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Observational approaches in tunnel design as method to reduce risk

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Summary

Tunnel design is a practice involving countless uncertainties and many failure modes.
Often, the knowledge of the geological structure, the hydro-geological conditions, the mechanical properties and the behaviour of the soil/rock is only fulfilled during construction.
The reduction of uncertainties progresses during excavation. For this reasons, a solution based only on the knowledge available at the time of the design phase can be unsafe or uneconomic. Conversely, information collected during excavation can be used to adapt the original design to comply with the actual conditions following an "observational approach".
This paper focuses on the key issues of the observational approaches and describes the case history of a tunnel of the Salerno-Calabria highway (Italy); it was excavated under-passing an area with shallow landslide in tectonized clay shale rock mass with poor geomechanical behaviour in presence of gas risk. In order to deal with this risks, an significant monitoring system was set up.

1. Tunnel risk: key issues

Doubtless, tunnel design involves much higher uncertainties than any other civil engineering work. A lot of tunnel projects reported significant cost overruns and contractors claims in percentage much greater than other civil construction works.
To be underground means that the context is difficult to know.
To realize what " difficult to know" means we have to compare a tunnel with a surface structure.
Even in a comprehensive exploration program we can test only a small fraction of the total ground to be affected by the tunnel construction. We can recover a relatively miniscule drill core volume that is less than 0.0002 % of the volume of rock mass that one has to consider in tunnel design. On the other hand, we have to think that for concrete structures controls are every 20-50 mc that means nearly 0.02% of the volume of a structures.
The second issue that we have to underline is that for the tunnel designer, the soil/rock surrounding a tunnel is effectively a construction material. It is an integral part of the final structure and plays a pivotal role in its stability; even if a tunnel structure often needs support systems made up of concrete and steel, it is the ground that is the major part of the structure, and this can have both a supporting and a loading role. Moreover, tunneling is not a question of gradually assembling materials (steel, reinforced concrete, etc.) with well known strength and deformation properties as with surface constructions; in tunneling one has to intervene in a pre-existing equilibrium and proceed in some way to a "planned disturbance" of it in conditions that are only known in part. So, the calculation model itself is not finally correct, but includes uncertainties. These can result from deliberate simplifications, both in the description of the problem, e.g. its geometry, and in the choice of which type of model should be applied.
For all these reasons, ground conditions account for the largest element of technical and financial risk in tunneling projects.

2. Predefined design: drawbacks

To deal with these uncertainties, the typical approach, known as 'predefined design', is to attempt to eliminate the uncertainty by assuming a conservative conditions, derived from the data available prior to construction. The design, also according to Eurocode 7, can be addressed through the following approaches:
1. design by prescriptive measures
2. design by calculations
Monitoring is carried out only to verify assumptions regarding soil/rock conditions and to confirm that system behaviour is within acceptable limits.
Design by prescriptive measures provides design solutions to problems based on experience from similar cases and should be limited to the stages of preliminary design for rock masses of good geomechanical properties (BTS 2004).
Design by calculations is the most widely used approach and includes various possible methods:

- the safety factor method
- semi-probabilistic method (partial factoring).
- the probability-based calculation method.

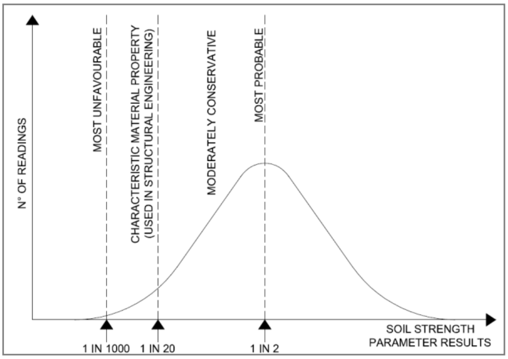
In all three cases, the final design is determined in advance of construction, based on ground parameters that take account of uncertainties inherent in natural soil/rock.
As main drawback we have to note that, this conservative approach can easily leads to poor value design due to the wasted resources of over design or with the additional extra costs of dealing with 'unforeseen ground conditions'.
The safety factor method is sustained by a vast amount of experiences. It introduces a safety factor which is defined as a ratio between total resistance and total force, and should be kept a value more than 1 given by experience. It has some weaknesses. The main is that the safety factor is not an absolute measure of the safety of a structure and gives only a partial representation of the true margin of safety that is available. Through standards or tradition, the same value of factor of safety is applied to situations that involve widely changing degrees of uncertainty. In theory, large uncertainties require large safety factors. However, the safety factors established in design codes are not calibrated to consider different ranks of uncertainty; thus, equal safety factors do not necessarily imply equal safety level. Therefore, a large safety factor might be applied without necessity, even if the level of uncertainty is low, or, worse, a too low safety factor can be applied in a case with great uncertainties.
Many design codes in the world today are in the process of revision from traditional "safety factor method" to Limit State Design (LSD) or Level I Reliability Based Design (RBD).
The reliability of a structure is estimated based on a criterion that splits desirable state (stable condition) from undesirable state (failure). The ultimate and the serviceability limit states are examples of the typical limit states employed in RBD. The uncertainties involved in basic variables concerning actions, material properties, shape and size, etc. are taken care by factors which are determined based on reliability analyses. In this case, the necessary safety margin is preserved by applying partial factors to characteristic values of basic variables. All the calculations are done deterministically; the designers, users of these factors, need not to have knowledge in reliability theory. Conversely, code writers need to fix the factors based on reliability analysis. For geotechnical structures, this approach has two main weaknesses (Honjo 2009):
- in modeling the uncertainties of forces and resistances, it is the tail part of the distribution that is really affecting the reliability of a structure. However, it is very difficult to accurately model the tail part from limited data.
- the design calculation method is always a simplification and idealization of real phenomena. It is rarely possible to evaluate the model uncertainty involved in this kind of method. Thus, the calculated failure probability cannot be an accurate indicator of the failure event.
At first sight a probability-based calculation method appears an objective way to measure the distance to failure. Assuming that soil parameters can be modelled in the probabilistic framework, and provided that the probability distribution functions of load and resistance, as well as the correlation functions of the shear parameters are known, the failure probability could be a good meas-

ure for the distance to failure. However, there are too many unknowns and crude estimations in such calculations, so that the failure probability is just another qualitative indicator for safety. Moreover, this demands a high statistical expertise. Attempts by statisticians to tackle geotechnical design have often failed and it is very difficult for one person to have a sufficient grasp of both disciplines so to combine them sensibly (Fellin et al. 2005).
The conclusion is that we cannot tackle the project of a geotechnical structure and even less of a tunnel with classical structural engineering methods.
The main point that we have to remember is that the input of a geotechnical calculation, whatever method is used to determine the parameters, is a cautious estimate and other uncertainties are ineradicable, first of all model uncertainty, so the result of the calculation is in any case an estimate.
In 1945 Terzaghi stated: *"In the engineering for such works as large foundations, tunnels …. Many variables … remain unknown. Therefore, the results of computations are not more than working hypotheses, subject to confirmation or modification during construction"*.
Despite the enormous developments in geotechnical theories and computational engineering since 1945, the variability of the soil and the model uncertainty will never vanish; tunnel engineers will always suffer from a lack of information. Thus even the result of the highest sophisticated numerical model will be more or less a crude estimate.
In spite of all numerical predictions it is indispensable to judge the soil/rock properties and to observe the construction behaviour continuously during the construction process.

3. The observational approach

We have seen that tunnel design involves much higher risks than any other civil engineering work, luckily there is an important aspect in tunneling from that we have to take advantage.
Since a tunnel is excavated step by step, usually from one portal, we obtain a lot of information on tunnel behaviour during construction and "we learn as we go" so we can improve the original design step by step during excavation.
The basic principle is that in any process where there are uncertainties, the most efficient way to manage these uncertainties is by monitoring performance of the process, considering options and taking corrective action when necessary. It uses observations and measurements carried out during construction to adapt the final design to suit actual site conditions.
This approach recognizes that uncertainty has two components, risk and opportunity: the ground conditions could turn out to be better than expected, or worse. The approach is based on a "predefined design" that is improved step by step during excavation. During the design implementation a rigorous monitoring and observation strategy is used to check and confirm the actual conditions found during construction. Performance indicators are selected for monitoring, related to the critical risks.
The main advantage of this approach is that we can constantly be aware of real ground behaviour from monitoring. To ensure success of such a flexible design approach, the processes of design and construction need to be integrated and close co-operation is required between all those involved in the design and construction process.
While designing, less conservative ground parameters have to be applied. Its application within a risk management framework pursues cost-effective ground risk remediation, while chasing hidden opportunities as well. Contrary to the concept of risk cause reduction, which is typically applied during design, risk effect reduction will only cost money when the risk indeed occurs, besides the costs for the (additional) monitoring and fall-back scenarios.
Also, hidden opportunities of better ground behaviour than expected, which are revealed during construction, may be used (Van Staveren 2006).
It is important to highlight that the objective of an observational approach is to deal with uncertainties, and not reduce factors of safety.

The choice of the parameters of calculation is one of the most debated item.
There are, essentially, two extremes approaches (fig.1):



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.

Fig. 1 *- soil strength parameters (CIRIA 185)*

The "most probable" is a set of parameters that represent the probabilistic mean of all the data. The "most unfavorable" parameter represents the 0.1% fractile (it represents the worst value that the designer believes might occur in practice). In assessing these parameters the designer should carefully consider the quality of the site investigation data and assess its suitability.
Assuming that we have also established an acceptable probability of failure.

If we design based on the "most unfavourable" set of parameters and apply a similar factor of safety that we would for the most probable condition, we will fall in the first category described by Peck (1969), i.e. a wasteful or over-conservative design (Figure 2). On the other hand, designing for the "most probable" decreases the degree of conservatism and would be more economical (Figure 3). However, as Peck (1969) stated, it is, to some extent, a gamble. If not there would be no need for the word "probable". By adopting an observational approach and a working hypothesis of the most probable conditions, one could say that it is at least a risk-controlled "gamble". If we properly monitor the ground behaviour during excavation, contingencies may then be put in place if we realise that our "gamble" has failed.

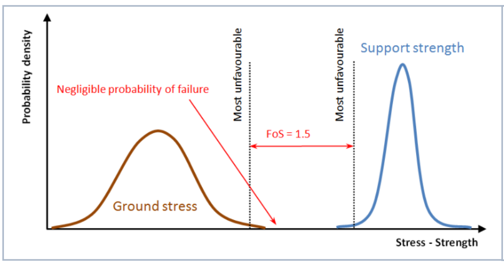


Fig. 2 *Design based on most unfavourable*

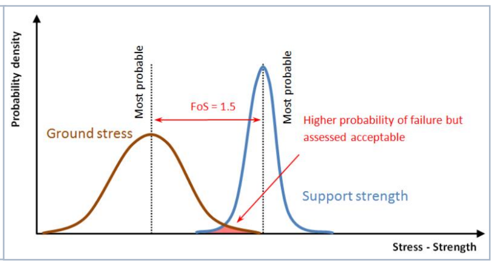


Fig. 3 *Design based on most probable*

4. Highway A3 Salerno-Reggio Calabria Serra Rotonda tunnel

(Client ANAS S.p.A. - General Contractor: Grandi Lavori Fincosit S.p.A.)
The Serra Rotonda tunnel is a twin bore tunnel with a length of nearly 3800 m, each bore has a section of 175 m2. It is the longest one of the Salerno-Calabria highway. At the north portal, for an extent of about 600 m, we find a stretch that presented special risks. It is characterized by a tectonized "clay shale" rock with poor geomechanical behavior and a great area with shallow landslides under-passed with a cover variables between 10m and 20m (figures 4 and photo 1).
A further risk factor was due to the possible presence of gas: so all construction phases were executed by explosion-proof systems.

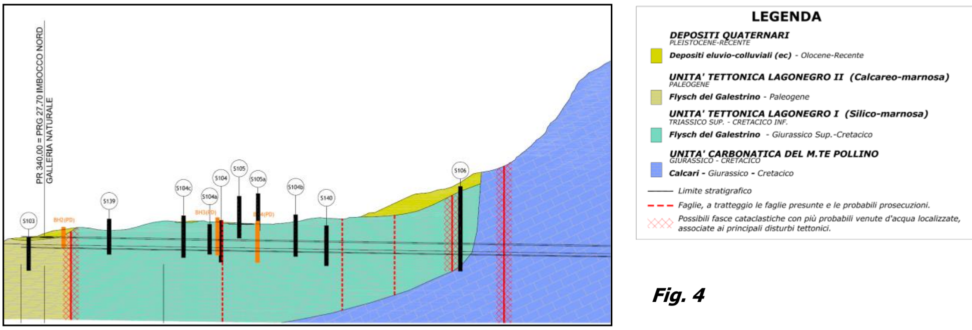


Fig. 4

The most challenging point was linked to the geomechanical behaviour of the rock to be excavated. Clay shale are 'soft' rocks in the sense that their unconfined compression strength is moderate or low. A typical feature of these rocks is that they weather (i.e. they have mechanical, physical and chemical changes) or degrade (i.e. they have a loss of mechanical competence) when they are exposed to stress or environmental conditions that differ from their undisturbed state. The behaviour of these materials in complex, it evolves from the behaviour of a rock, when undisturbed, to the behaviour of a soil clay when subjected to straining or weathered. This type of rock can be represented as a composite medium made of a clay matrix and a quasi brittle bonding microstructure; it is very difficult to characterize the mechanical behavior of these rocks and to define a constitutive model to be used in a design calculation model (Pinyol et al. 2007).
The design of stabilization measures was 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, heavy pre-confinement 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 around the tunnel.

Photo 1

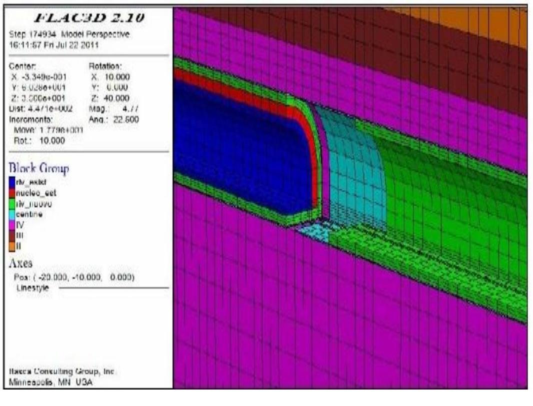


Fig. 5

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. The design of stabilization interventions was carried out with the parameters resulting in back analysis with reference to the "most probable" value.
A Flac 3D model was used to define the excavation system (figure 5).
The pre-confinement interventions consisted in the reinforcement of the core and of a shell around the future excavation by fiber-glass tubes injected with grout. The preliminary lining was composed of HEB 200÷220 steel ribs and "invert rib" at intervals of 0.80 - 1.00 m and a 30 cm layer of fiber reinforced shotcrete; tunnel invert and final concrete lining, was casted at short distance from the face relating to monitoring data. (figure 6).

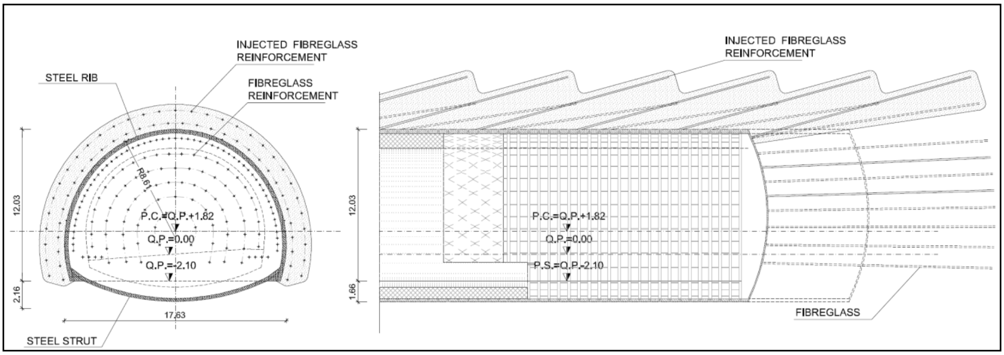


Fig. 6

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 (figures 7, 8).

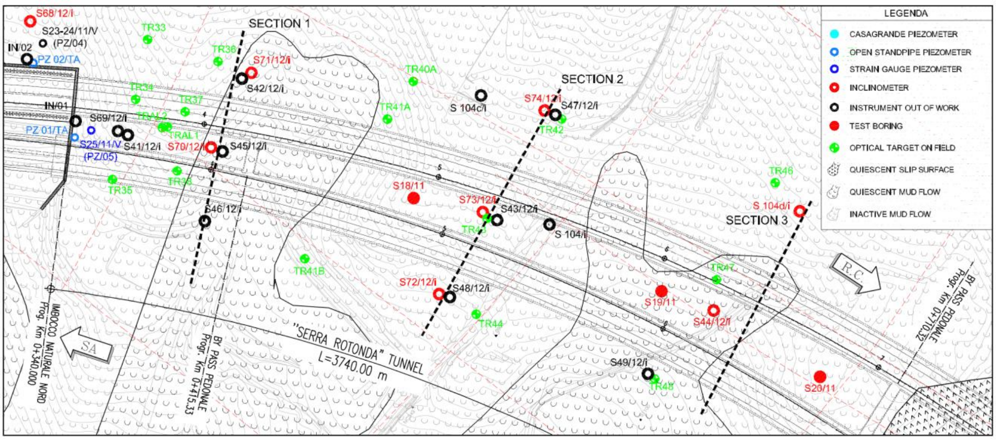


Fig. 7

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 7 and 8.

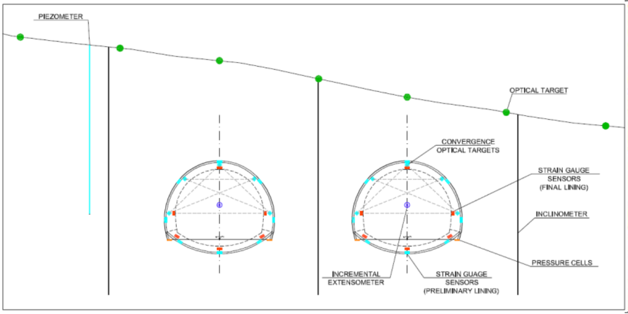


Fig. 8

Photo 2

During construction the original stabilization system was refined on the basis of an observational approach according to the scheme showed in figure 9.
Essentially, it was developed a detailed design defining the expected ground behaviour (in terms of displacement expected during construction) and the stabilization system (1); during construction it was measured displacement (and stress) by mean of the monitoring system (2); data collected during excavation was at last used to improve the original design model (3) and to refine consequentially the stabilization system (4). This procedure was repeated every 10 meters of excavation that is the span tunnel after which they have to do the interventions of ground improvement "in advance". In this way we were able to refine ground improvement intervention and preliminary lining on base of data collected in the stretch just dug.

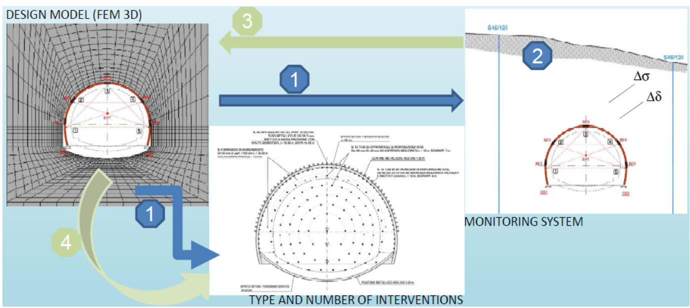


Fig. 9

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 geo-structural conditions. Through the analysis of the recorded data (convergences and extrusion), pre-confinement and support interventions have been gradually reduced proceeding with the excavations.

The main important success of this construction approach was that it enabled to preserve 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.

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

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 pre-confinement provided (maximum extrusions were equal to 8 cm).

The analysis of the monitoring data and in particular the results of the stress readings (strain gauge) (figure 10) has enabled also the optimization of steel ribs proceeding southbound.

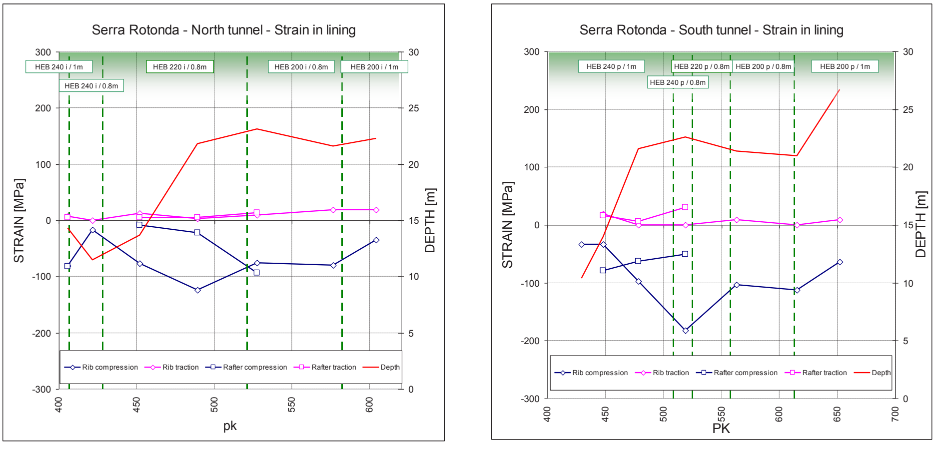


Fig. 10

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