in situ state of stress. The most crucial factor is that the soil/rock as building material cannot be prescribed as all other types of building material as concrete and steel. Adequate and comprehensive site investigations are generally considered to be important for reducing data uncertainties. It is difficult, however, to provide general recommendations for appropriate methods and a suitable number of investigations. The types and number depend on the complexity of the project.

Model uncertainty: by necessity, models are incomplete representation of the real world. The uncertainty is generated by an incomplete understanding and a partial representations of the structure of the analyzed systems and the constituent interacting processes. For tunneling design it may be divided into uncertainty in the basic principles for describing the soil/rock and uncertainty in the interaction between the rock and the different engineered components. The uncertainties in the ground model increase with the complexity of the geological conditions. Model uncertainty is handled by attention to the problem identification and system analysis. The choice of the conceptual models is made on the basis of the individuals engineering judgment and experience.

The uncertainties can be reduced but not eliminated. In tunneling: length and depth of the structure, effect of in-situ stress, difficulty in accessing investigation areas, influence of the executive procedures contribute to complicate the issue. That's why, it is always necessary the control and calibration during construction and is inconceivable a tunnel without observation and monitoring. The reduction of uncertainties progresses during excavation. The design of the unexcavated part can be updated based on experience from the excavated parts and some of the design uncertainties are reduced.

6. Choice of design data and model formulation

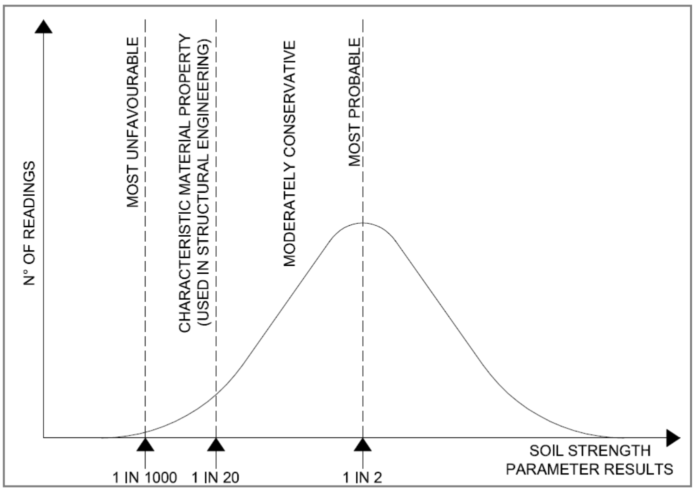

Fig. 3 *- soil strength parameters (CIRIA 185)*

The choice of the parameters of calculation is one of the most debated item. There are, essentially, two extremes approaches (fig.3):

1) based on ***'most probable'*** conditions. Contingency measures are prepared before construction and are implemented if observed behaviors exceed critical limits;
2) based on a "**most unfavorable**" set of parameters. Observations during construction are used to actively optimize the design.

There are, obviously, intermediate approaches. Peck adopted the "most probable" design and then reduced the design to "moderately conservative" parameters, where triggers were exceeded. CIRIA considers a "safer" approach to design by adopting a "progressive modification" of the design starting with the design based on moderately conservative parameters, and then reverting to most probable conditions through field observations. CIRIA uses the terms "most probable" and "most unfavorable" to describe the range of soil conditions as illustrated in fig. 3.

The "most probable" is a set of parameters that represent the probabilistic mean of all the data, although a degree of engineering judgment must be used in assessing this to take account of the quality of the data. The "most unfavorable" parameter represents the 0.1% fractile (it represents the worst value that the designer believes might occur in practice). The "moderately conservative parameter" (CIRIA 185) or "characteristic value" of geotechnical parameters (EC7) represents an "cautious estimate of the value affecting the occurrence of the limit state". It should ideally result in prediction of the upper 5% fractile. The "moderately conservative parameter" is not a precisely defined value. It is a cautious estimate of a parameter, worse than the probabilistic mean but not as severe as the most unfavorable. In assessing these parameters the designer should carefully consider the quality of the site investigation data and assess its suitability.

Wood (2000) observed that statistical evidence of geotechnical variability for a tunnel could rarely be presented in a significantly reliable manner to permit the designation of 'most probable' condition. The notion of the 'most probable conditions', presents problems since it would imply that a high proportion of the work would require modified design and additional work in order to suit the actual conditions. With reference to a simplified case he shows that the initial provision should be safer than that 'most probable' condition.

In the observational approach, the suitable limits of behavior, is a "serviceability" calculation. These provide the predictions against which the field performance can be monitored and reviewed.

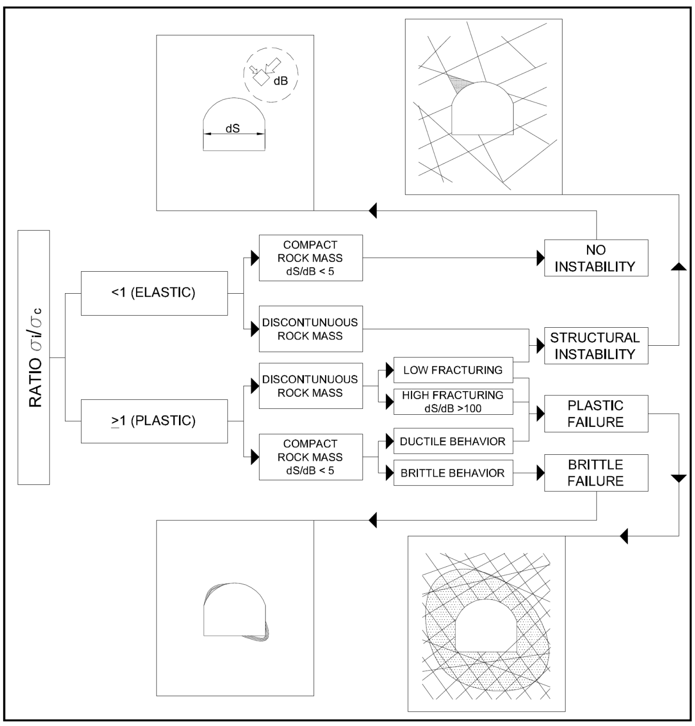

Fig. 4 *Selection of computational model/failure mechanism*

The definition of a model represents the simplification and rationalisation of the data generated by the site description. The scope is to explain the principal items which will be expressed in the stress/strain behaviour of the model. For instance, mechanical properties of the rock mass are identified by mean values, major structural features are assigned a regular geometry and average shear strength properties, and a representative specification is accepted for the in situ state of stress. The need arises from the limited details that can be accommodated in the analytical or numerical methods. It is clear that considerable discrepancies may be introduced at this stage, by lack of recognition of the engineering significance of particular features of the geomechanical situation.

Sometimes, designers rush into carrying out complicated analyses using sophisticated methods that require input data, knowledge of which is very uncertain. There is therefore a mismatch between the sophistication of the method of analysis and the lack of knowledge of the input data. The use of sophisticated calculation models can lead to the false self-assurance that a good design analysis has been carried out and can lead to "false certainty". For the selection the computational model, the key issue is to recognize the failure mechanism that governs the behavior of the cavity under construction. In figure 4 is shown a flow chart for this purpose.

7. Monitoring

It must be observed that the use of an observational approach requires a reliable system for information management, as all actors must be kept updated. Without going into details of the choice of the monitoring system, some points that should be taken into account are proposed.

An important part of the effective implementation of the observational method is the selection of representative observation parameters, parameters which are both possible to predict and monitor. An appropriate observation parameter should, according to Powderham (1994), be comprehensive, reliable, repeatable and simple. Control parameters should yield relevant answers concerning the acceptable behavior of the construction and if they are irrelevant and should be stopped monitored. The results from the measurements must be given in time in order to confirm predictions and make it possible to implement contingency measures in time.

It is common practice to establish 'trigger values' for key measurement parameters associated with a project, for example displacement. If these values are exceeded then certain actions need to be clearly defined. Two trigger values are normally established:

- warning value (amber limit): this could be a pre-determined value or a rate of change in a parameter that is considered to indicate a problem;
- alarm value (red limit): this could be where threshold values for safe operation are exceeded. This should involve pre-determined action.

The results of the monitoring shall be assessed at appropriate stages and the planned contingency actions shall be put in operation if this becomes necessary. Limit values with respect to the monitoring system at first defined must also be provided and so during construction additional monitoring shall be undertaken if this becomes necessary.

Measurement uncertainties are also important to consider. Far too often measurements are believed to be exact, which they unquestionably are not. One should always be aware of the uncertainty of the measurements and include it by describing the measurement result as a stochastic variable. The measurement uncertainty is partly statistical, and partly the result of measurement technique, including both the instruments and their application.
The statistical uncertainty can be reduced by repeated measurements, as it is a sampling problem and as such can be handled using statistical sampling theory. The instrument and application uncertainties are more difficult to reduce as they may not be as obvious. Using recognized instrumentation and techniques should reduce these uncertainties. One must acknowledge that there are limits to the resolution of the measurements, which is caused by measurement noise.

8. "Best way out" approach

According to Peck, two main approaches are possible:

- "ab initio" approach, adopted from inception of the project; it is planned from the start of work;
- "best way out" approach, adopted after the project has commenced and some unexpected event has occurred; it is used to establish a way of getting out of a difficulty during work.

Although almost all underground design provides the use of a robust monitoring system, the application "ab initio" is very rare and examples of application are related to the "best way out" approach. This approach can be broken down into four steps:

1) data collection and review. Collect all available data to define the behavior of the structure for use in the back analysis. Particular emphasis should be placed on understanding the actual conditions and behavior operating in the field, rather than justifying the original design assumptions. Sources of data should include: soils data and stratigraphy, construction records, actual sequence of events to inform back analysis process, and observations and physical measurements leading up to the unexpected event.
2) Back analysis. Refine the understanding of the actual behavior of the structure and reduce uncertainty in the design. The process involves: establishing most probable parameters; developing a satisfactory model; comparing results with monitoring data and field observations; revising parameters if good agreement is not achieved; and once a reliable model has been produced, proceeding to design.

3) Verify/modified design. Predict the future behavior using the realistic model and set of parameters developed from back-analysis, for the remaining construction stages.
4) Output plans and triggers the process has to be agreed with all stakeholders and management teams, with appropriate contingency and monitoring plans and setting up of trigger values.

9. Examples of works

In the next chapters some recent experiences, where the authors were involved, are presented. They are experiences of tunnel construction in difficult geomechanical conditions in which there was a successful refinement of the project according to the analysis of the monitoring data.

Serra Rotonda tunnel - Highway A3 Salerno-Reggio Calabria
(Client ANAS S.p.A. - General Contractor: Grandi Lavori Fincosit S.p.A.)

The Serra Rotonda tunnel has a length of about 3800 m and is the longest one of the Salerno-Calabria highway. It is a twin bore tunnel, each bore has an excavation area of approximately 175 m^2.

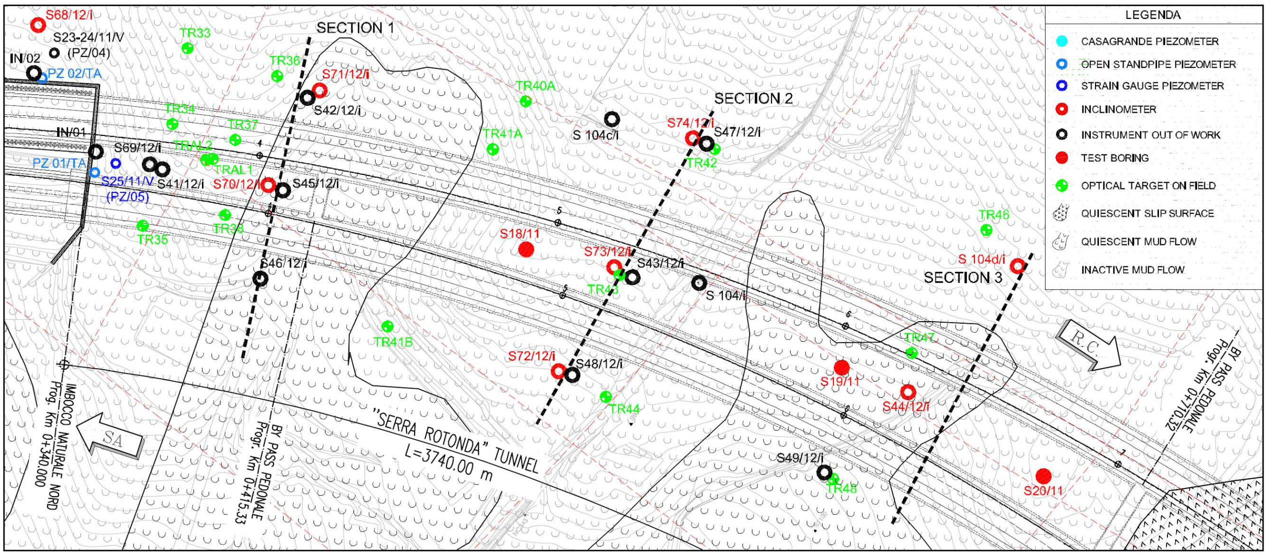

***Fig. 6** Serra Rotonda monitoring plan*

At the north entrance, for an extent of about 600 m, we find a stretch that presented special challenges.

It is characterized by the presence of a tectonized clay shale rock mass with poor geomechanical behavior and a large area with shallow landslide phenomena that are under-passed with a cover variables between 10m and 20m.

A further difficulty was due to the possible presence of gas: so all construction phases were executed by explosion-proof systems. In order to verify the effectiveness of interventions and allow their calibration an important monitoring system both at the surface and inside tunnel has been set up (fig. 6, fig. 9).

The design of stabilization measures has been carried out on the basis of back analysis conducted on the stability of slopes, overlooking the tunnel portal, in limit equilibrium conditions. This analysis highlighted the need to execute, for the tunnel dig, preconfinement interventions able to minimize the disturbance caused by the excavation, to the already critical equilibrium conditions of the slopes and able to minimize the extension of the plastic zone at the face and around the tunnel.
This objective was achieved by imposing very strict limits to the deformations of the rock mass and in particular the respect of the elastic limit for the stress-strain ground conditions subsequent to the excavation.

Fig. 7 *Face excavation*

The design of stabilization interventions was carried out with the parameters resulting in back analysis with reference to the "most probable" value. The preconfinement interventions consisted in the reinforcement of the core and of a shell around the future excavation by fibre-glass tubes injected with grout (fig. 7, fig. 8).

The monitoring system, in the most critical stretch, consists of: extrusion and convergence measurements, strain gauges on the temporary lining, inclinometers and benchmarks arranged according to the scheme of figures 6 and 9. During the first stretch, despite the important measures of the preconfinement, very high (up to 50 cm) surface subsidence and inclinometers displacements (up to 30 cm) have manifested (fig. 10, fig. 11).

The examination of the monitoring data has allowed a constant adjustment of the interventions of preconfinement and support. This enabled not to change the equilibrium conditions existing prior to the excavation and to achieve the stabilization of the deformation of the rock immediately after the passage of the excavation face with the implementation of the preliminary lining.

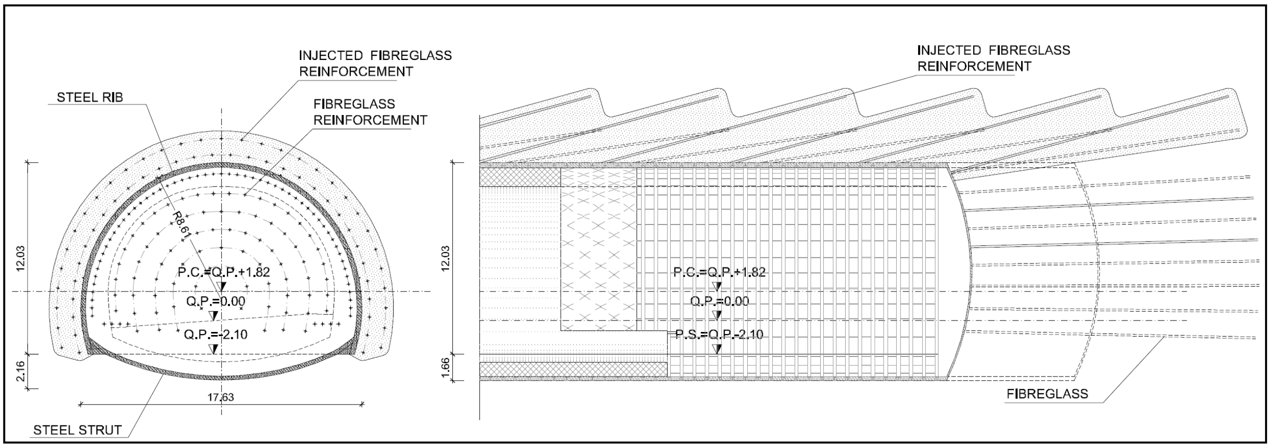

Fig. 8 *Serra Rotonda typical section*

A A

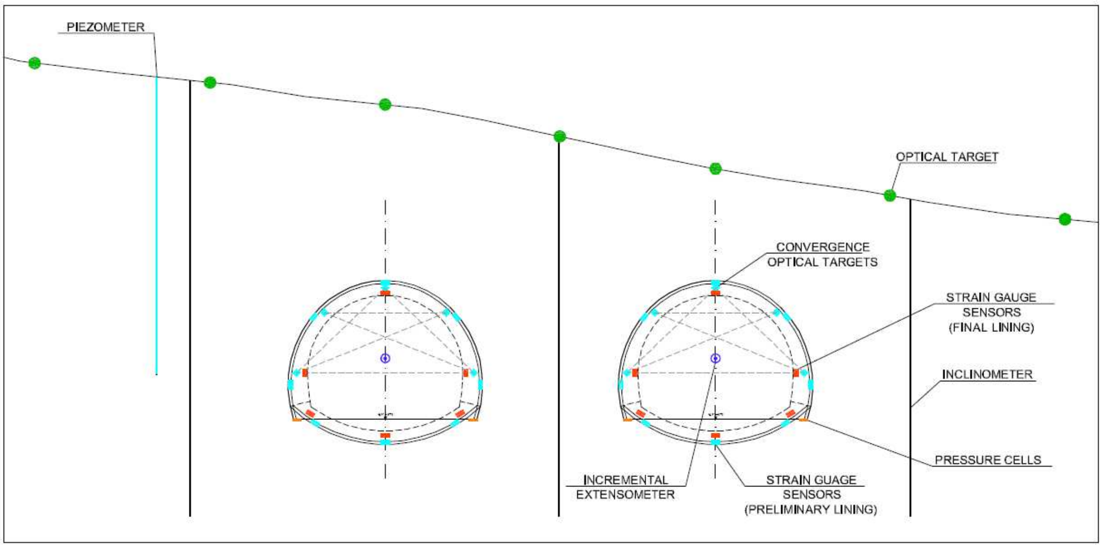

Fig. 9 *Serra Rotonda typical monitoring section*

summary of the inclinometer and topographical data relating to this stretch is shown in fig. 11.
In the section n.1 (INS41) we read displacement maximum of about 30 cm. In the section n.2 (INS42/INS71, INS45/INS70, INS46) we read a maximum displacement speed (at the face passage) of about 9 mm/g. The maximum displacements were comparable to those of the first alignment. In the section n.3 (INS47/INS74, INS43/INS73, INS72) we read a maximum displacement speed (at the face passage) of about 2 mm/g. The maximum displacements was about 9 cm.
Proceeding with the excavations southward, with the increase of depth and moving away from the area more disturbed by shallow landslide phenomena, the analysis of the monitoring data revealed a progressive improvement in the deformation response to excavation in terms of gradient and in absolute value.

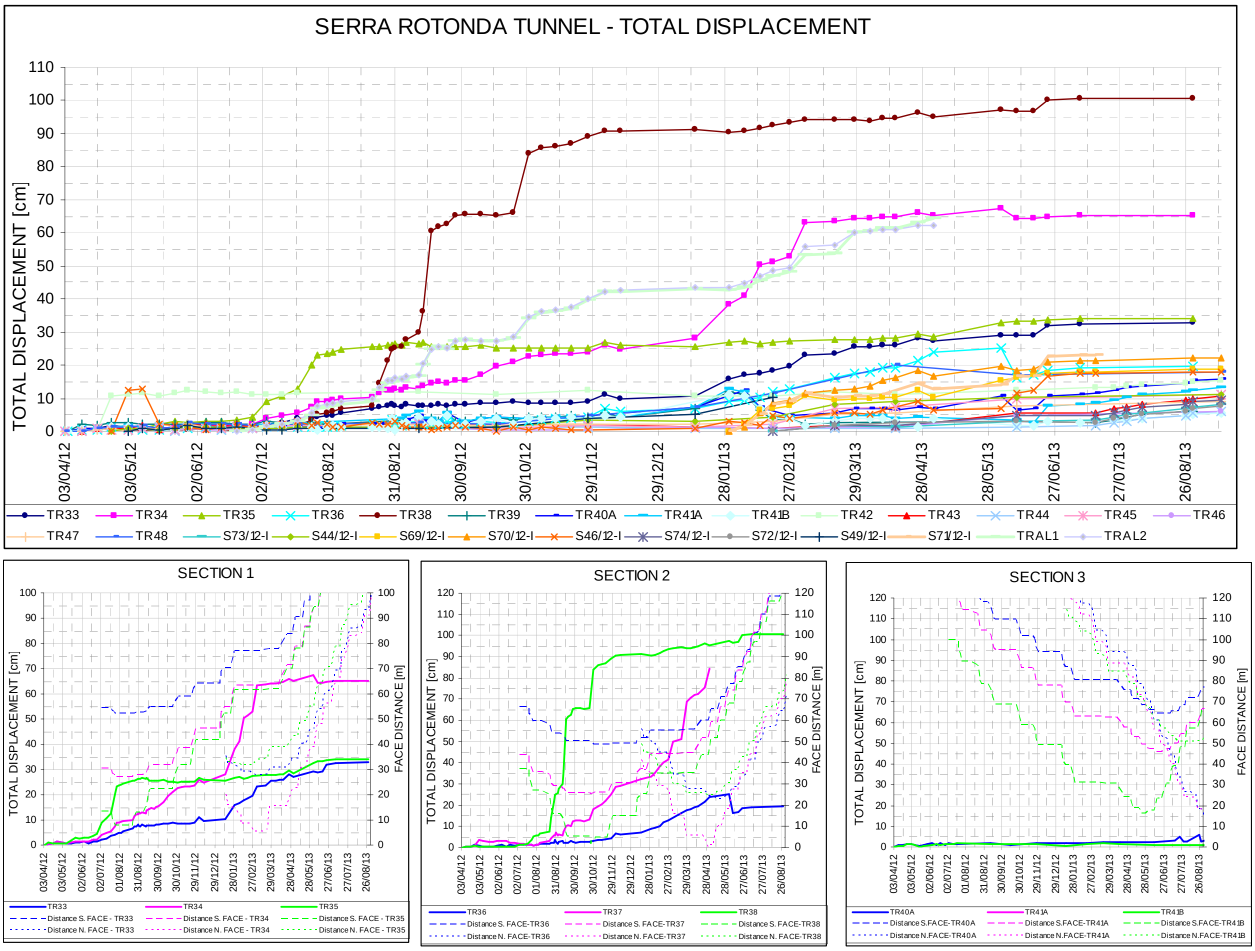

__Fig. 10__ Serra Rotonda total displacement on field

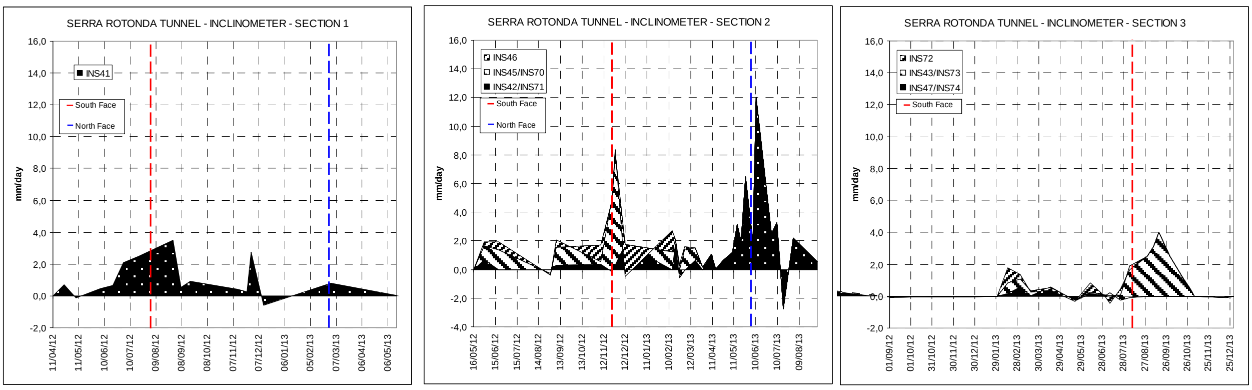

__Fig. 11__ Serra Rotonda inclinometer displacement

The continuous refinement of the project involved the preliminary lining (shotcrete shell+steel ribs) and the preconfinement interventions.

The analysis of the monitoring data and in particular the results of the stress readings (strain gauge) (fig. 12) has enabled the optimization of steel ribs proceeding southbound.

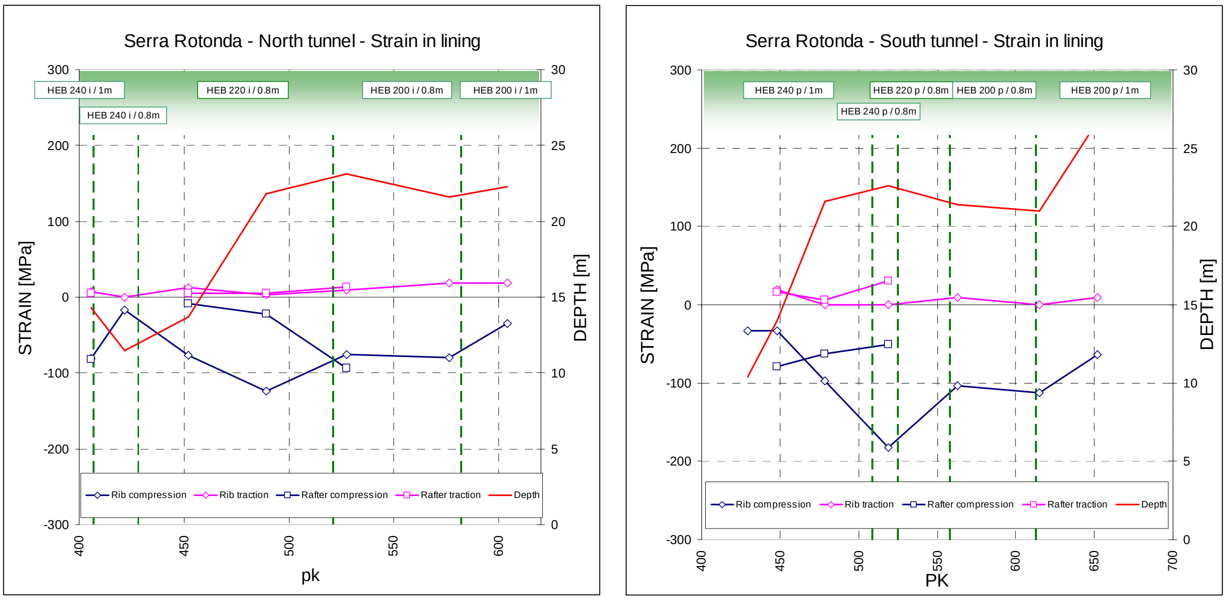

Fig. 12 *Serra Rotonda strain in lining*

Through the observations collected during excavation it was found that the stability of the face was strongly influenced by the conditions of local stability of small single portions; this especially in case of zones of pronounced tectonic disturbance encountered during excavations. This was despite a situation of global stability always ensured by the actions of preconfinement provided (maximum extrusions were equal to 8 cm).

In the early stage of excavation there were several episodes of localized instability caused by movement of interlocking blocks that show a significant weakening in unloading conditions.
They were immediately confined by the workers at the face; otherwise, they could trigger progressive rupture phenomena capable of involving the entire cavity.
For this reason, we proceeded constantly refining the intervention of confinement of the excavation face. They were analyzed simultaneously the conditions of global stability, guaranteed by the overall strength of the intervention of stabilization, and those local type, conditioned by diffusion and geometry of the intervention. It was done by modulating length, density, overlap and geometrical distribution in order to ensure the required stability conditions (global and local) in all geomechanical and geostructural conditions.

Through the analysis of the recorded data (convergences and extrusion), preconfinement and support interventions (fig. 8) have been gradually reduced proceeding with the excavations.

National Road S.S. 106 JONICA – Pantalogna, Schiavo 1 and Schiavo 2 tunnels

(Client ANAS S.p.A. - Construction Company: De Sanctis S.p.A.)

The tunnels (Schiavo 1, Schiavo 2, Pantalogna) of the SS106 variant of Marina di Gioiosa Jonica (CZ) are excavated within a Plio-Pleistocene marine formation lithology with pelitic-silty.

In the case of tunnels excavated into a clay mass the greatest difficulty is the need to analyze the different drainage conditions that can occur: 1) short term (undrained), 2) a transitional regime in which the pore pressure evolves towards a equilibrium condition, 3) long-term (drained). The occurrence of one of these conditions depends on the permeability of clay mass and the rate of

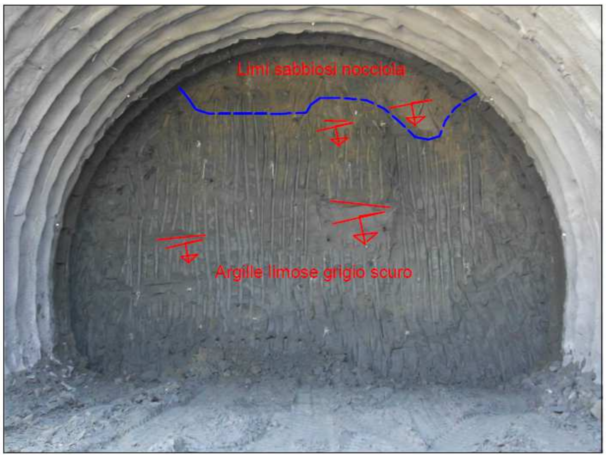

Fig. 13 *Pantalogna face condition*

advancement of the face. In the project under review, the presence of random sandy levels (fig.13), which promote the transition from undrained to drained conditions, did not allow the detection of in advance drainage conditions to be considered in design of the works.

All this influenced in a decisive way the definition of stabilization measures. Hypothesizing, in the design, the excavation in drained (most unfavorable) conditions would led to a considerable increase in the construction costs. The choice was to design the tunnel using undrained parameters (most probable) and manage by a "robust" monitoring such decision, so to calibrate and optimize continuously the stabilization actions.

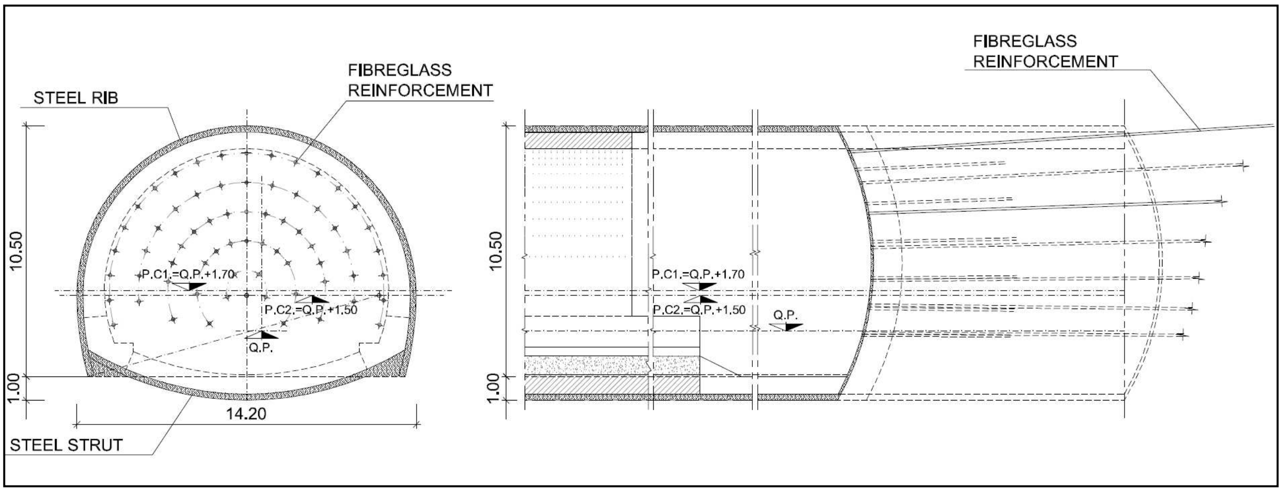

Fig. 14 *Gioiosa Jonica tunnels typical section*

The scheme of the preconfinement and support interventions used is reported in fig. 14.
Through the monitoring data collected during excavation (convergence and extrusion measurements, strain gauges in the preliminary lining) it was possible to refine stabilization interventions and adapting to the behavior actually encountered.

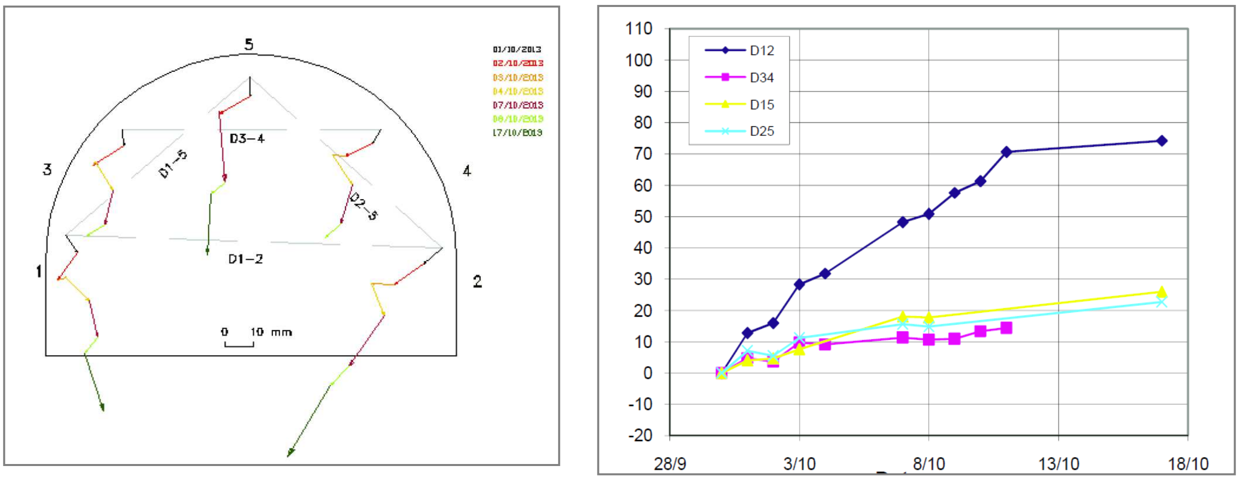

Fig. 15 *Gioiosa Jonica covergence monitoring*

In various stretches it was necessary to integrate the preconfinement interventions at face and proceed to the early closure of the preliminary lining with the invert because the measures of deformation exceeded the forecasts made in the design phase. Several times, it was found that the design assumptions of undrained (most probable) behavior was not guaranteed and the actual conditions were instead of transition between the undrained and drained ones (most unfavorable).

This was kept under control by comparing the measures of convergence and extrusion of the face with the threshold values identified in the design phase (fig. 15, fig. 16).

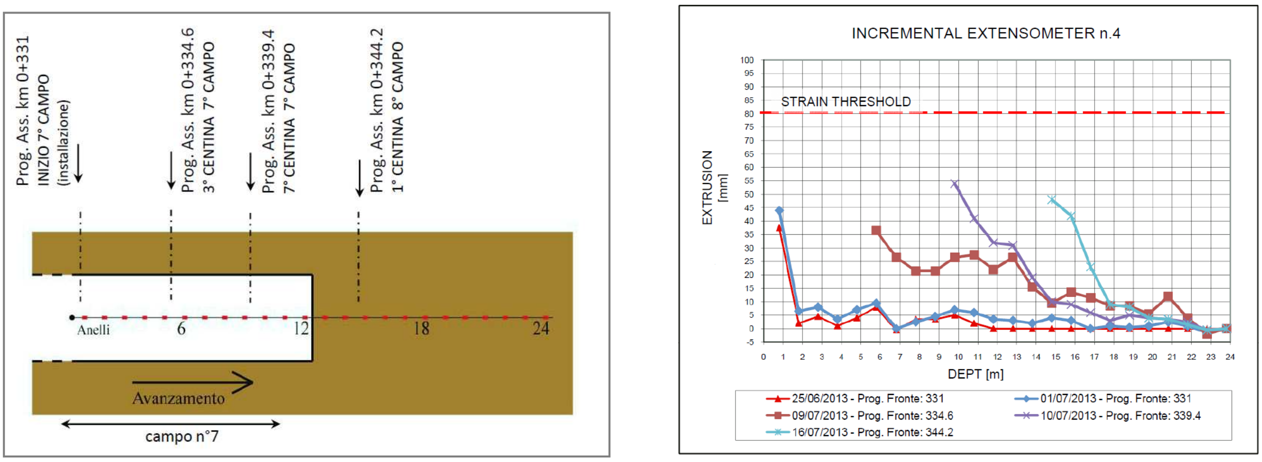

Fig. 16 *Gioiosa Jonica extrusion monitoring*

We show the approach of calibrating the project in a stretch of underpass of an inhabited center; this was based on the analysis of the measurements of surface subsidence.

Several (with spacing of about 20 m) were placed in the area concerned by the buildings (fig. 17). On the basis of the settlements measured in the stretch prior to that affected by the buildings, profiles of longitudinal subsidence have been reconstructed; namely, subsidence percentage characteristic of each point, depending on the distance of the face from the same point (fig. 18).

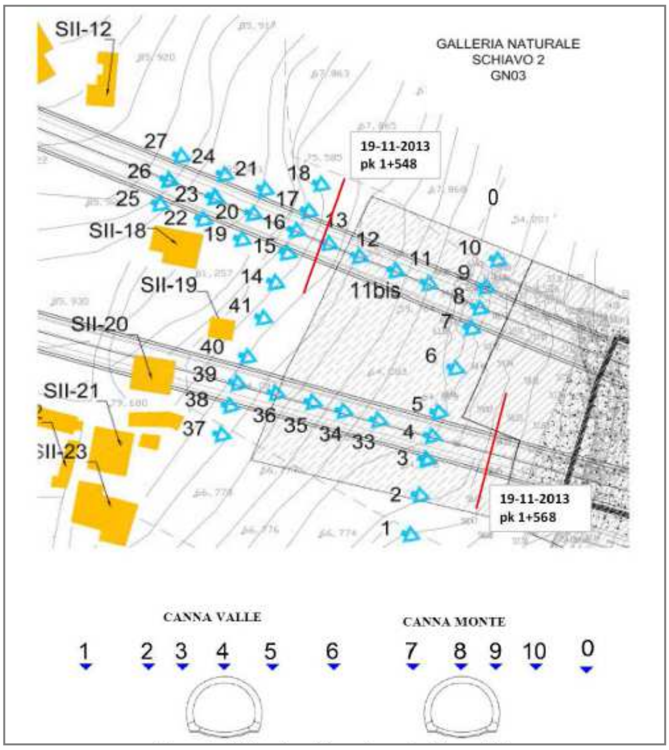

Fig. 17 *Schiavo 2 monitoring plan*

The profile of average subsidence so calculated, and continually refined with each advancement on the basis of new data gradually acquired, was considered as a reference for estimating future settlements.

During the excavation the prediction of subsidence on the buildings has been constantly updated according of the results arising from the monitoring.

With reference to most critical stretch, in the graph in fig. 18 are compared the magnitude of measured settlements (on axis) with the thresholds of project forecasts. Are displayed:

- attention threshold, equivalent to the maximum settlement predicted by calculation;
- alarm thresholds, with respect to the structural damage and aesthetic damage.

The graph also shows the percentage of subsidence developed for the benchmarks which have not yet ran out of subsidence (calculated on the basis of the basin longitudinal reference previously defined).

The settlements were always higher than the values calculated in the project (attention threshold - in undrained conditions) indicating a condition in evolution to a drained condition (most unfavorable).

The constant adjustment made to the stabilization measures (intensity of the consolidation of the front, inertia of the preliminary lining steel ribs) has enabled control the subsidence within values that will not cause structural damage to any of the buildings interfering.

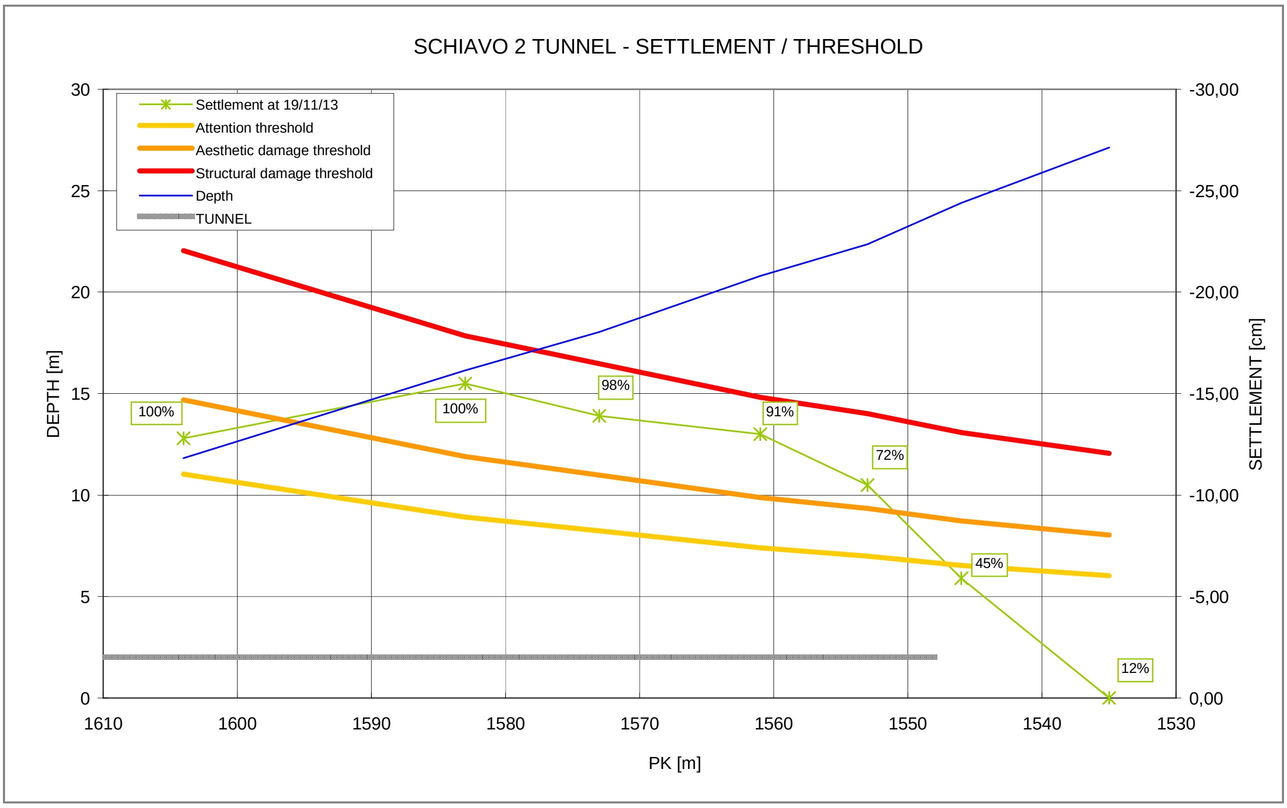

Fig. 18 *Schiavo 2 subsidence thresholds and settlement expectation*

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