Traditional systems for the construction of tunnels in difficult ground

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SUMMARY: it is known that the evolution in the projectual management of preconsolidations, consequent to a continuous and spiteful search through tests of laboratory, numerical simulations and experimental comparisons in situ has gone of equal footstep to the demand to industrialize the operations of constructions of galleries, as already happened in other sectors of engineering. On the base of an advance projectual management and a correct executive formulation, it has been reached the industrialization for the operations of construction of a gallery in every type of ground, with sure advantages both for the enterprises and for the administration.The succession of the operations and their correct execution are very important for a method based on control and regulation of deformed phenomena

1 THE WORK CYCLE: GENERAL BACKGROUND

It is well known that developments in the design of ground improvement ahead of the face, the result of continuous and meticulous research consisting of laboratory tests, numerical simulations and *in situ* experiments conducted over the last twenty years, have gone hand-in-hand with the demand to industrialise tunnel construction operations, as has already occurred in other engineering sectors.

Thanks to proper management of design both when operations are planned and during construction, the following has become established practice:

– the definition of proper relationships between the rigidity of the advance core (either natural or increased by ground improvement in advance) and the rigidity of the preliminary lining and that of the final lining;
– the full and unequivocal definition of each structural component of a tunnel, with clear specifications of the criteria by which they may vary to fit the real conditions encountered;
– the listing, in order, of all the construction processes specified in the design;
– the monitoring of these construction processes in accordance with ISO 9000 criteria;
– planning of operations in compliance with safety regulations.

In other words the industrialisation of tunnel construction operations (Fig. 1) based on advanced management of design and planning of operations has in fact been achieved in all types of ground with undoubted advantages both for contractors and clients.

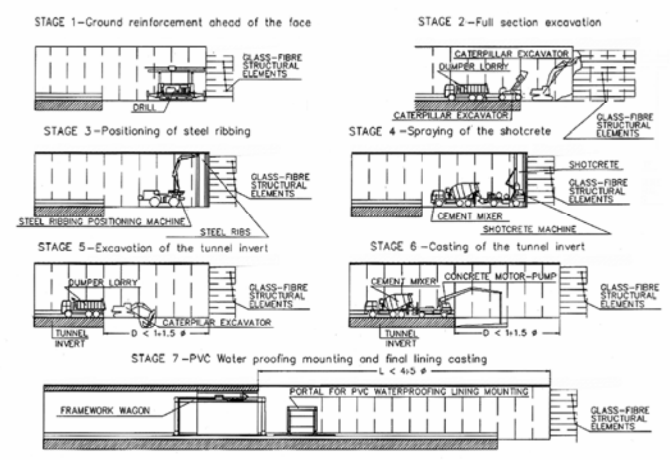

Figure 1. Typical tunnel construction stages

This was made possible to a large extent by the use of the full face excavation method with ground improvement ahead of the face (ADECO-RS) developed at the end of the nineteen eighties. It can still be considered an innovative method and in fact is not always “understood” by the design engineers and contractors who now employ it widely. Consequently it is advisable to thoroughly analyse the operational stages of which it is composed and to specify them in detail in tunnel designs.

The sequence of operations and their proper implementation assume increasingly greater importance in a method based on the monitoring and control of deformation phenomena.
We feel that it is important to underline that today it is the design engineers themselves who must forecast the time and costs required for the construction of a tunnel, which assumes they have in-depth knowledge of the means used, the time employed and of the relative costs.
Many considerations lead to the conclusion that the construction stages of a tunnel are of great importance both with regard to the statics and economic aspects (production).
As far as statics aspects are concerned, proper implementation of construction operations is a necessary condition for the stability of a cavity not just in the short term, but also in the long term. Poor implementation of ground improvement, excessive layers of overbreak, badly placed steel ribs and shotcrete in the linings and pauses and delays in tunnel advance all affect the deformation response of a cavity with possible instability in the short term and greater loads as a consequence on the final lining in the long term.
Here we will simply consider the finding that the response of a cavity improves markedly with increases in average daily production rates.

This can be seen in relation to two facts: one is that in ground with rheological behaviour, as is the case of almost all ground with a clayey component, faster advance rates result in less deformation of the rock mass and the other is that higher production rates are achieved when contractors become more “familiar” with conservation techniques and construction operations, performing them more rapidly and better, with the consequences already mentioned.

As concerns “fine tuning the design”, it is important to be able to assess construction times if the right balance is to be struck between different types of operation with the proper sequence of construction operations and if both faster production rates and more effective control of deformation are to be achieved.
We will see how this analysis should be performed and the results and modifications to which it can lead.

Obviously the economic and contractual implications and repercussions of this subject are considerable and they often constitute a crucial point in relations between the contractor and the design engineer. Reference is made to a sample tunnel actually constructed and an examination is made of the operational stages of the full face excavation method with ground improvement ahead of the face.

The main aspects of the method are described with a focus on techniques employed during construction to improve average advance rates, with huge increases in operational safety, savings on construction costs and improvements in the statics of the cavity in the long term.

1.1 *Face ground improvement stage*

The purpose of improving the ground in the face using fibre glass structural elements in saturated clayey soils is to make the core of ground at the face a *structural element*, which helps improve the statics of a tunnel with predictable and controllable behaviour and characteristics.
The aspects to be defined are the geometry, the methods for drilling and placing structures and the structural characteristics of the fibre glass elements.

1.1.1 *Geometry*

A ground reinforcement operation is fully defined by a set of geometric data that is directly related to the kinematics of the positioner rods.
The following is defined for each hole:

- position, expressed in polar co-ordinates:
  - distance from the centre of the ground
  - angle from the vertical
- angle of slope (radial)
- length

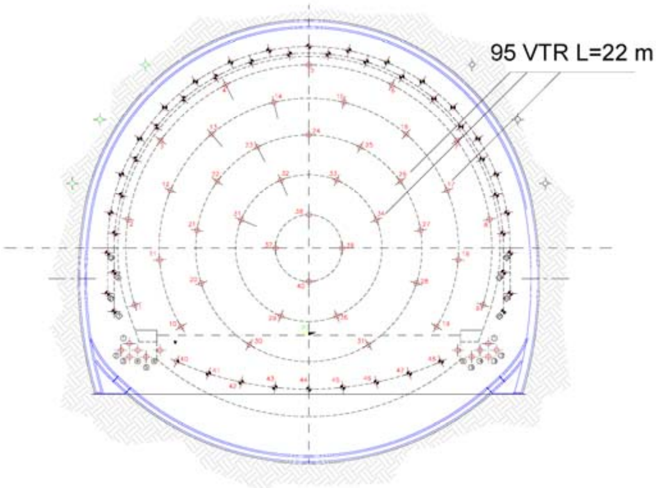

Figure 2 Geometrical layout of the fibre glass elements

The reinforcement used in the sample tunnel (Fig. 2) for the tunnel section type C_p consists of 95 fibre glass elements with a length of 22 m.

1.1.2 *Tracing*

Tracing the positions for inserting ground reinforcement is performed with the positioner (Fig. 3), which is then used for drilling, by operating the rod controls until the rod is positioned on the co-ordinates (offset and distance from centre) specified in the design.
The point which is found is marked and tracing of the next point then commences.

Figure 3Detail of the positioner during the installation of fibre glass elements

1.1.3 *Drilling procedures*

Reinforcement of the face forms a substantial part of the entire work cycle (approximately one third) and clearly any measures taken to improve efficiency will have a considerable effect on construction times.

Drilling procedures are therefore carefully planned since many hours can be gained or lost in this operation alone.

We have therefore given a full account of a brief study which we conducted on the most commonly used machinery and techniques in order to furnish a picture of the data to be considered and the problems involved in this stage of operations.

Figure 4 Detail of the machine used for drilling

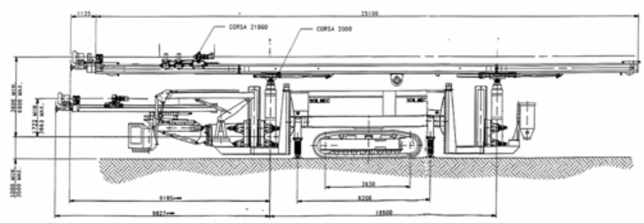

Figure 5Detail of the machine used for drilling

1.1.3.1 Characteristics of the drill bit

The characteristics of the drill bit consist of its dimensions (diameter of the bit and the hammer), the geometry of the bit, its strength (depending on the abrasive power of the rock), the shape, the dimensions and layout of the buttons and the possible use of a down-the-hole hammer.

In the case in question a Ø 152 mm bit was used with spherical buttons in widia, which could be reconditioned (even it the high cost of the operation made complete replacement less costly).

Holes are present on the face of the bits for the entrance and exit of compressed air (direct circulation).

The bits fit on a hammer (properly termed) of Ø 101 mm, fitted with a down-the-hole hammer action.

In softer ground, augers are also used with different flute depths used according to the type of ground.

1.1.3.2 Rod characteristics

The rods used were each 1.5 m. in length, with conical screw joints (for less strain on the threads).

1.1.3.3 Methods for increasing hole lengths

The positioners had guides with an effective travel of the rotary bits of between 14 m. and 21 m. depending on the versions.

Often the ground reinforcement lengths required were longer than this. The ground reinforcement performed in the sample tunnel was initially 22 m. in length, but this then grew to 24 m.

When conventional rotary bits are used with a drill string, drilling must be interrupted when 21 m. is reached in order to unscrew the rods from the bit, add an extension rod, screw it on, resume drilling and then remove the extension rod when each hole is finished in order to start drilling the next hole.

This set of operations lengthens the time taken for ground reinforcement considerably.

Some measures can be taken to overcome these limitations.

The simplest consists of using a slightly longer rod and exploiting the travel of the guide piston to advance a further 2 m.

Where this is not sufficient, the drive for the rotation of the rods can be transmitted using a connecting spindle, fitted with retractable clamps.

When these are used, once the end of travel of the bit has been reached, drilling is continued by opening the clamps and moving them along the rod which can then be lengthened because it is no longer limited by the end of travel position of the rotary bit.

Another measure that can be taken is to use telescopic rods: once the end of travel has been reached, the telescopic joint is loosened, the rotary bit is moved, the joint is tightened again and drilling is resumed.

Although this second method allows shorter rods to be used, delays are sometimes encountered due to problems with the telescopic joints, especially where there is an angle of slope to the ground reinforcement.

1.1.3.4 Drilling times

The theoretical optimum drilling time in soft clayey soils is approximately 40 m./min. The actual rate of production achieved must obviously include the time taken for the whole set of operations (tracing, positioning the arm, interruptions to lengthen the rods, delays and pauses, etc.). A total of 30 hours was employed in the sample tunnel to complete 2,090 m. of drilling (95 x 22 m.) using a positioner with two arms which gave an average speed of approximately 35 m. per hour per arm.

Operations then followed to place the fibre glass elements, caulk and cement them, which required approximately 40 hours (again for 95 fibre glass elements x 22 m). The total time required for ground reinforcement was therefore 70 hours, equivalent to approximately 32 m. per hour of fibre glass elements installed.

1.1.3.5 Dust abatement technology

Drilling for ground reinforcement at the face must be performed dry.

The use of slurries or foams disturbs the ground on the walls of the hole greatly, loosening it, particularly if it is under pressure and especially if it is in clayey ground with a schistose structure, and this prevents the fibre glass element from being effectively cemented into the ground.

Consequently appropriate technologies are employed with good results to reduce the dust produced by the tool. Often dust removers are employed which capture dust from the mouth of the hole. It is then stored at special points on the back of the positioner and subsequently collected and conveyed out of the tunnel.

1.1.4 *Cementation*

The placing, caulking and cementation of the fibre glass structural elements is performed immediately after drilling. If possible the operation is performed at the same time as drilling, cementing the holes as soon as they are drilled. Otherwise, as an alternative, 4-5 holes are cemented at a time. Proper implementation of cementation is important if tangential stresses are to be transmitted around the fibre glass elements. Particular attention must therefore be paid to see that the injection tube does not become clogged. The recent development of “expansive” grouts obtained mainly by the addition of aluminates, has led to a certain improvement in the cementation effect with respect to implementation times.

1.1.5 *Ground reinforcement times*

A total of 73 hours were employed to install 95 fibre glass elements at the face, equivalent to approximately 30 m. of fibre glass elements per hour.

It should also be considered that when an extrusion meter is installed, this requires three hours and time must also be added for readings to be taken which requires work at the face to be halted. This brings the time used to take extrusion measurements to 6-8 hours.

Once tunnel advance was fully underway, the time employed reduced to approximately 60 hours due to greater confidence with the machine and other measures taken.

The charts in figure 6 give the times taken for the various operations.

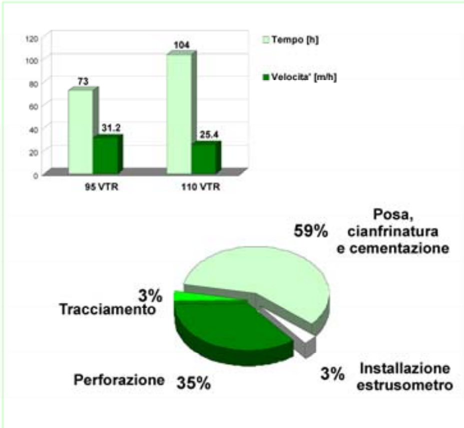

Figure 6 Ground reinforcement times

1.2 *Excavation and spoil removal stage*

Figure 7. Excavation at the face

The tool used for excavation changed according to the type of ground. In homogeneous clayey grounds a three shank ripper was used with an action sufficient to remove the material, which was detached in chunks of appropriate size. However, the frequent presence of rock inclusions even of some metres in size led to the use of a hammer excavator (Fig. 7) which was used without percussion to excavate the clay matrix and with percussion when boulders were encountered.

Excavation times can be improved in tunnels with a cross section of greater than 120 sq. m. by performing excavation and soil removal simultaneously.

1.3 *Preliminary lining*

Figure 8 Placing the preliminary lining

The preliminary lining, consisting of steel ribs and fibre reinforced shotcrete, plays a very important role in the full face excavation method and above all has a very different function to that which it plays in NATM type methods.
With these methods, the preliminary lining is used to exert *confinement pressure* around the cavity with the function of limiting convergence, especially where this would lead to instability of the cavity and the face in the absence of a lining. However, it has the characteristic of being a lining which *yields*.
The size of the preliminary lining is decided by observing the deformation response and establishing a limit for acceptable deformation.
Experience has now shown that acceptance of substantial deformation and therefore of relaxation of the rock mass is not practical under difficult geological conditions, where relaxation of the rock mass is followed by a considerable fall in the level of the geomechanical characteristics (deformability and strength) with consequent deformation and increasingly greater pressure on linings. Cases in which considerable instability and tunnel collapse occurred were not infrequent. This has resulted in the use of NATM being abandoned now in most parts of the world.
In Italy we learn from the technical literature that pressure exerted by swelling clayey soils during the construction of the Great Apennine Tunnel was strong enough to break the linings and cause convergence of more than 1 m., requiring repeated resumptions of excavation and the placement of new linings. The tunnels for the high speed Bologna–Florence railway driven through the same ground were constructed using full face excavation and reinforcement of the face in advance with only a few centimetres firstly of extrusion and then of convergence.
When full face excavation techniques are employed the preliminary lining functions in conjunction with the conservative operations performed ahead of the face designed to prevent and control deformation in the advance core, while the preliminary lining contains deformation that occurs in the tunnel after the passage of the face. Conservative operations and the placing of the preliminary lining therefore combine to form a continuous action to control deformation phenomena in the ground.
The entire sequence of the excavation operations is designed to produce the least possible disturbance to the rock mass. Clearly the lining must be sufficiently rigid to withstand deformation and it must be placed as close as possible to the face. The rigidity is achieved not just by the size of the components of the preliminary lining (ribs, shotcrete, etc.), but also and above all by the design of the geometry (polycentric ribs) and the use of measures (a strut in the tunnel invert, casting the invert at a distance of not more than one tunnel diameter from the face, etc.) designed to contain deformation as much as possible.

1.3.1 *Strut in the tunnel invert*

Figure 9 Detail of the strut in the tunnel invert

Particular mention is made of the strut in the tunnel invert (Fig. 9): it has a very important function in the control of deformation (convergence and extrusion) and it has produced excellent results. To underline this, the term “inverted pre-arch” has come into use, in analogy with the lining and preliminary lining combination.
In fact, although the rigidity is considerably less, the strut provides an adequate counter force against the steel ribs closing inwards (which given the high Ko value in the ground in question usually manifests with 1-5 cm. of closure at the base, and that is with strong lateral stresses and negative bending in the crown), partly prevents the steel ribs from digging into the invert and provides a “point of strength” to withstand extrusion and the tendency of the invert to rise.

1.3.2 *Excavation and preliminary lining times*

Excavation and spoil removal is not normally performed simultaneously but in sequence and it requires a total of 4 hours. Another 4 hours are required to place each rib in the preliminary lining.

The entire work cycle for excavation and the installation of a rib at average intervals of 1 m. therefore required 8 hours.
Figure 10 gives details of the time required for the individual operations to be performed to place the preliminary lining.

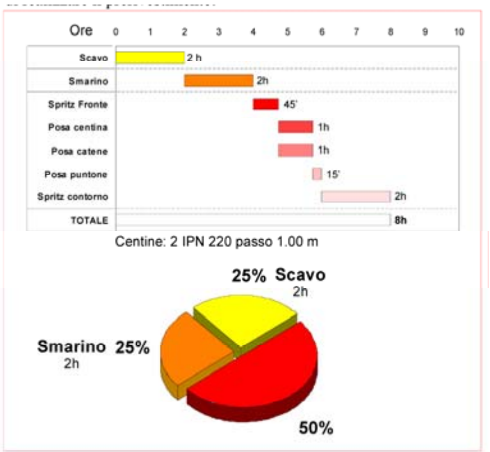

Figure 10 Excavation and preliminary lining times

1.4 *The tunnel invert stage*

Figure 11 Placing reinforcement for the tunnel invert

The tunnel invert (Fig. 11) is a structural component of fundamental importance for controlling deformation in tunnels. Its high rigidity, together with the practice of always casting it very close to the face, allows it to contain predicted deformation effectively.
It must also be considered that should unexpected events occur, it is able to withstand considerable stress with little deformation and it can therefore withstand a large proportion of the deformation which might be generated by such events. On the basis of those considerations, the tunnel invert can be cast every 6 m. up to the face. This allows unexpected stress-strain behaviour manifesting under shallow overburdens due to sudden changes in lithology and stratigraphy to be overcome without excessive difficulties. Where the geomechanical and stress-strain conditions were much better, it was possible to increase the length of the tunnel invert sections to 11 m. again casting them right up to the face.
It is worth noting that the length of the tunnel invert sections is directly proportional to the production rates that can be achieved. Casting the tunnel invert involves a series of operations for which the time take is independent of the volume cast (formwork, reinforcement, setting, etc.), which results in a great loss of time if short lengths are cast. In this case too, as in others, it is best to seek the optimum solution which provides adequate margins of safety with regard to stability, but which also allows acceptable production rates at the same time.

1.4.1 *Tunnel invert times*

In the case considered, the time taken for excavation, which depended on the length of the tunnel invert sections (6 m.), was 3 hours. The time taken for casting the tunnel invert also depends on the length of the sections and was 7.5 hours in the case considered.
Placing of the reinforcement, the formwork and shuttering required almost 5 hours even though the reinforcement placed was pre-assembled.
It was then necessary to wait 7 hours for the concrete to set before work could resume.
By improving the geomechanical conditions of the ground at the face and around the cavity it was possible to adopt a single tunnel invert section of between 11 m. and 12 m. instead of two 6 m. sections and this resulted in a considerable increase in productivity (Fig. 12).

Figure 12 Tunnel invert construction times

1.5 *The final lining stage*

As a consequence of all the considerations that have been made on the role of the preliminary lining and the tunnel invert, the final lining plays a less critical and less important role during construction than previously, precisely because it plays a fundamental in the service life of a tunnel as a whole. The final lining must nevertheless be placed within a certain distance from the face, above all to prevent rheological phenomena from developing as a function of time (the face effect having run itself out),

which can generate substantial deformation if the lining is nor sufficiently rigid.
The distance at which the final lining is placed on average must be large enough not to interfere with the work cycle, or in other words placing the lining should not require the interruption of operations at the face.

1.6 *Work cycle times*

The two charts that follow summarise the work cycle times for the Cp tunnel section (Fig. 13 e 14).
The first is for the period before observations and improvements had been made, while the second relates to three months later when the site workers had perfected the changes made to the various operational stages.
A marked increase in production rates can be seen due to effective improvements made to the design on the basis of specific times measured.

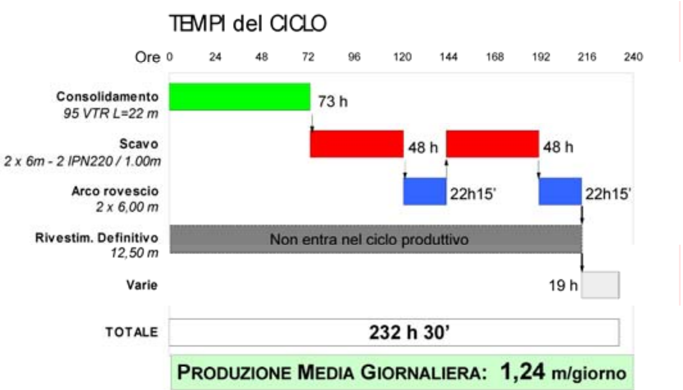

Figure 13 Work cycle times in the initial period

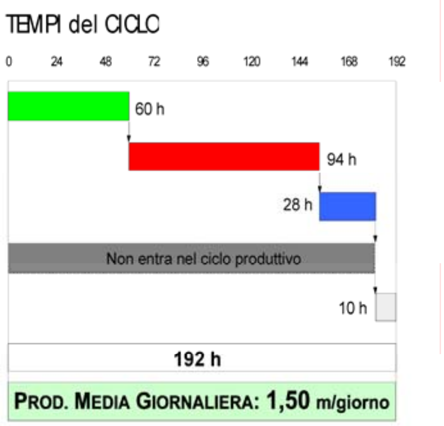

Figure 14 Work cycle times three months later

The production rates reported were maintained for almost five kilometres of tunnel which demonstrates that with advanced management of design and proper implementation of operations, tunnel construction can be industrialised in all types of ground with undoubted advantages both for contractors and for clients.

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