The design and construction of tunnels using the approach based on the analysis of controlled deformation in rocks and soils

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1.0 INTRODUCTION

Anyone who sets out to construct underground works, finds himself having to tackle and solve a particularly complex civil engineering problem because, compared to surface constructions, it is extremely difficult to determine the basic design data of underground works in advance.

First of all it is not a question, as with surface construction, of gradually assembling materials (steel, reinforced concrete, etc.) with well known strength and deformation properties to build a structure which finds its future equilibrium in the desired final configuration, but of acting on a pre-existing equilibrium and proceeding in some way to a "planned disturbance" of it in conditions that are known only approximately.

Another characteristic of underground works, well known to design and construction engineers, but not always given sufficient weight, is that very often the stage at which the structure is subject to most stress is not the final stage when the tunnel is finished and subject to external loads predicted at the design stage, but the intermediate construction stage, very much more delicate because at this stage the effects of the disturbance caused by excavation have not yet been completely confined by the final lining. At this stage the pre-existing stresses in the rock mass deviated by the opening of the cavity are channelled around it ("arch effect") creating zones of increased stress on the walls of the excavation.

Just how delicate this intermediate stage is becomes clear if one considers that it is precisely the correct channelling of stresses around the cavity that determines the integrity and life of a tunnel. Channelling can be produced, depending on size of the stresses in play and the strength and deformation properties of the ground, as follows *(fig. 1)*:

1) close to the profile of the excavation
2) far from the profile of the excavation
3) not at all

The first case occurs when the ground around the cavity withstands the deviated stress flow around the cavity well, responding elastically in terms of strength and deformation.

The second case occurs when the ground around the cavity is unable to withstand the deviated stress flow and responds anelastically, plasticising and deforming in proportion to the volume of ground involved in the plasticisation phenomenon; the latter also causes an increase in the volume of the ground affected, and propagates radially with the result that the channelling of the stresses is deviated away from the tunnel into the rock mass until the triaxial stress state is compatible with the strength properties of the ground. In this situation, the "arch effect" is formed far from walls of the excavation and the ground around it that has been disturbed at this point is only able to contribute to the final statics with its own residual strength and will give rise to deformation of considerable entity.

The third case occurs when the ground around the cavity is completely unable to withstand the deviated stress flow and responds in the failure range producing the collapse of the cavity.

It follows from this analysis of these three situations that:

- an arch effect only occurs naturally in the first case;
- an arch effect is only produced naturally in the second case if the ground is "helped" with appropriate intervention to stabilise it;
- in the third case, since an arch effect cannot be produced naturally it must be produced artificially by operating on it appropriately before it is excavated.

The first and most important task of a tunnel design engineer is to determine if and how an arch effect can be triggered when a tunnel is excavated and then to ensure that it is formed by calibrating excavation and stabilisation operations appropriately as a function of the particular stress strain conditions.

For this purpose a knowledge of the following is indispensable *(fig. 2)*:

– of the medium in which operations take place;
– of the action taken to excavate;
– of the reaction expected following excavation.

The medium (the ground), which is the actual construction material employed for building tunnels, is an extremely anomalous material when compared to traditional civil engineering construction materials: it is discontinuous, unhomogeneous and anisotropic. On the surface, it presents very varied characteristics which, however, depend exclusively on its own intrinsic nature (natural consistency), which conditions the morphology of the earth's crust. At depth, however, its characteristics will change as a function of the stress states it is subject to (acquired consistency), conditioning its response to excavation.

The action occurs when the face advances through the medium. It is therefore a distinctly dynamic phenomenon: the advance of a tunnel may be visualised as a disk (the face) that proceeds through the rock mass with a velocity V, leaving an empty space behind it. It produces a disturbance in the medium, both in a longitudinal and transverse direction, which changes the original stress states.

Within the disturbed zone, the original field of stresses, which can be represented as a network of flow lines, is deviated by the presence of the excava-

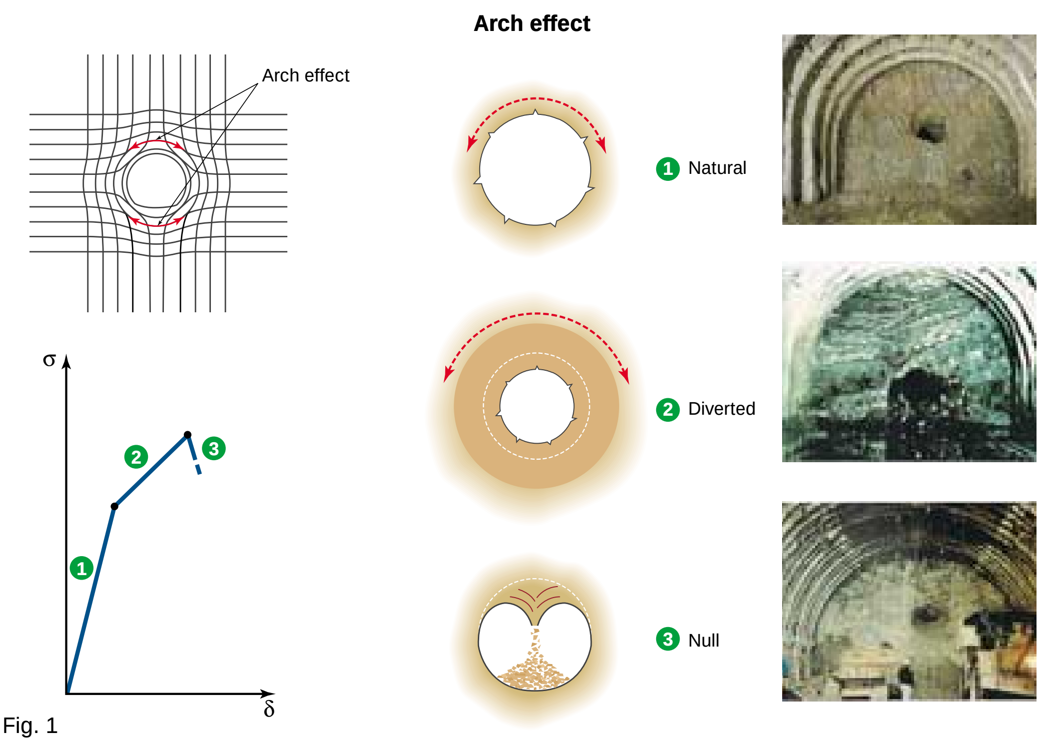


Fig. 1

tion *(fig. 1)* and concentrates close to it producing increased stress. The size of this increased stress determines the amplitude of the disturbed zone for each medium (within which the ground suffers a loss of geomechanical properties with a consequent increase in volume) and therefore the behaviour of the cavity in relation to the strength of the rock mass σ_{gd}.

The size of the disturbed zone close to the face is defined by the radius of influence of the face R_f, which identifies the area on which the design engineer must focus his attention and within which the passage from a triaxial to a plane stress state occurs (the face or transition zone); proper study of a tunnel requires three dimensional methods of calculation and not just two dimensional methods.

The reaction is the deformation response of the medium to the action of excavation. It is generated ahead of the face, within the area that is disturbed following the general increase in stress in the medium around the cavity and depends on the medium (consistency) and on the way in which face advance is effected (action). It may determine the intrusion of material inside the theoretical profile of the excavation. Intrusion is synonymous with instability of the tunnel walls.

Three basic situations may arise:

If on passing from a triaxial to a plane stress state during tunnel advance, the progressive decrease in stress at the face (σ_3 = 0) produces stress in the elastic range ahead of the face, then the wall that is freed by excavation (the face) remains stable with limited and absolutely negligible deformation. In this case the channelling of stresses around the cavity ("arch effect") is produced naturally close to the profile of the excavation.

If on the other hand, the progressive decrease in the stress state at the face(σ_3 = 0) produces stress in the elastic-plastic range in the ground ahead of the face, then the reaction is of importance and the wall that is freed by excavation, the face, will deform in an elastic-plastic manner towards the interior of the cavity and give rise to a condition of short term stability. This means that in the absence of intervention, plasticisation is triggered, which by propagating radially and longitudinally from around the excavation produces a shift of the "arch effect" away from the tunnel into the rock mass. This movement away from the tunnel can only be controlled by intervening with adequate means of stabilisation.

If, finally, the progressive decrease in the stress state at the face (σ_3 = 0) produces stress in the failure range in the ground ahead of the face, then the deformation response is unacceptable and a condition of instability exists in the ground ahead of the face, which makes the formation of an "arch effect" impossible: this occurs in non cohesive or loose ground and an "arch effect" must be produced in it artificially since it cannot occur naturally.

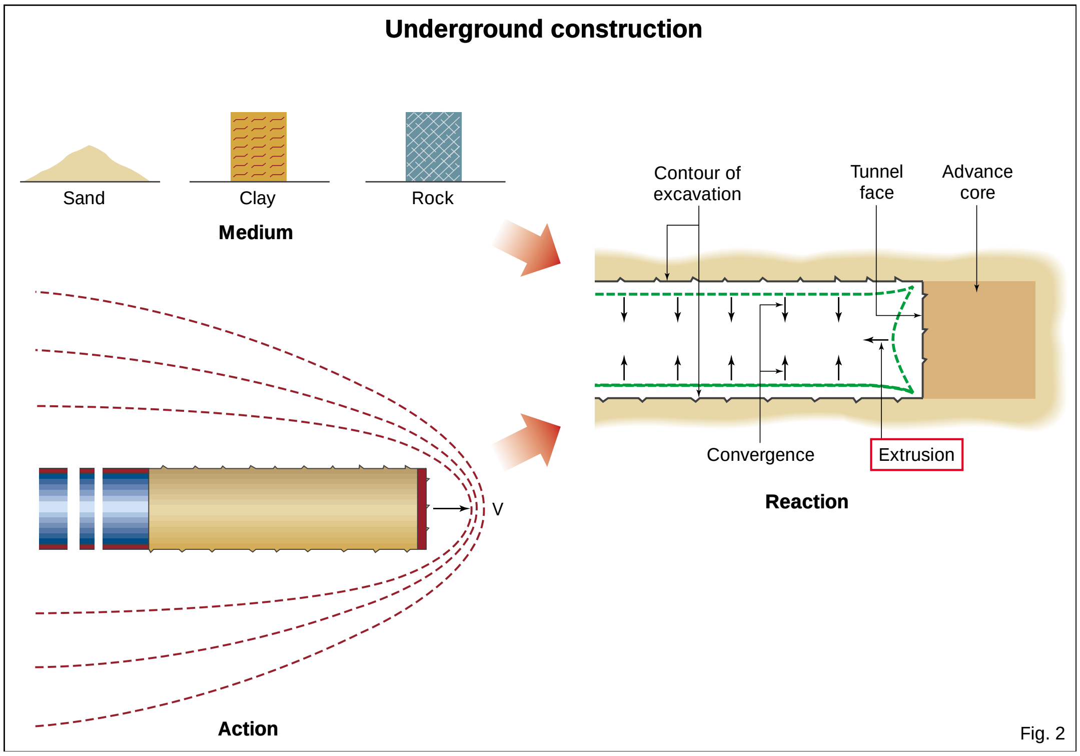


Fig. 2

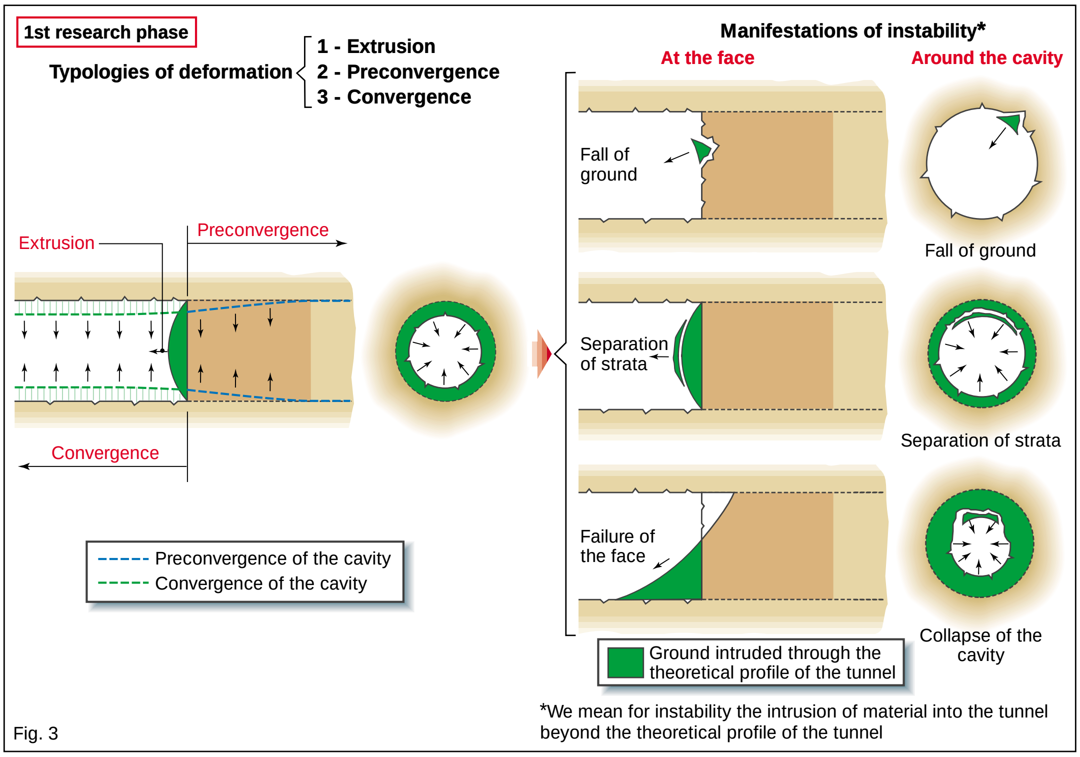


Fig. 3

It follows therefore that the formation of an arch effect and its position with respect to the cavity (on which we know that the long and short term stability of a tunnel depends) are signalled by the quality and the size of the "deformation response" of the medium to the action of excavation.

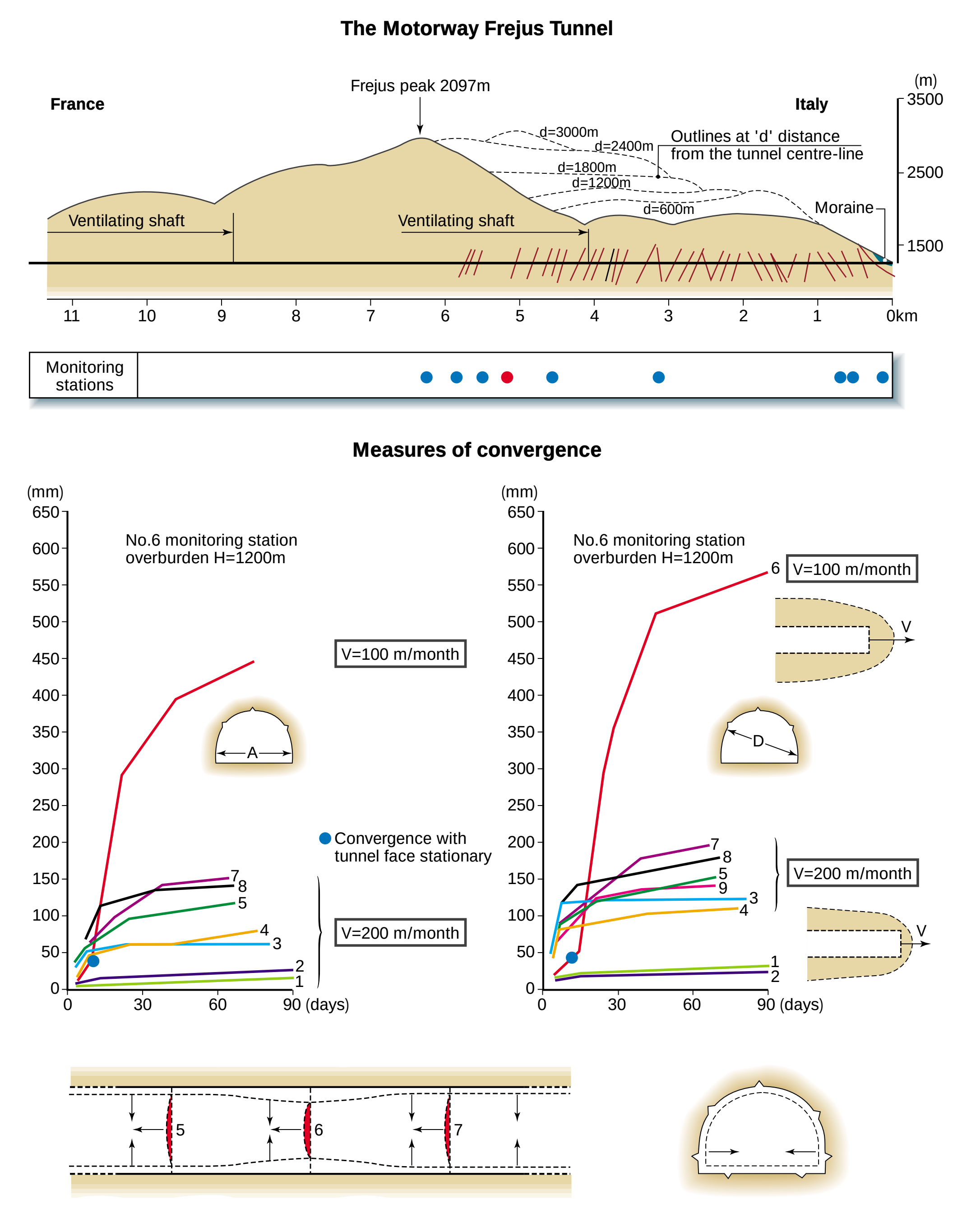
With these considerations as our starting point, over twenty five years ago, we started to study the relationships between changes in the stress state in the medium induced by tunnel advance and the consequent deformation response of the tunnel.

2.0 RESEARCH ON THE DEFORMATION RESPONSE OF THE MEDIUM

The analysis of the deformation response of the rock mass (effect) has developed during the course of research, both experimental and theoretical, that began twenty five years ago and is still continuing.

The "**first research stage**" was dedicated above all to systematic study of the stress-strain behaviour of a wide range of tunnels during construction. Particular attention was paid to the behaviour of the face and not just that of the cavity as is normally done. The complexity of what we were studying, the deformation response (effect) became clear very soon and we needed to identify new terms of reference in order to define it fully *(fig. 4)*:

- **the advance core**: the volume of ground that lies ahead of the face, virtually cylindrical in shape with the height and diameter of the cylinder being approximately the same size as the diameter of the tunnel;
- **extrusion**: the primary component of the deformation response of the medium to the action of excavation that develops largely inside the advance core; extrusion depends on the strength and deformation properties of the core and on the original stress field to which it was subject; it manifests on the



Monitoring station	1	2	3	4	5	6	7	8	9
Chainage (m)	845	861	2772	3954	4507	5172	5533	5915	6066
Overburden (m)	490	500	580	590	740	1200	1400	1530	1640

Fig. 4

surface of the face in the direction of the longitudinal axis of the tunnel and its geometry is either more or less axial-symmetric (bellying of the face) or of gravitational churning (rotation of the face);

- **preconvergence** of the cavity: convergence of the theoretical profile of the tunnel ahead of the face, strictly dependent on the relationship between the strength and deformations properties of the advance core and its original stress state.

Subsequently, during the "**second research stage**", detailed analysis - above all in terms of timing - was performed on instability phenomena observed during the construction of at least 400 km of tunnel in an extremely wide range of ground types and stress strain conditions. The aim was to seek a connection between the behaviour of the face-advance core system (extrusion and preconvergence) and that of the cavity (convergence).

Once we had established that the deformation response as a whole (extrusion, preconvergence and convergence) is systematically conditioned by the rigidity of the core of ground at the face (which is therefore the real cause of it), at a third stage, "**the third research stage**" we worked to discover to what extent the deformation response of the cavity (convergence) could be controlled by acting on the rigidity of that core. To do this the stress strain behaviour of the advance core, systematically compared to that of the cavity was analysed both in the absence and the presence of intervention to protect and to reinforce the core.

2.1 THE FIRST RESEARCH STAGE

The first research stage (systematic observation of the deformation behaviour of the face-advance core system) was conducted by using instruments and visual observation to monitor the stability and deformation behaviour of the advance core and walls of tunnels, paying particular attention to the following phenomena *(fig. 3)*:

a) extrusion of the face;
b) preconvergence of the cavity;
c) convergence of the cavity (decrease in the size of the theoretical cross section of the excavation after the passage of the face).

Systematic visual observation performed inside cavities enabled the following manifestations of instability located on the face or around it (instability is intended as occurring whenever material intrudes into an excavation beyond the theoretical profile) to be associated with the types of deformation mentioned above:

a) fall of ground, spalling and failure of the face located at the face-advance core system and following extrusion of the face and preconvergence;

b) fall of ground, spalling and collapse of the cavity located on the roof and walls of the cavity and following convergence of the cavity itself.

2.2 THE SECOND RESEARCH STAGE

Once the types of deformation and manifestations of instability that occur on the core at the face and on the roof and walls of a tunnel had been identified, we asked ourselves whether observation of the former might in some way give us an indication of what the type and size of the latter would be. The second stage of research thus commenced [to seek possible connections between the deformation of the face-advance core system (extrusion and preconvergence) and that of the cavity (convergence)]. It was performed by studying, observing and monitoring deformation at the face and in the cavity, with particular attention paid to the magnitude and chronological sequence in relation to systems, stages and paces of construction employed in the various tunnels.

It is important here to briefly illustrate the observations made, with a few significant examples before examining the results of this experimental stage.

2.2.1 THE EXAMPLE OF THE FREJUS MOTORWAY TUNNEL (1975)

Ninety five percent of the route of the Frejus motorway tunnel (13 km in length with overburdens of up to 1,700 m.) passes through a metamorphous formation of schistose crystalline limestone, that is lithologically homogeneous for the length of the route.

The design of the tunnel was made on the basis of a geological and geomechanical survey conducted from the adjacent railway tunnel (completed in 1860) and from the service tunnels. The strength and deformation tests performed on samples of the schistose crystalline limestone gave the following average geotechnical values:

– angle of friction: 35°;
– cohesion: 30 Kg/cm^2 (= 3 MPa);
– elastic modulus: 100,000 Kg/cm^2 (= 10,000 MPa).

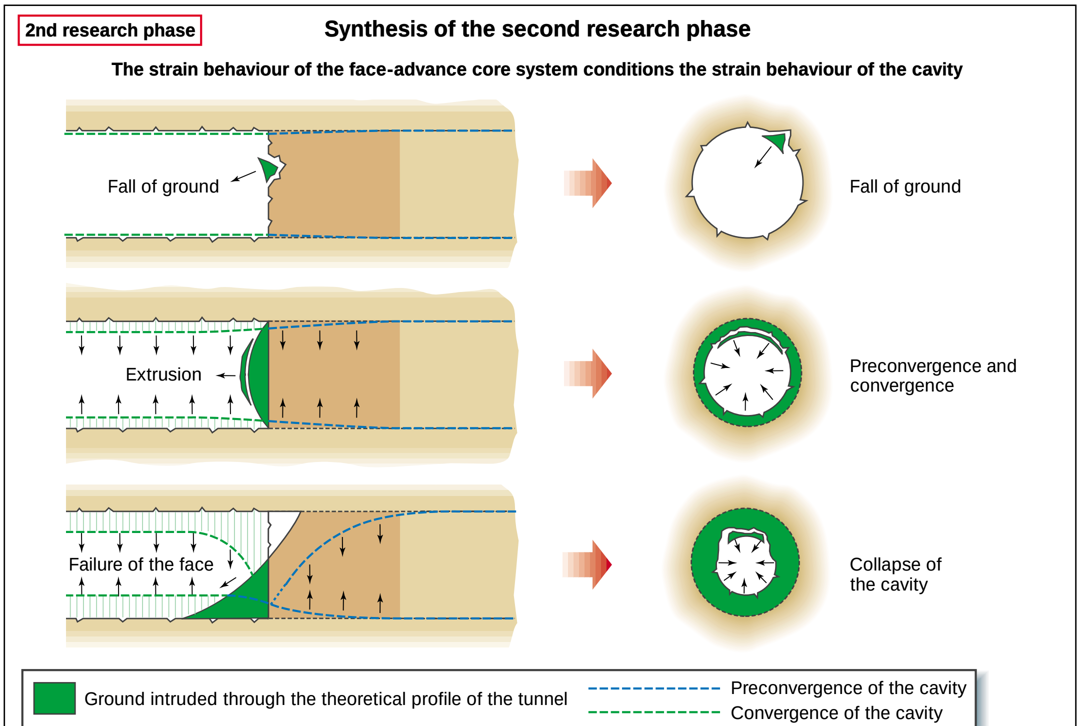


Fig. 5

No forecasts of deformation behaviour were made in the original design for the tunnel (1975), since this was not part of standard practice at that time.

Account was taken of Sommeiller's experience during the construction of the adjacent railway tunnel about a century before. Full face advance was decided with immediate stabilisation of the ring of rock around the cavity with end-anchored active roof bolts to a depth of 4.5 m. supplemented with shotcrete. The final lining was in concrete with an average thickness of 70 cm. cast afterwards to complete the tunnel.

Study of deformation behaviour constituted the most significant part of the campaign of observations and measurements performed during construction. The purpose was to monitor the response of the rock mass to the stabilisation measures under the exceptional circumstance of driving a tunnel for the first time though a homogeneous rock mass (crystalline limestone) with variable overburdens subject to a stress field which increased and varied as a function of the overburden (0 - 1,700 m.).

Up to an overburden of 500 m., the rock mass remained stressed within the elastic range with the tunnel manifesting stable face behaviour, negligible deformation and limited instability at the face and in the cavity, consisting exclusively of fall of ground.

As the overburden and consequently the stress state increased, the rock mass entered the elastic-plastic range and tunnel behaviour became face stable in the short term, with convergence of the cavity measurable in decimetres (diametrical convergence 10 - 20 cm.). The band of reinforced rock contributed effectively towards the statics of the tunnel, limiting the amount of convergence and preventing further manifestations of instability.

The good quality of the rock also helped to maintain advance rates at 200 m. per month until work halted temporarily at metre 5,173 for the Summer holiday break, in a zone of homogeneous rock with an overburden of approximately 1,200 m.

Convergence measurement station No. 6 installed immediately 1 meter back from the face (metre 5,172) showed maximum deformation of approximately 10 cm, after 10 days of zero tunnel advance *(fig. 4)*. It was undoubtedly creep deformation (with a constant load) only, since the face had remained at a complete halt during that time. When advance recommenced, diametric convergence for that cross section increased very brusquely to values never before encountered reaching 60 cm. after 3 months, while further ahead it returned to normal values (diametrical convergence of 20 cm.) after 20 or 30 metres as advanced resumed.

It must be pointed out that before work halted the tunnel had been reinforced up to one metre from the face with more than 30 radial roof bolts per linear metre of tunnel, but no ground improvement had been performed on the core at the face.


Photo 1: S. Stefano Tunnel (Italy, Genoa To Ventimiglia Rail Line, ground: tectonised and laminated marly-limestone flysch, overburden: 150 m, tunnel diameter: 12 m.). Collapse of the cavity.

Once advance resumed, stabilisation procedures around the cavity continued with the same intensity and the same work rhythms as before.

The conclusion drawn was that while tunnel advance was halted, the core of ground at the face, not being assisted by any reinforcement intervention, had had time to extrude into the elastic-plastic range and this had triggered a phenomenon of relaxation of stress, through creep, of the rock mass around it (preconvergence). This had in turn caused the considerable increase in convergence of the cavity with respect to normal values.

2.2.2 THE EXAMPLE OF THE "SANTO STEFANO" TUNNEL (1984)

The "Santo Stefano" tunnel is on the new twin track Genoa to Ventimiglia railway line located on the section between S. Lorenzo al Mare and Ospedaletti.

It runs through a Helminthoid flysch formation characteristic of western Liguria. It consists of clayey and clayey-arenaceous schists with thin banks of folded and intensely fractured sandstones and marly limestones. The clay schist component is heavily laminated. An extremely tectonised transition zone marks the passage between the H1 zone and the more marly-limestone-like H2 zone .

Strength tests carried out in the laboratory on samples gave angle of friction values varying between 20° and 24° and cohesion from 15 Kg/cm^2 (= 1.5 MPa) to 0.

In this case too, with work beginning in 1982, no forecasts of tunnel deformation behaviour were made.

The original design involved full face advance with steel ribs and shotcrete for the primary lining and a thick ring of concrete (up to 110 cm.) for the final lining.

During excavation it was found that as long as the ground remained elastic, deformation at the face and in the tunnel was negligible and there were practically no manifestations of localised instability (stable face behaviour). When the tunnel entered a zone affected by residual stress states of tectonic origin, where the stress state was in the elastic-plastic field, deformation phenomena began to cause some difficulty, also due to the appearance of sizeable asymmetrical thrusts caused by rigid masses dispersed in the plastic matrix. As this occurred, layers of material were seen to break off at the face, a sure sign of the presence of extrusion-type movement typical of a face stable in the short term situation; convergence reached decimetric values.

At a certain moment the stress state of the rock mass had obviously developed into the failure range and the entire face failed (unstable face situation), followed within a few hours by the collapse of the cavity with diametric convergence of over 2 m. also for a considerable stretch of more than 30 m. of tunnel behind the face, even in the part where ribs and shotcrete had been placed (photo 1).

It should be noted at this point, that the type of ground encountered in the three stress-strain situations mentioned was essentially the same and that the phenomenon of tunnel collapse with convergence measurable in metres, even in a part of the tunnel where the primary lining had already been placed, only occurred when the core at the face was no longer rigid and able to contribute to the statics of the tunnel.

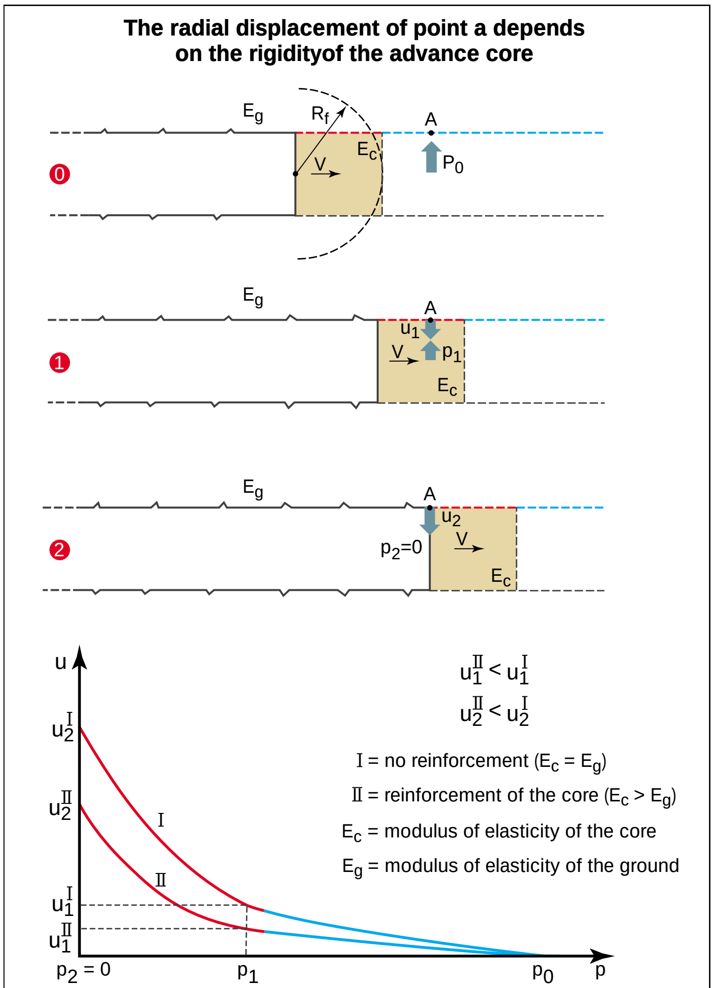


Fig. 6

2.2.3 THE EXAMPLE OF THE "TASSO" TUNNEL (1988)

The "Tasso" tunnel is one of a series of tunnels excavated towards the end of the 1980's for the new "High Speed" Rome to Florence railway line. The area in which the tunnel is located lies in the lake basin of the Valdarno Superiore and consists of silty sands and sandy silts with interbedding of levels of silty clays containing sandy lentils and levels saturated with water.

The original design involved half face advance, stabilising the walls with ribs and shotcrete. The ribs were anchored at the feet with sub-horizontal tie bars and given a foundation of micropiles or columns of ground improved by jet-grouting.

Initially excavation proceeded under face stable in the short term conditions with no appreciable deformation phenomena either at the face or in the tunnel.

As the overburden increased, and therefore also the stress state of the medium, and given also the poor geomechanical characteristics of the ground, conditions of face stable in the short term rapidly changed to those of an <u>unstable face</u>. Following the failure of the face despite half-face advance, approximately 30-40 m. of tunnel already excavated and protected with ribs and shotcrete also collapsed during the course of one single night with convergence in the order of 3-4 m. *(fig. 26)*

2.2.4 RESULTS OF THE SECOND RESEARCH STAGE

Study and analysis of the cases illustrated and other similar cases, which would take too long to describe here, produced many ideas of great interest. The following points appeared clearly from the Frejus study:

- when advancing through ground in elastic-plastic conditions it is very important to maintained constant and sustained excavation cycles in order to avoid giving the core time to deform: it is thus possible to prevent the triggering of extrusion and preconvergence phenomena which constitute the starting point of subsequent convergence of the cavity.

On the other hand it emerged from the other experiences cited and other cases that:

- the failure of the core and the collapse of the cavity never occur without one being followed by the other and in particular without the latter being preceded by the former.

The following was clear from the second research stage *(fig. 5)*:

1) there is a close connection between extrusion of the core at the face and the phenomena of preconvergence and convergence of the cavity;

2) there are close connections between the failure of the advance core and the collapse of the cavity even if the latter has been stabilised;

3) deformation phenomena in the cavity are always chronologically consequent to and dependent on those that involve the core of ground at the face.

It also became clear that it was necessary for the formation of an arch effect, which as we know conditions the stability of a tunnel, to have already been triggered ahead of the face to be able to continue to function in a determined cross section after the face has already passed ahead of it.

2.3 THE THIRD RESEARCH STAGE

The results of the second research stage reinforced the impression, which we already had that the deformation of the advance core of a tunnel was the true cause of the whole deformation process in all its components (extrusion, preconvergence and convergence) and that as a consequence, the rigidity of the core played a determining role in the stability of a tunnel both in the long and short term.

If one takes a point **A** located on the profile of the crown of a tunnel still to be excavated, it is in fact quite clear that its radial movement **u** (preconvergence) as the face approaches it depends on the strength and deformation properties of the ground inside the profile of the future tunnel.

If the course of its radial movement is plotted on a graph **p-u** (where **p** is the confinement pressure exerted radially on **A**) it can be seen *(fig. 6)* that as long as the face is still distant (distance from **A** greater than the radius of influence of the face R_f) the stress condition of **A** remains unchanged (radial confinement pressure p_0 = original pressure). As the face approaches, however, the radial confinement pressure **p** also diminishes as a result: **A** will then start to move radially towards the inside of the future cavity. How far it moves depends, as we have said and as is quite obvious, on the stresses present and on the deformation properties of the core which determines its equilibrium and not only on the geomechanical characteristics of the surrounding ground.

After the passage of the face, on the other hand, the radial movement of **A** will continue, in the elastic or elastic-plastic range, as a function of the pre-existing stress states, of the characteristics of the ground behind the lining and of the radial confinement pressure exerted by stabilisation works (preliminary lining, final lining) on which the equilibrium of point **A** depends.

The qualitative graph in *fig. 6* shows, other conditions remaining unchanged, the course of the deformation to which **A** is subject in the case of a deformable advance core (curve I) and in the case of a rigid advance core (curve II): obviously, up to the passage of the face, the radial deformation of point **A** as the radial confinement pressure **p** reduces is less for a rigid core than for a deformable core. It would also appear probable that even after the passage of the face, and there-

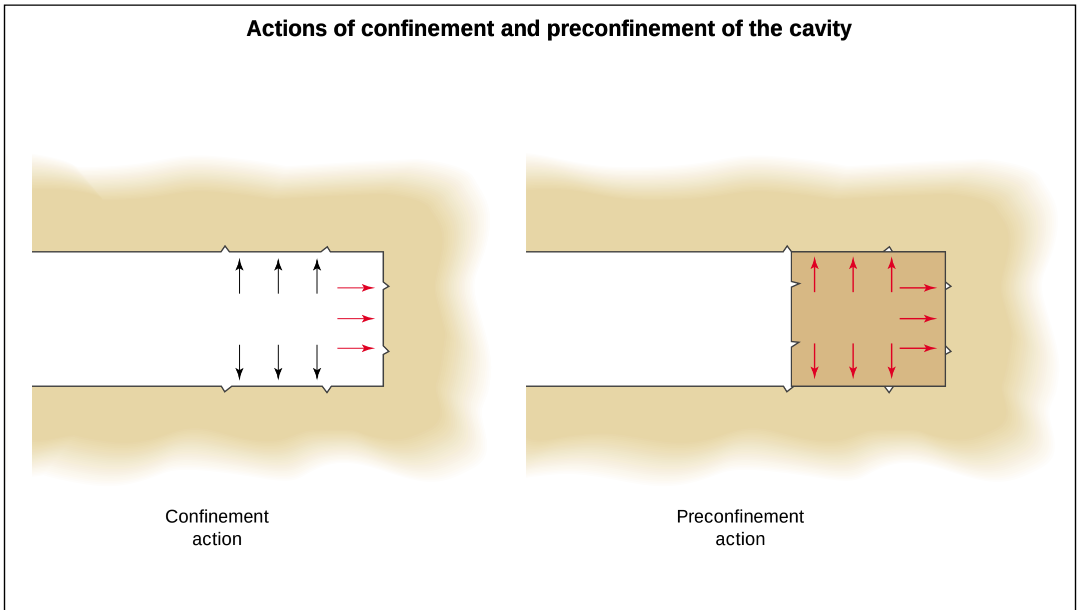


Fig. 7

fore without the confinement exerted by the advance core, the curves I and II would remain clearly separate and that the movement of point **A** would be determined by its previous stress-strain history. It follows that the deformation properties of the advance core constitute a factor capable of conditioning the deformation response of the ground to excavation and it must be considered the true cause of it.

Now, if the deformation properties of the advance core constitute the true cause of the deformation response of the ground to excavation, it seems logical to hypothesise making use of the core as a new tool for controlling that response, by acting on its rigidity with appropriate intervention.

We therefore worked on the possibility of regulating the rigidity of the advance core in order to ascertain to what extent this would make it possible to control the deformation response of the cavity.

To do this, new technologies and new types of intervention had to be researched and developed that would act on the core protecting it from too much stress (protective techniques) and/or conserving or improving its strength and deformation properties (reinforcement techniques). These particular types of intervention are termed "conservation techniques" or alternatively "cavity preconfinement techniques" to distinguish them from ordinary confinement which only acts on the ground surrounding the cavity after the passage of the face *(fig. 7)* [1].

The new ideas were then tried out on the construction of a few tunnels under very difficult stress-strain conditions. One such experiment on a particularly difficult tunnel and therefore also very significant is described below.

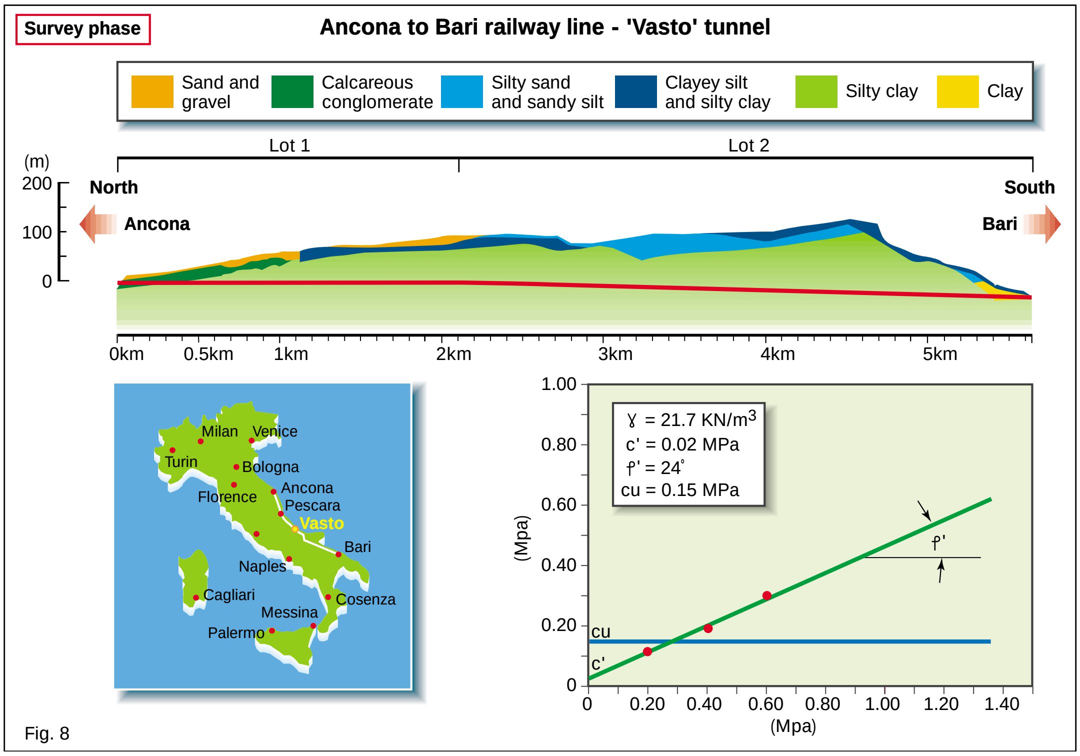


Fig. 8

2.3.1 THE "VASTO" TUNNEL

The route of the tunnel, part of the new Ancona to Bari railway line, runs for approximately 6,200 metres under the hills on which the village of Vasto lies.

From a geological viewpoint *(fig. 8)*, the bottom and middle part of this hilly relief consists of a complex of grounds mainly of a silty, clayey constitution, grey in colour, stratified with thin sandy intercalations, while the top part consists of a bank of conglomerates, cemented to a varying extent, over which a layer of sandy-silty ground extends, yellowish brown in colour.

The tunnel runs entirely through the base clay formation with the exception of the initial sections near the portals. At the depth of the tunnel the ground is saturated with water and extremely sensitive to dislocation.

2.3.1.1 A BRIEF HISTORY OF THE EXCAVATION

The works began in 1984 at the North portal and continued slowly between repeated and serious incidents until April 1990.

The original design involved half face excavation, immediately protected with a temporary lining of shotcrete, steel ribs and welded steel mesh reinforcement. The final lining in reinforced concrete, one metre thick was cast straight behind the face and still in the presence of the core. The side walls of the tunnel were cast subsequently for underpinning and the casting of the tunnel invert completed the work.

After the first serious incident of tunnel deformation, an attempt was made to resume tunnel advance employing several methods , which all, however proved to be completely inadequate and finally a disastrous cave-in occurred at chainage Km. 38+075 under an overburden of 38 m., which involved the face (photo 2) and then a section of approximately 40 m. behind it. It produced deformation of enormous size (greater than one meter) in the final lining and it was impossible to continue working.

At this point I was called in to find a solution that would allow the halted works to resume and to complete the tunnel. I tackled the far from simple problem by employing a new advance method for the rest of the tunnel which was based on principles of controlling deformation by stiffening the core at the face, and then by preconfinement techniques.

Photo 2: Vasto Tunnel (Italy, Ancona to Bari Rail Line, ground: silty clay, overburden: 135 m, tunnel diameter: 12.20 m.). The extrusion of the ground through the face during heading and bench excavation (according to NATM principles).

2.3.1.2 THE SURVEY PHASE FOR THE "VASTO" TUNNEL

Before commencing a new design, it was felt wise to acquire a more detailed geotechnical knowledge of the material to be excavated.

The ground, belonging to the bottom clay formation, was classified as clayey silts or silty clays from medium to highly plastic and impermeable, markedly susceptible to swelling when soaked in water.

Though direct shear tests in a triaxial cell provided rather a wide range of values for cohesion and angle of friction, they did give very low average values for strength.

Some "triaxial cell extrusion tests" were then used to model tunnel advance in the laboratory under the actual stress conditions of the tunnel in situ. These, integrated with simple finite element mathematical models made it possible to calibrate the geomechanical parameters (c, Ø, E) for use in the subsequent diagnosis and therapy phases. Direct simulation of the triaxial cell extrusion tests available (integrated with both consolidated and non consolidated triaxial cell shear tests) were used to determine the following ranges of variation for the main geomechanical parameters:

c_u = undrained cohesion = 0.15 - 0.4 Mpa (= 1.5 - 4 Kg/cm^2)

c' = drained cohesion = 0 - 0.2 MPa (= 0 - 2 Kg/cm^2)

$Ø_u$ = undrained angle of friction = 0° - 10°

Ø' = drained angle of friction = 18° - 24°

E = Young's elastic modulus = 500 - 5,000 Mpa (= 5,000 - 50,000 Kg/cm^2).

2.3.1.3 DIAGNOSIS PHASE FOR THE "VASTO" TUNNEL

The input for this phase was the geological, geotechnical, geomechanical and hydrogeological knowledge obtained using theoretical and experimental methods on the data from in situ surveys and laboratory tests performed on the ground in question. It was used to make forecasts of the stress-strain behaviour of the face and the cavity in the absence of intervention to stabilise the tunnel. The purpose was to divide the tunnel into sections, each with uniform deformation behaviour in terms of the three basic stress-strain conditions that may occur .

The diagnosis study then continued with an analysis of the shear mechanisms and instability kinematics that would be produced following the development of deformation phenomena and concluded with an estimate of the extension of the unstable zones and the size of the loads mobilised which, however, do not fall within the scope of this article.

2.3.1.4 FORECASTING STRESS-STRAIN BEHAVIOUR

Forecasts of the stress-strain behaviour along the route were made using two different procedures *(fig. 28)*, both valid for low, medium and high stress states: the first, more immediate, is based on **characteristic line theory** (calculated using analytical or numerical methods according to the situation), the other, rather lengthier, is based on triaxial cell **extrusion tests**, mentioned in the paragraph on the survey phase.

In the case of the "Vasto" tunnel, apart from short sections near the portals, both procedures forecast unstable face behaviour with considerable extrusion and, as a consequence, also preconvergence and convergence (over 100 cm. radially). These values are such as to produce serious manifestations of instability, such as the failure of the face and, as a consequence, the collapse of the cavity.

2.3.1.5 THE THERAPY PHASE FOR THE "VASTO" TUNNEL

The forecasts produced in the diagnosis phase were used as a basis for deciding on the type of action to exert (preconfinement or simple confinement) and on the intervention required, in the context of the behaviour category that was forecast, to achieve complete stabilisation of the tunnel.

First, consideration was given to the characteristics of the ground to be tunnelled (including the Southern portal to be opened under land slip conditions) and the results of the diagnosis phase which forecast face unstable behaviour for the whole length of the underground route (stress in the failure range, zero arch effect, typical manifestations of instability: collapse of the face, failure of the cavity). It was then decided to stabilise the tunnel by exerting preconfinement action, intervening decidedly ahead of the face to guarantee the formation of an artificial arch effect, again, ahead of the face.

Translated into more detail, it was decided to employ full face advance after first adopting mixed conservation techniques to create a preconfinement effect by acting both around the core (protective action) and on it directly (reinforcement action).

Three alternative tunnel section types *(fig. 9)* were then designed to be adopted according to the homogeneity and consistency of the ground encountered during tunnel advance.

The only difference between the three types was the type of treatment (preconfinement or prereinforcement)

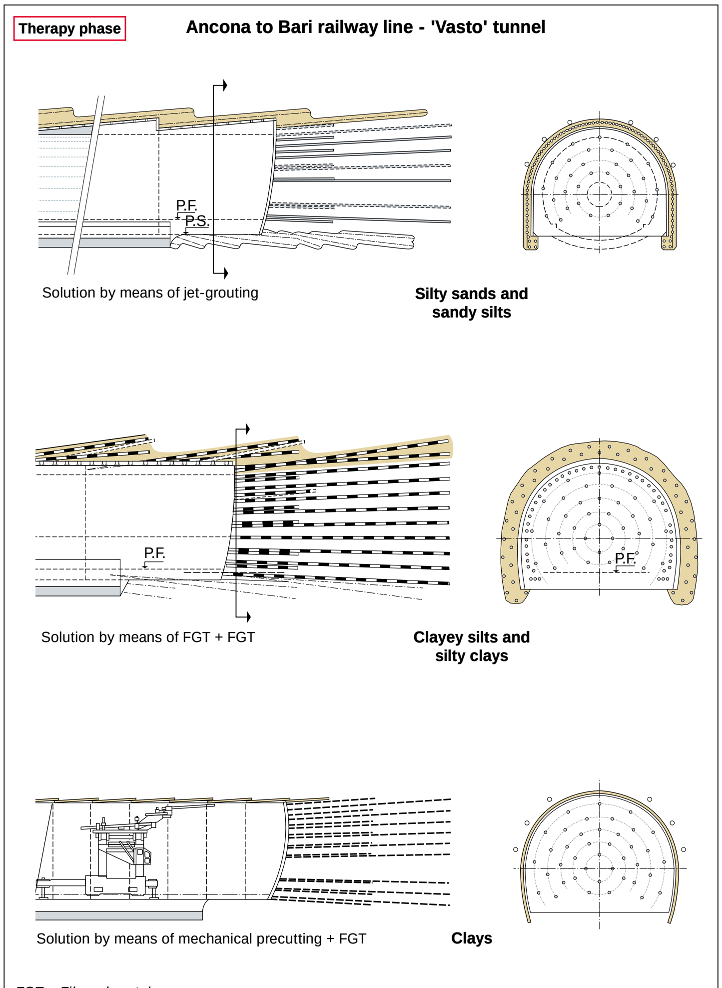


Fig. 9

to be employed ahead of the face around the future cavity, while the method of ground reinforcement employed in the advance core was the same for all three.

The choice of technique to be employed around the core was strictly dependent on the nature and the acquired consistency of the ground to be tunnelled.

In granular ground or where cohesion was poor, characterised by weak shear strength, horizontal jet-grouting was specified.

The most appropriate technology to create strong shells ahead of the face in cohesive, compact and homogeneous ground, able to guarantee the mobilisation of an "arch effect", is, as is now well known, that of mechanical precutting.

In grounds where the values for undrained shear strength and cohesion make the use of this technology inadvisable a band of improved ground ahead of the face around the core and the cavity can be obtained using claquage injections performed using specially fitted fibre glass tubes.

All three tunnel section types were completed with a preliminary confinement action down from the face consisting of steel ribs and shotcrete, closed with the tunnel invert and then afterwards, the placing of the final lining in concrete.

Once the tunnel section type had been decided, design of the reinforcement of the advance core using fibre glass tubes was performed. This consisted of deciding the number of tubes to be inserted, their length and the geometry of their configuration on the face.

As with the approach adopted in the diagnosis phase to forecast cavity behaviour, two different procedures were adopted to decide on the number of the fibre glass tubes *(fig. 10)*.

The first procedure was based on the characteristic line method, taking into account, in a simplified manner, the effect of reinforcing the advance core when calculating the corresponding characteristic line.

The second procedure for the design of ground reinforcement of the advance core was based on interpretation of the extrusion curves obtained from triaxial extrusion tests. Having first identified the minimum confinement pressure curve P_i required to stabilise the face (defined as the borderline pressure between the "elastic" and the "elastic-plastic" arm of the extrusion curve), experimental diagrams of the type shown in the same figure were used to calculate the number of tubes required to guarantee the safety of the face with the desired safety coefficient.

Both approaches (extrusion tests and characteristic lines) furnished comparable results, in confirmation of the conceptual analogy that they have in common.

2.3.1.6 THE OPERATIONAL PHASE FOR THE "VASTO" TUNNEL

In 1992 work resumed almost simultaneously on both portals, at the North portal to repair the collapsed section of tunnel and at the South portal to begin tunnel advance. Average advance rates, working 7 days per week were approximately 50 m. per month of finished tunnel (photo 3).

Figure 11 compares charts for average monthly advance rates graphs with those for convergence measured during the same period. The distinct tendency of the latter to follow the former in inverse proportion is particularly significant and confirms the fact that the less time that the core has in which to deform, the less extrusion and preconvergence is triggered. Since convergence depends on the extrusion and preconvergence; it is more limited as a consequence.

Photo 3: Vasto Tunnel: reinforcing of the advance core with fibre-glass tubes during full face excavation (according to A.DE.CO.-RS principles).

2.3.1.7 THE MONITORING PHASE DURING CONSTRUCTION FOR THE "VASTO" TUNNEL

The monitoring phase began at the same time as excavation and involved interpreting the deformation response of the medium to excavation for the purpose of optimising and calibrating the various techniques employed to stabilise the tunnel.

In addition to the normal measurements of convergence and pressure, systematic and simultaneous measurements were also taken of extrusion and convergence for the "Vasto" tunnel. These constituted a novelty of particular interest, especially considering the results that they have furnished to date.

The results of these measurements are summarised in the graphs given in *fig. 12* that show the simultaneous course of extrusion and convergence for a complete work cycle.

Examination of the graphs shows that as the face advances and as the depth of the reinforced part of the advance core reduces from an initial 15 m. to only 5 m. (with a consequent reduction in its average rigidity) a deformation response of the face itself (extrusion) and of the cavity down from the face (convergence) develops and this gradually moves from the elastic range towards the elastic-plastic range. The convergence curves in particular start with an initial tendency typical of a situation which moves rapidly towards stability (maximum values of the order of 10 cm., which are produced following maximum values for extrusion of less than 2.5 cm.) and then gradually change course in a direction indicating an increasing probability of deformation being unable to stop. For example, when the length of the reinforced core falls below 5 m. extrusion of the order of 10 cm develops which gives rise to convergence four times greater than that measured at the beginning of the work cycle.

In this context, then, combined reading of extrusion and convergence of the cavity is an extremely important indicator for the design engineer to be able to establish the moment at which advance must halt to carry out another cycle of reinforcement and restore the minimum depth of reinforced core required to maintain the ground, if not in the elastic range at least away from the failure range.

2.3.2 RESULTS OF THE THIRD RESEARCH STAGE

The study and experimentation performed on the Vasto tunnel clearly showed both the existence of a close connection between deformation that occurs in the advance core of a tunnel (extrusion) and that which develops subsequently around the cavity after the passage of the face (convergence) and also (*fig. 13*, results of the third research stage), that deformation of the cavity

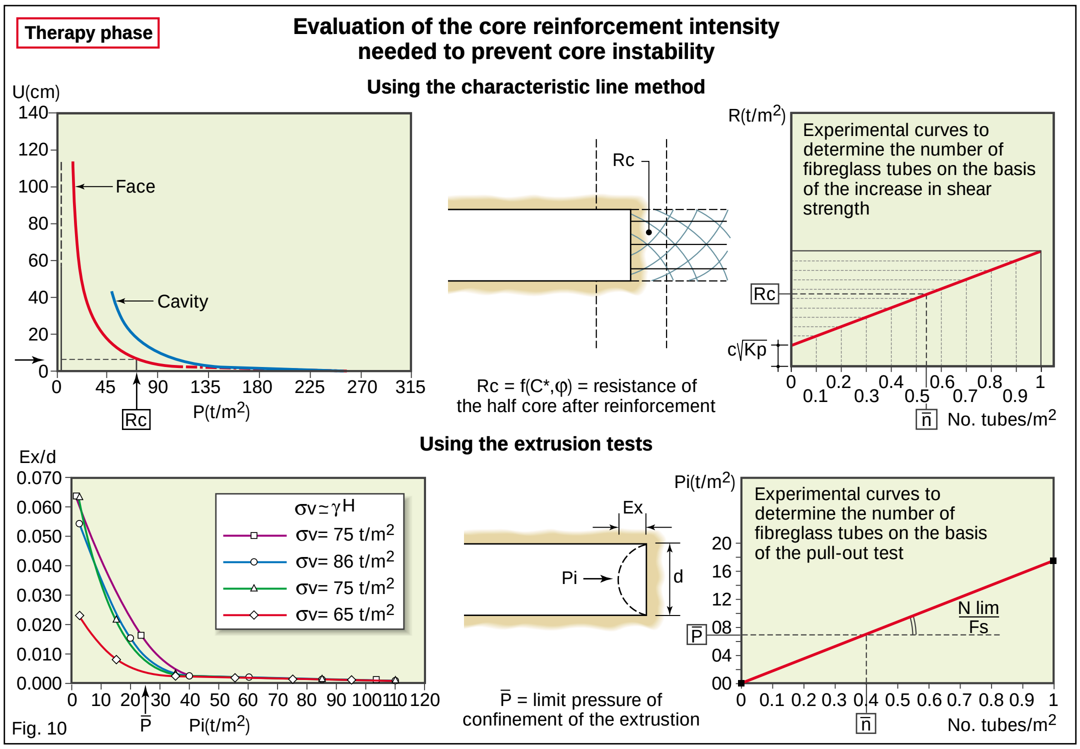


Fig. 10

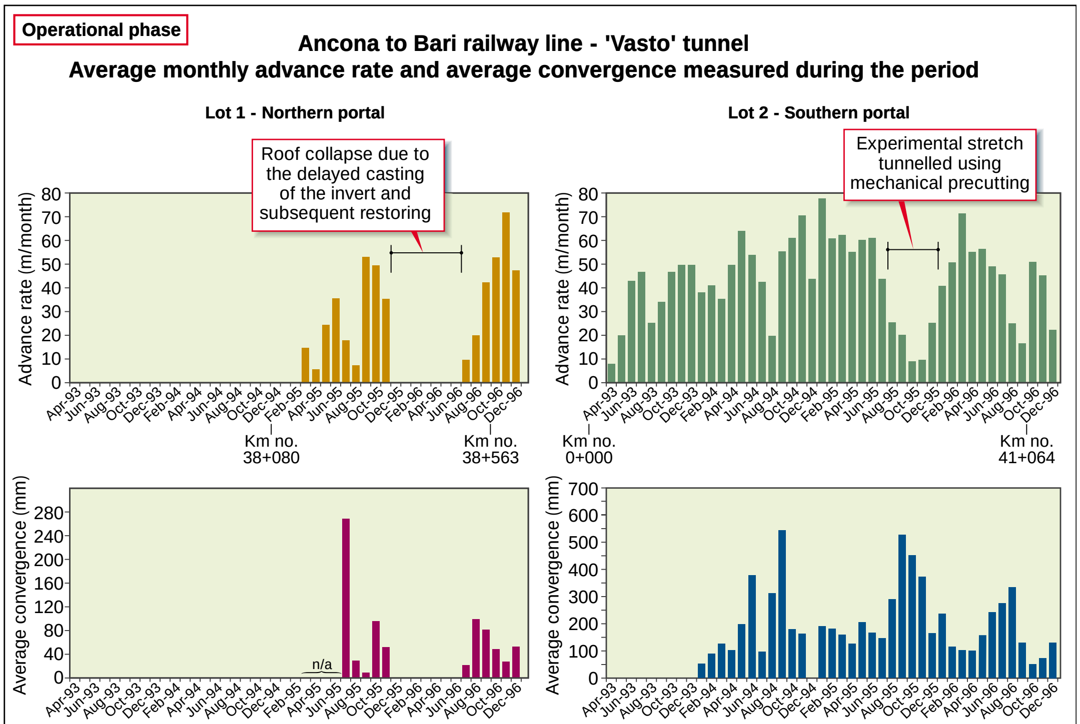


Fig. 11

can be controlled and considerably reduced by artificially regulating the deformation properties, and therefore the rigidity, of the advance core (confinement of extrusion). This is possible by employing appropriate stabilisation techniques carefully dimensioned and balanced between the core at the face and the cavity, as a function of the strength and deformation properties of the medium in relation to the contingent stress conditions.

In this respect, if a medium is stressed in the elastic-plastic range:

- if the stress state is low relative to the characteristics of the medium, then it may be sufficient to act on the cavity only, using radial measures with no longitudinal measures into the advance core at all;
- if, however, the stress state is high then it will be necessary to act above all on the advance core reinforcing it with longitudinal measures, making no use at all of radial measures after the passage of the face.

If a medium is stressed in the failure range, it is imperative to stiffen the advance core with preconfinement action on the future cavity. This may be supplemented with appropriate confinement action down from the face. In these cases experience (and that described in the previous paragraphs is particularly significant) advises:

- working ahead of the face on the form and volume of the core by creating a protective crown of improved ground around it. During the construction of the Vasto tunnel, this way of working was effectively used to advance through particularly difficult ground.

If this is not sufficient then it will be necessary to:

- perform further radial ground improvement around the cavity of a size sufficient to absorb residual convergence that the core, though stiffened, is not able to prevent alone.

In the latter case, the balancing of intervention between the core and the cavity decided at the design stage, can be adjusted or "fine tuned" during construction.

2.4 THE ADVANCE CORE AS A STABILISATION TOOL

The results of the research can be very briefly summarised as follows:

- during the "first research stage" three fundamental types of deformation (face extrusion at the face, preconvergence and convergence) and consequent manifestations of instability (fall of ground, spalling, failure of the face and collapse of the cavity) were identified;
- in the "second research stage" experimental confirmation was obtained showing that all the deformation behaviour (extrusion at the face, preconvergence and convergence) and manifestations of instability visible inside the cavity and resulting from the deformation (fall of ground, spalling, failure of the face and collapse of the cavity) depend directly or indirectly on the rigidity of the advance core;
- in the "third research stage" experimentation was performed on how it was possible to use the advance core as a stabilisation tool by acting artificially on the rigidity of the core itself to regulate deformation of the cavity.

The results of the research were as follows:

- they confirmed that the deformation response of the medium to the action of excavation must be the principal question with which a tunnel designer is concerned, because, amongst other things, it is an indicator of the triggering and position of an arch effect or in other words the level of stability reached by the tunnel;
- they show that the deformation response begins ahead of the face in the advance core and develops backwards from it along the cavity and that it is not only convergence, but consists of extrusion, preconvergence and convergence. Convergence is only the last stage of a very complex stress-strain process;
- they clearly indicate the existence of a direct connection between the deformation response of the face-advance core system and that of the cavity in the sense that the latter is a direct consequence of the former underlining the importance of monitoring the deformation response of the face-advance core system and not just the cavity;
- they demonstrate that it is possible to control deformation of the advance core (extrusion, preconvergence) and as a consequence also control deformation of the cavity (convergence) by acting on the rigidity of the core employing measures to protect and reinforce it.

In conclusion the results of the research allow the advance core to be seen as a new stabilisation instrument in the short and long term for the cavity: an instrument whose strength and susceptibility to deformation play a determining role as it is able to condition that aspect which should worry tunnel designers more than any other: **the behaviour of the cavity when the face arrives**.

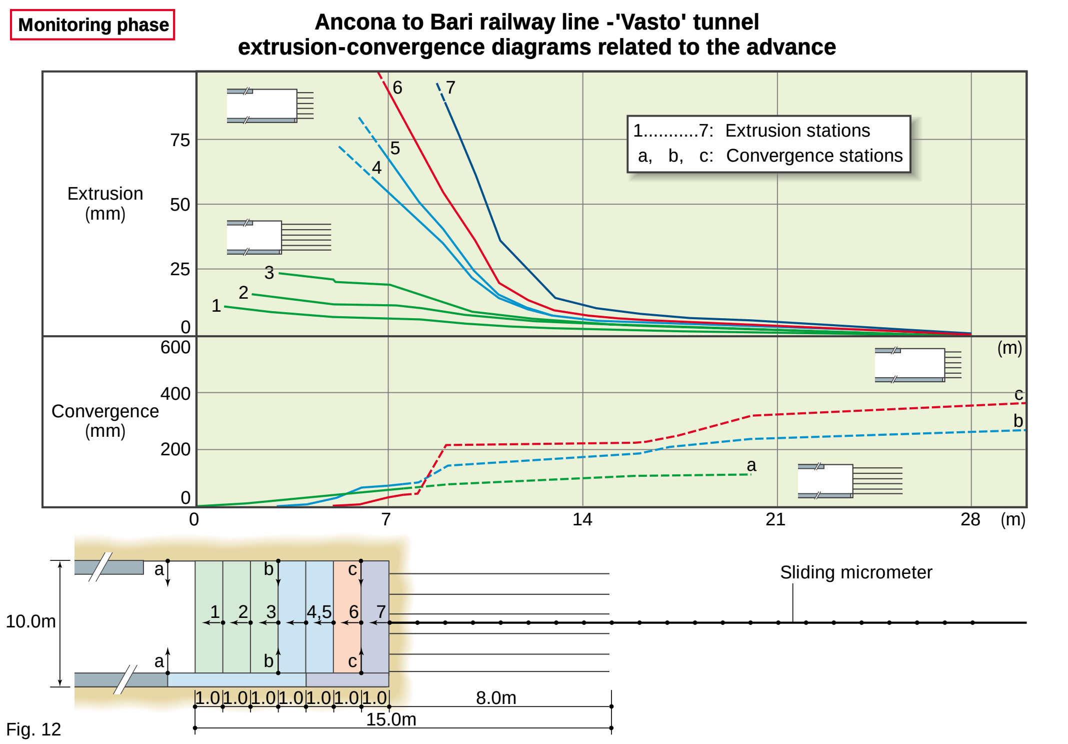


Fig. 12

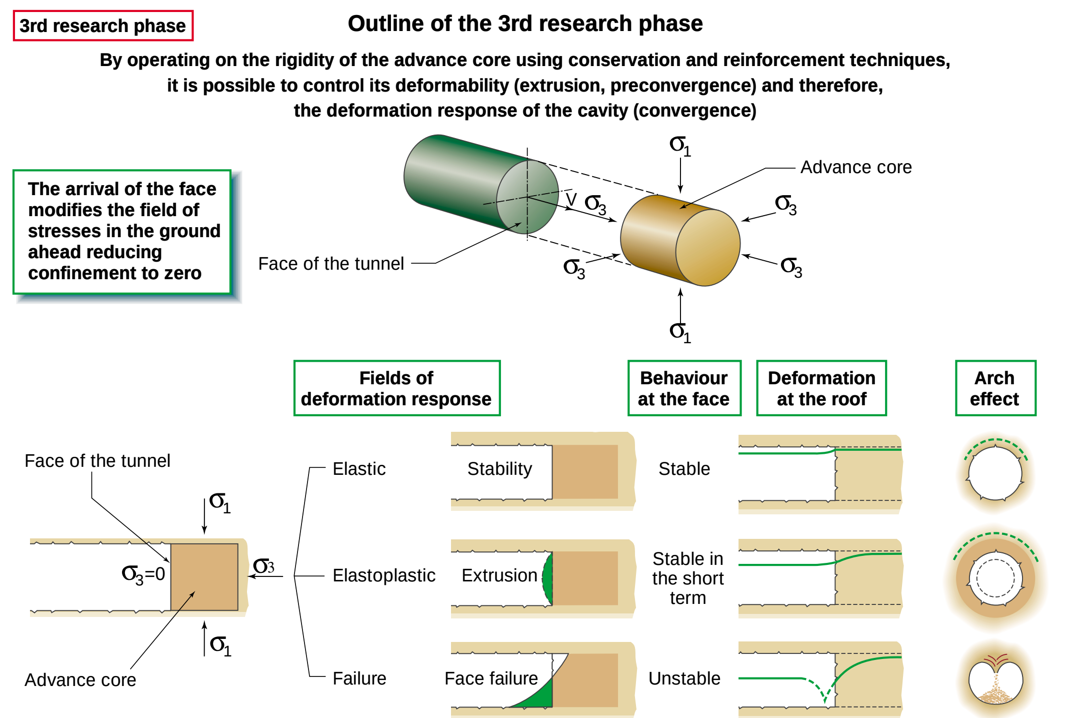


Fig. 13

3. THE ADVANCE CORE AS A POINT OF REFERENCE FOR TUNNEL SPECIFICATIONS

If the advance core is an effective tool for long and short term stabilisation of a cavity, capable of conditioning its behaviour when the face arrives, it can be stated that tunnel designers must direct all their attention to the stress-strain properties, which is to say the stability, of the face-core system if they wish to be able to draw up designs capable of guaranteeing the long and short term stability of a tunnel.

It follows that the stability of the face-advance core system can be assumed as a point of reference for standardising tunnel specifications with the advantage that it is an indicator that conserves its validity in all types of ground and in all statics conditions.

In this context the three fundamental stress-strain conditions of the face-advance core system already described in paragraph one (see also *fig 13*) also identify three possible types of cavity behaviour *(fig. 14)*:

- stable face behaviour (behaviour category A);
- short term stable face behaviour (behaviour category B);
- unstable face behaviour (behaviour category C).

Where there is stable face behaviour, the overall stability of the tunnel is practically guaranteed even in the absence of stabilisation intervention. In situations B) and C) the results of the research indicate that in order to prevent instability of the face, and therefore of the cavity, and to try to return to stable face conditions (A), preconfinement measures must be adopted, appropriately balanced between the face and the cavity, of an intensity adequate to the actual stress conditions relative to the strength and deformation properties of the medium.

The application of these concepts in

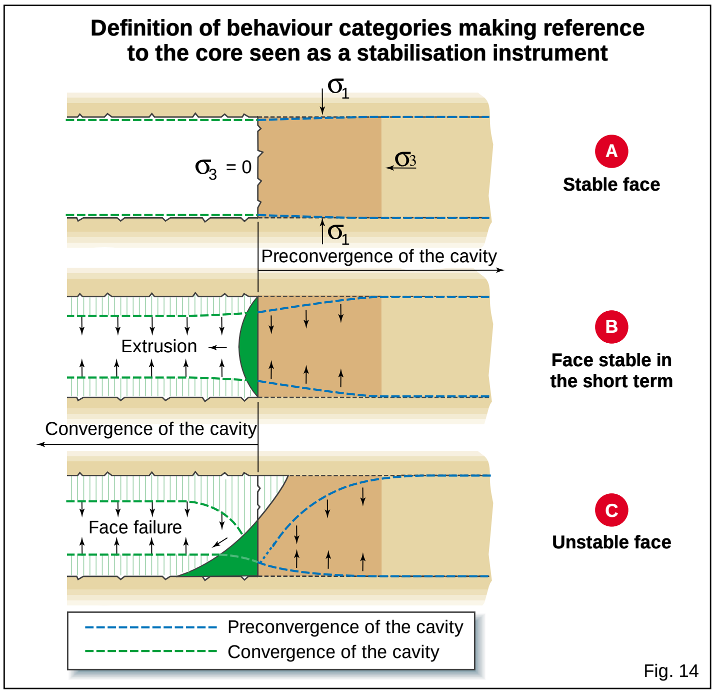


Fig. 14

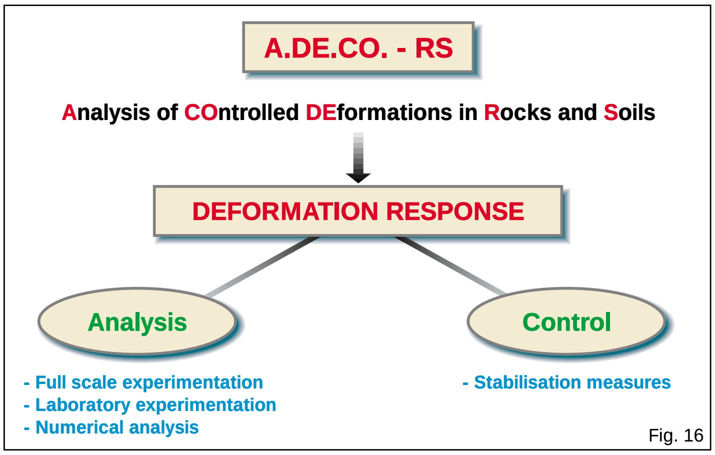


Fig. 16

the practice of design and construction has allowed the author to achieve numerous significant successes. *Fig. 15* groups together graphs of advance rates obtained during the excavation of tunnels designed and constructed in Italy over the last ten years, under an extremely wide range of different geological conditions and stress states [2]. What is striking here is not only the high average advance rates relative to the type of ground involved, but above all the linearity of the rates, an indicator of industrial type construction performed in regular stages and without hold-ups.

At this point it seemed both necessary and urgent to take the knowledge acquired to its extreme conclusion and to develop a design and construction approach which adhered more to reality than do those in common use.

To achieve this along the lines of the research already performed, and to complete it, a further programme of study, both theoretical and experimental had to be started in which the stress-strain behaviour of the advance core, systematically compared to that of the cavity, was studied in terms of stability and deformation both in the absence and the presence of protective and reinforcement intervention.

The A.DE.CO.-RS approach (acronym of the Italian for Analysis of COntrolled DEformation in Rocks and Soils) *(fig. 16)* sprang from and also came to crown these studies. This approach, by observing:

- that the phenomena involved in the excavation of a tunnel can be brought down to a process of cause and effect (action-reaction);
- that normally in order to effectively control the effect in these types of processes it is necessary to first fully identify the cause;
- that complete identification of the cause must necessarily involve in-depth analysis of the effect;

attention is drawn to the latter (deformation response of the ground) both ahead of and down from the face and that by analysing first its genesis and development employing full scale and laboratory experimentation as well as mathematical tools focused on the behaviour of the advance core, the cause can be identified in the deformation properties of the ground ahead of the face.

By then controlling the susceptibility to deformation of the ground ahead of the face (advance core) by using appropriate stabilisation techniques, it is found that it is possible to thereby control the deformation response of the ground too, incontrovertible evidence that it is the true cause of the process under examination.

4. ANALYSIS OF THE DEFORMATION RESPONSE ACCORDING TO THE A.DE.CO.-RS APPROACH

The stress-strain behaviour of the advance core, compared systematically to that of the cavity was analysed in terms of stability and deformation both in the absence and the presence of protective and reinforcement intervention employing a series of observations and measurements both in situ and in the laboratory.

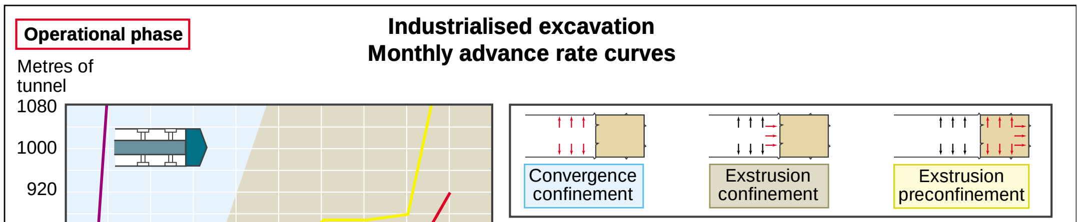
4.1 FULL SCALE EXPERIMENTATION

The behaviour of the advance core **in terms of stability**, was analysed following an observational approach. More than 1,000 tunnel faces were classified and the data for them summarised on special cards.

In deformation terms, on the other hand, the advance core was studied by systematic measurement of *(fig. 17)*:

- extrusion, obtained by equipping the advance core with a horizontal extrusion meters (sliding micrometer type) of a length equal to 2 - 3 times the diameter of the excavation. These furnish longitudinal deformation in absolute terms of the ground that makes up the advance core, both as a function of time (static phase, face halted) and as a function of face advance (dynamic phase) *(fig. 18)*;
- topographical plottings of movements in absolute terms of the face, by means of optical targets, taken with the face at a halt;
- preconvergence taken from the surface, whenever the morphology of the terrain and the size of the overburden in question allowed, by setting up multibase extensometers, inserted vertically into the ground before the arrival of the face above the crown and sides of the tunnel being driven [3].

These measurements were naturally always accompanied by traditional measurements such as measures of



Work	ø (m)	Ground	Excavation technique	Production (m/day) Aver.	Production (m/day) Max
T.Prato Tires (F.S.)	3.5	Ignimbrite	TBM	35	80
Milan Urban Link Line (F.S.)	8	Sand and gravel	EPB shield	5.3	22
Rome Metro	10.64	Heterogeneous sands and gravel	Hydroshield	4.0	5.5
'Campiolo' Tunnel (F.S.)	12.20	Rubble-slope	Horizontal jet-grouting (H.J.G.)	1.7	2.5
Tunnel on Sibari to Cosenza Railway (F.S.)	10.50	Clay	Precutting	2.3	3.2
Tunnel on Florence to Arezzo Railway (F.S.)	12.20	Clay	Precutting + F.G.T	2.0	3.2
'St Vitale' Tunnel (F.S.)	12.50	Clay	F.G.T. + F.G.T.	1.6	2.4
'Vasto' Tunnel (F.S.)	12.20	Silty sand and clayey silt	H.J.G.+ F.G.T.	1.8	2.3
Tartaiguille Tunnel (SNCF)	15.00	Marly clay	F.G.S.E.	1.4	1.9

F.G.T. = Fibre glass tubes
F.G.S.E. = Fibre glass structural elements

Fig. 15

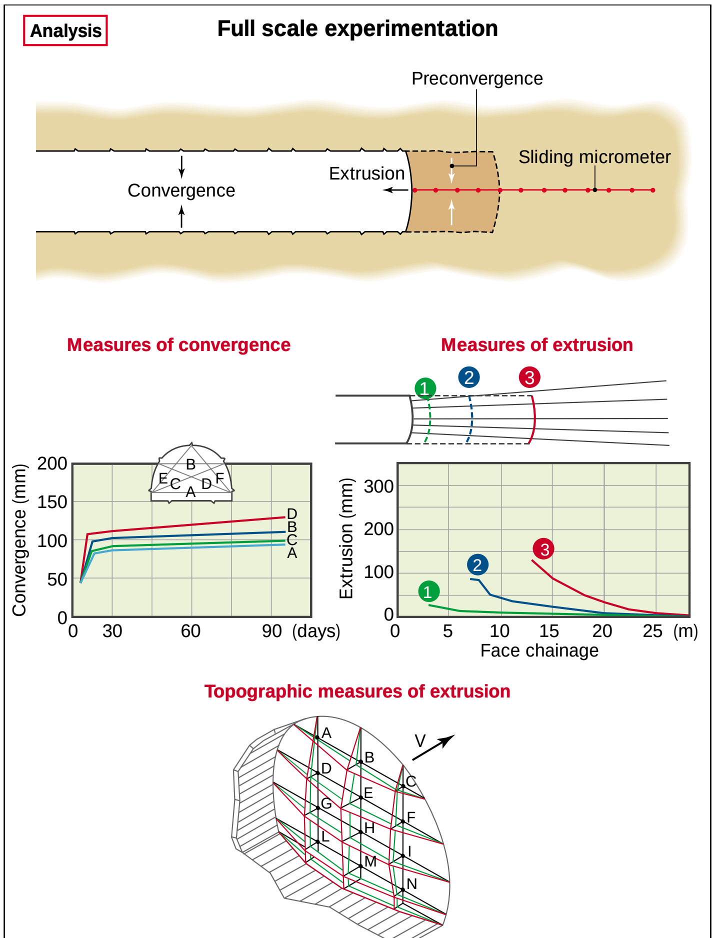


Fig. 17

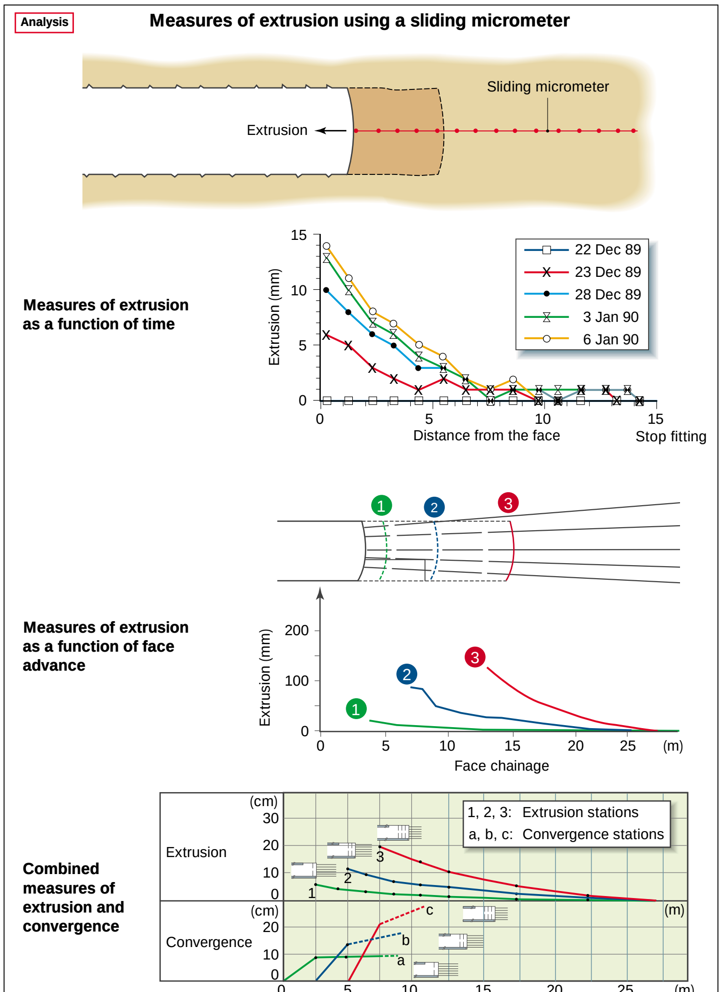


Fig. 18

convergence and of stress on linings.

Full scale experimentation allowed the following to be achieved:

- confirmation, by the construction of special extrusion-convergence diagrams *(fig. 18)* of the existence of a close correlation between the entity of extrusion in the advance core and that of convergence that manifests after the passage of the face and also how these both decrease as the rigidity of the core is increased;
- establishing that the advance core extrudes through the wall of the face (surface extrusion) with three different basic types of deformation (cylindrical, spherical crown, combined) as a function of the material involved and the stress state it is subject to;
- calculation in absolute terms of preconvergence, using simple volumetric calculations that can be easily performed using tables, even when it is not possible to measure it directly from the surface *(fig. 19)*;
- verification that as preconfinement action of the cavity increases and the band of plasticised ground around the tunnel decreases as a consequence, there is subsequently a proportionally smaller load on the preliminary and final linings.

4.2 LABORATORY EXPERIMENTATION

Since the already existing extrusion tests, invented by Broms and Bennemark in 1967 studied the phenomenon only in terms of trigger thresholds, two new types of tests were developed in order to analyse the course of the phenomenon *(fig. 20)*:

- the triaxial cell extrusion test;
- the centrifugal extrusion test.

In the triaxial cell extrusion test, the sample of ground is inserted into a cell and the original stress state σ_o of the rock mass is recreated. The pressure of the fluid is then used to also reproduce the stress state inside a special cylindrical volume termed an “extrusion chamber” and cut out of the inside of the test sample before the test. The chamber is coaxial to the sample and simulates the situation of a tunnel around the face.

By maintaining the stress state around the sample constant and gradually reducing the pressure P_i of the fluid inside the extrusion chamber, a realistic simulation is obtained of the gradual decrease in stress produced in the medium for a given cross-section of tunnel as the face approaches. A forecast of extrusion at the face as a function of time or as a function of the decrease in internal confinement pressure P_i is obtained with curves similar to those in *fig. 20*. They can be immediately used in the design stage for calculating the preconfinement pressure required to guarantee a given rigidity of the core and thereby, as a consequence the desired control of preconvergence.

Some considerations can be formulated from the numerous extrusion tests performed in triaxial cells :

1. given the modest dimensions of the sample, these tests apply mainly to the matrix of the ground, which must be mainly clayey;
2. any non uniform elements in the ground (schists or scaliness) only give good results if they are of negligible size relative to that of the sample;
3. the more homogeneous the ground the more probability there is of the results being applicable to the full scale situation.

Centrifuge extrusion tests were developed and performed for those cases in which gravity has a significant influence on extrusion. Use of these is limited to a few specific cases due to their high cost and complexity.

Special markers and transducers for measuring deformation and interstitial pressure are inserted in the sample of ground, which is then placed in a special box with a transparent wall. After having cut out the opening of the tunnel in this, a steel tube is inserted representing a first approximation of the preliminary lining, the lining and the tunnel invert. The cell thus obtained is filled with a fluid kept under pressure. The natural geostatic pressure is then recreated in the centrifuge and once this is reached the pressure in the cell is reduced to simulate the excavation of ground at the face.

The results obtained (*fig 20* gives those for a centrifuge extrusion test performed on a reconstituted sample of ground) show that extrusion at the face manifests rapidly at the moment in which the pressure is released and develops with increasing speed as relaxation of the core progresses. The figure shows both the instant and the viscous extrusion separately for each step of pressure release. The figure clearly shows that the latter accounts for 50% of total extrusion at the end of the test.

Laboratory experimentation, by reproducing the phenomenon of extrusion of the advance core in the laboratory together with the results of full scale measurements was fundamental for the correct weighting of the geomechanical strength and deformation parameters (c, ø, E) in the mathematical models used in the theoretical part of our analysis on controlling deformation responses.

4.3 NUMERICAL ANALYSES

The complexity of the mechanisms that are triggered ahead of the face and the initial difficulty in identifying objective criteria for predicting the stress-strain behaviour of the advance core that go beyond intuition and practical experimentation, involved making the effort to produce an organic and unitary interpretation of the numerous aspects investigated which in turn would provide a general theoretical framework capable of overcoming the limits of current theories.

To do this, our analysis of the deformation response continued using theoretical tools. Three different approaches were employed:

- initially we tried to make use of existing calculation theories updating them where necessary;
- we then sought to solve the problems using finite elements and finite differences axial-symmetric mathematical models;
- finally we resorted to three dimensional mathematical modelling.

4.3.1 STUDIES USING ANALYTICAL APPROACHES

To start with, we tried to solve the problem by modifying and bringing existing analytical calculation approaches up-to-date. In particular we sought to introduce the concept of the core and reinforcement of it into some of the classic formulas used for designing tunnels, for example the Convergence-Confinement Method [4] and the Characteristic Line Method [5], this being the only one in which the concept of the core appears explicitly.

Both of these methods allowed us to simulate the effects of ground improvement to the core and to reproduce some of our experimental results, the resulting reduction of the radius of plasticisation R_p and the deformation in the zone around the face.

Due to the separation in the calculation between the stress-strain situation at the face and that at a distance from it, the two methods cannot hold in memory the effects of what has happened ahead of the face in the equations that are valid for the zone back from the face. Consequently they are not able to interpret and represent the phenomena correctly as a whole [6].

The phenomena referred to here are the decrease in the radius of plasticisation R_p and the consequent reduction in the deformation of the cavity (convergence) and in the loads acting on the preliminary and final linings. These phenomena were not found in the results obtained using the analytical methods considered, while they have been systematically observed from experimental measurements [6].

We were therefore forced to conclude that these approaches, though useful in the diagnosis phase for defining the behaviour of the ground when excavated in the absence of preconfinement intervention, they are not of use in the therapy phase where preconfinement is employed. This is because they cannot predict deformation of the cavity with sufficient accuracy and similarly with the calculation of the preliminary and final linings.

It was therefore decided to abandon these types of approach and to take the path of numerical models (finite elements or finite differences), which are able to take into account the whole stress-strain history of the medium around the excavation passing from the area ahead of the face to that behind it.

4.3.2 STUDIES USING AXIAL-SYMMETRIC NUMERICAL MODELLING APPROACHES

The effect of improving the ground in the core was therefore investigated using finite elements and finite differences models. We began using axial-symmetric type models since they are easier to manage than three dimensional models.

Although we were unable to overcome some of the intrinsic limitations of the above mentioned analytic methods (tunnel perfectly circular, uniform stress states in the ground around it, the impossibility of considering any linings other than closed ring linings and therefore of simulating real construction stages), the use of these models did nevertheless show that ground reinforcement of the core produces a different distribution of stresses ahead of the face and around the tunnel. This finally gave confirmation from the use of calculation that ground reinforcement of the core results in a reduction both ahead of the face and back from it, in the extension of the band of plasticised ground and of all deformation around the tunnel (not only extrusion and preconvergence but also convergence). In addition, analyses conducted using axial-symmetric models showed that it is not possible to control extrusion and preconvergence by varying the rigidity of tunnel linings and/or the distance from the face at which they are placed alone. In other words, they demonstrated that it is impossible to remedy what has already happened ahead of the face with confinement action only.

Although axial-symmetric models showed a discreet capacity to simulate tunnel advance in the presence of ground reinforcement to the advance core and furnished results on stress and deformation of the ground in line with the findings of experimental research, they did not show an equal capacity to predict loads acting on the preliminary and final linings, which with this type of model would be more or less equivalent to those that would result in the absence of ground reinforcement to the core, other conditions remaining unchanged.

This contrasts, as has already been said, with observations made during experimental research and confirmed many times in practice during construction. It is due to the impossibility with these types of models of taking account of the gravitational effects produced by the plasticised ground around the tunnel and of the real construction stages involved in placing the preliminary and final linings.

4.3.3 STUDIES USING 3D NUMERICAL MODELLING APPROACHES

In order to overcome the contradictions encountered with axial-symmetric numerical models, resort was made to three dimensional numerical modelling. With this method it was in fact possible to introduce the real geometry of the tunnel into the calculations so that is no longer circular as in the case of the Convergence-Confinement method, of Characteristic Line method and (finite elements or finite differences) axial-symmetric analysis.

It is also possible to consider stress states of the ground that are not of the hydrostatic type, which take due account of gravitational loads and which also calculate the effects which the various construction stages have on the statics of the tunnel by simulating the real geometry of lining structures and the sequence and the distance from the face in which they are placed. It was therefore possible, as is illustrated below, to employ numerical calculation to study how the distribution of extrusion movements at the face and the consequent different failure mechanisms vary as a function of the distance at which the tunnel invert is placed.

The results obtained from 3D models generally agree well with experimental observations, both as far as deformation is concerned (extrusion, preconvergence and convergence) and with regard to stresses on linings, which are lower when the advance core is reinforced, as found in experimental research.

4.3.4 RESULTS OF EXPERIMENTAL AND THEORETICAL STUDY OF THE DEFORMATION RESPONSE

Experimental and theoretical analysis of the deformation response <u>using the advance core as the key means of interpreting long and short term deformation phenomena in tunnels</u> enabled us to identify, with certainty, the strength and deformation properties of the advance core as the true cause of the whole deformation process (extrusion, preconvergence and convergence). It also confirmed beyond any reasonable doubt that it is possible to control deformation of the advance core (extrusion, preconvergence) by stiffening it with protective and reinforcement techniques and as a consequence also control the deformation response of the cavity (convergence) and the size of the long and short term loads on the tunnel lining.

Consequently, if the strength and deformation properties of the advance core constitute the true cause of the

Analysis — **Calculation of the preconvergence of the cavity using experimental measures of extrusion**

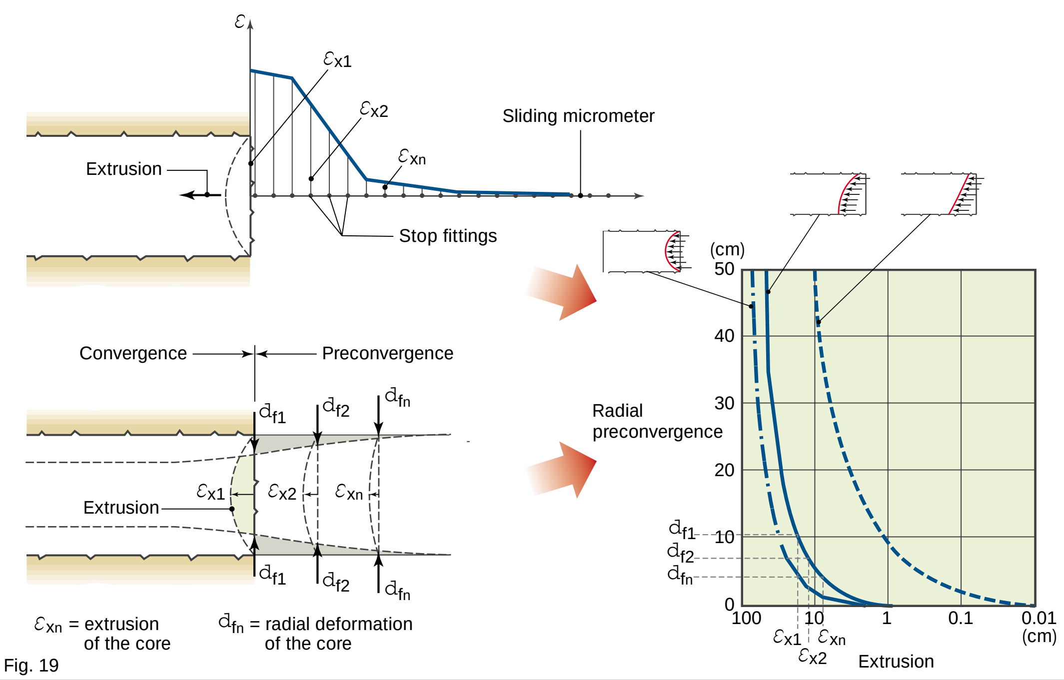


Fig. 19

deformation response of the ground to excavation, it is possible to consider it as a new tool for controlling that response: an instrument whose strength and deformation properties play a determining role in the long and short term stability of the cavity.

5. CONTROL OF THE DEFORMATION RESPONSE ACCORDING TO THE A.DE.CO.-RS APPROACH

The theoretical and experimental research on the deformation response of the ground shows that the true cause of the entire stress-strain process (extrusion, preconvergence and convergence) that is triggered when a tunnel is excavated lies in the susceptibility of the advance core to deform. It follows therefore that in order to succeed under all types of stress-strain conditions, but above all in difficult ground, one must act first on the advance core to regulate its rigidity appropriately. In terms of forces this means employing preconfinement and not just confinement action where preconfinement is defined as any active action which favours the formation of an arch effect in the ground ahead of the face.

It follows that complete control of the deformation response in the ground must necessarily be achieved *(fig. 21)*:

1. ahead of the face, by regulating the rigidity of the advance core using appropriate preconfinement techniques;

2. down from the face in the cavity, by regulating the manner in which the advance core extrudes with confinement techniques in the tunnel capable of providing continuous active confinement of the cavity close to the face.

5.1 CONTROL AHEAD OF THE FACE

In order to regulate the rigidity of the advance core and to thereby create the right conditions for complete control of the deformation response of the ground and therefore, in the final analysis, for complete stabilisation of the tunnel in the long and short term, the A.DE.CO.-RS approach proposes, as will be seen, numerous types of intervention that have been fully illustrated in numerous articles, some of which are listed in the bibliography [7].

All these types of intervention can be divided into two single categories *(fig. 22)*:

- protective intervention, when the intervention channels stresses around the advance core performing a protective function that ensures that the natural strength and deformation properties of the core are conserved (e.g. shells of improved ground by means of sub-horizontal jet-grouting, shells of fibre reinforced shotcrete or concrete placed in advance of the face by mechanical precutting);
- reinforcement intervention, when the intervention acts directly on the consistency of the advance core to improve its natural strength and deformation properties by means of appropriate ground improvement techniques (e.g. ground improvement of the core by means of fibre glass structural elements).

Although these types of intervention for controlling the deformation response ahead of the face have a rather circumscribed field of application in relation to the nature of the ground when considered individually, considered as a whole, they are able to guarantee solutions for all possible geotechnical conditions. Naturally, in extreme stress-strain conditions, there is no reason why two or more types of intervention cannot be used simultaneously to obtain a mixed action of protection and reinforcement *(fig. 23)*.

5.2 CONTROL DOWN FROM THE FACE

As opposed to the teachings of traditional tunnel advance principles, which ignore the cause of deformation allowing the core to deform and then require the installation of flexible linings to absorb deformation which has already been triggered (a practice which in really difficult stress-strain conditions frequently turns out to be inadequate), the application of these new concepts in tunnel advance, in the presence of a rigid core characteristic of the A.DE.CO.-RS approach, requires the use of equally rigid linings as an absolutely essential condition, if the advantage obtained by reinforcing the core ahead of the face is not to be lost behind it. It is also just as important that maximum care and attention be paid to ensure that the continuity of action in the passage from preconfinement to confinement occurs as gradually and as uniformly as possible, never forgetting that the cause of the whole deformation process that must be controlled lies in the strength and deformation properties of the advance core.

On the other hand, numerical analyses performed on computers shows extremely clearly that:

1. when extrusion is produced, it occurs through an ideal surface termed

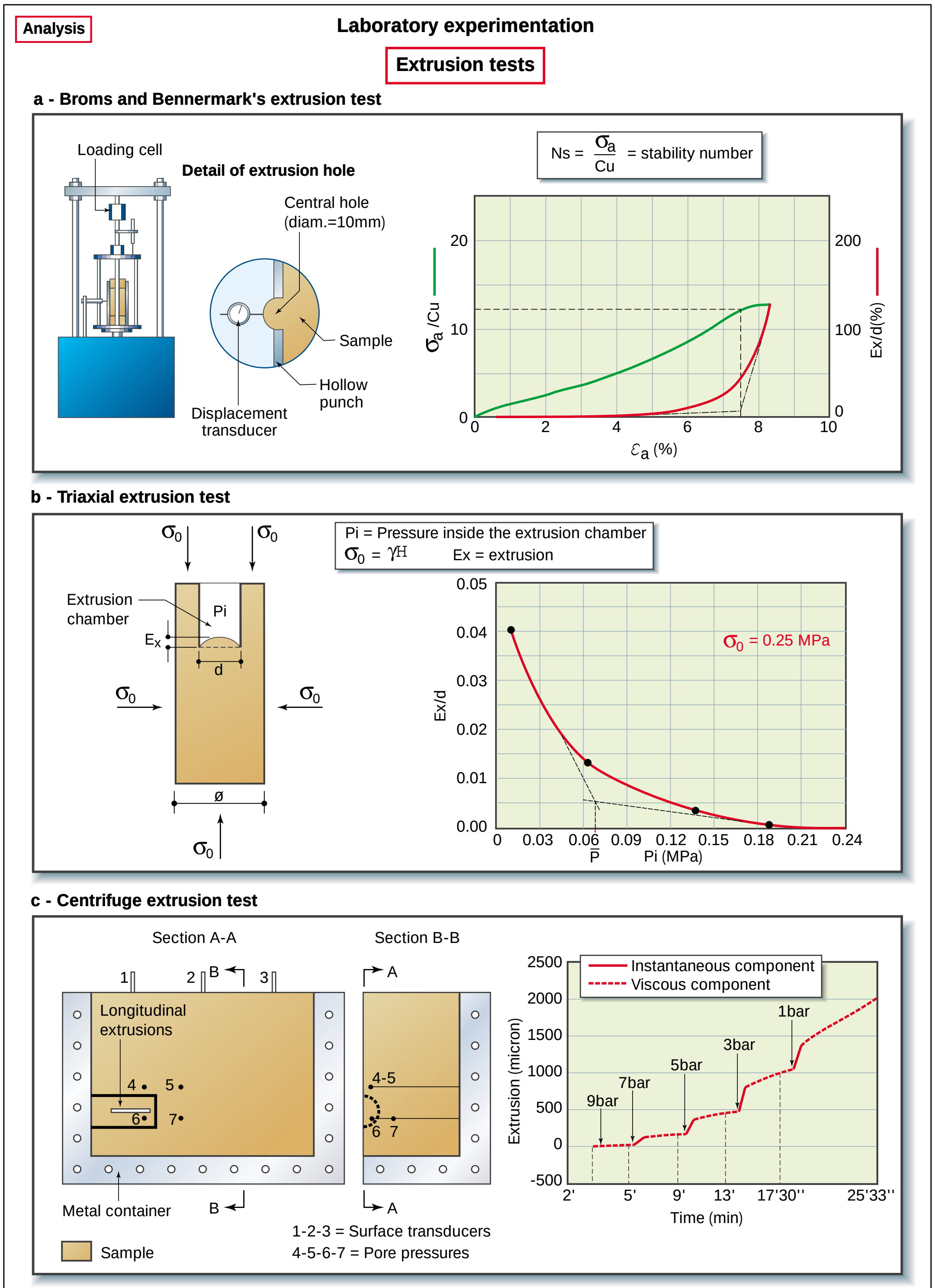


Fig. 20

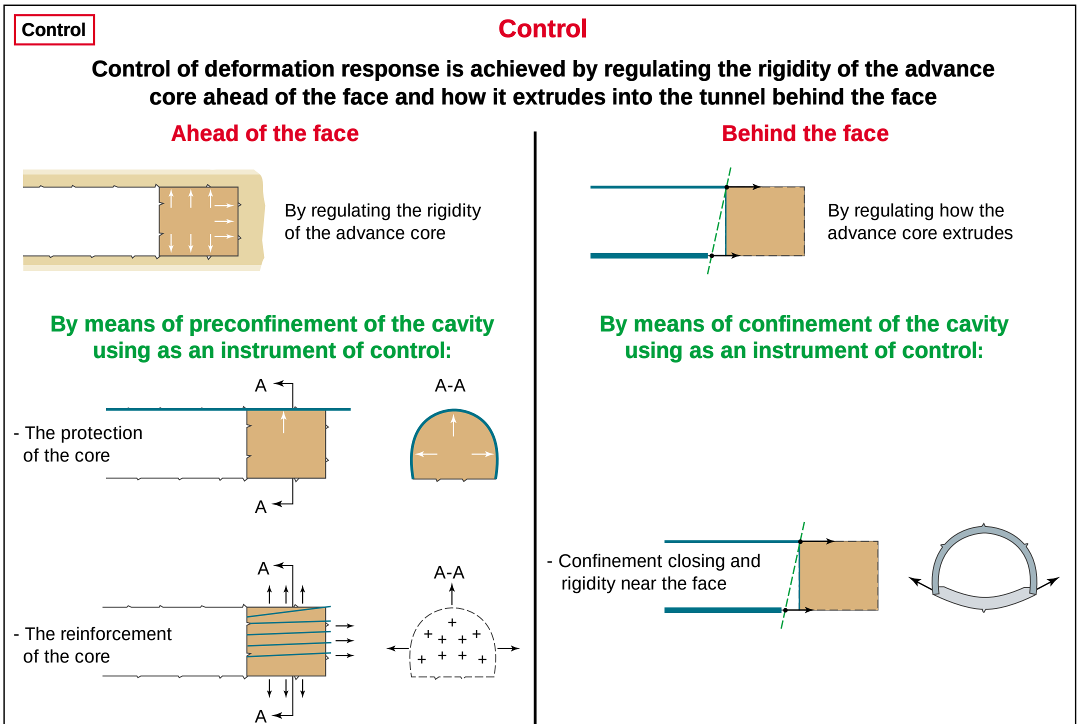


Fig. 21

the **extrusion surface**, which extends from the point of contact between the ground and the leading edge of the preliminary lining and the point of contact between the same ground and the leading edge of the tunnel invert *(fig. 24)*;

2. casting the tunnel invert closer to the face progressively reduces the extrusion surface and thereby produces an equally progressive decrease in extrusion (which tends to occur more symmetrically over the height of the face) and therefore of convergence also *(fig. 25)*.

The same numerical analyses also show that:

- with the tunnel invert cast at the same distance from the face, the deformation calculated for half-face advance is comparable to that obtained for full-face advance (in other words, casting the tunnel invert a long way from the face is like advancing in stages with headings);
- half-face advance always produces greater total deformation than full-face advance.

It follows that the tunnel design engineer has the chance (which becomes of fundamental importance in extreme stress-strain conditions) to make the action which controls the deformation response, already begun ahead of the face by regulating the rigidity of the advance core, a continuous process by casting the kickers and the tunnel invert as close as possible to the face. Not taking this chance and casting the latter far from the face means accepting a greater extrusion surface, non symmetrical extrusion and an advance core of greater dimensions that is more difficult to deal with, conditions which all lead to tunnel instability *(fig. 26)*.

At this point the time was ripe to start to translate the theoretical principles of the A.DE.CO.-RS into a new approach to the design and construction of tunnels which would overcome the limitations of traditional approaches and make it possible to design and construct tunnels in all types of ground and stress-strain conditions, to industrialise tunnel advance and even to make reliable forecasts of construction times and costs as is normal for all other types of civil engineering project. Before starting, it was essential to set down guidelines as a reference for those who set out to construct underground works.

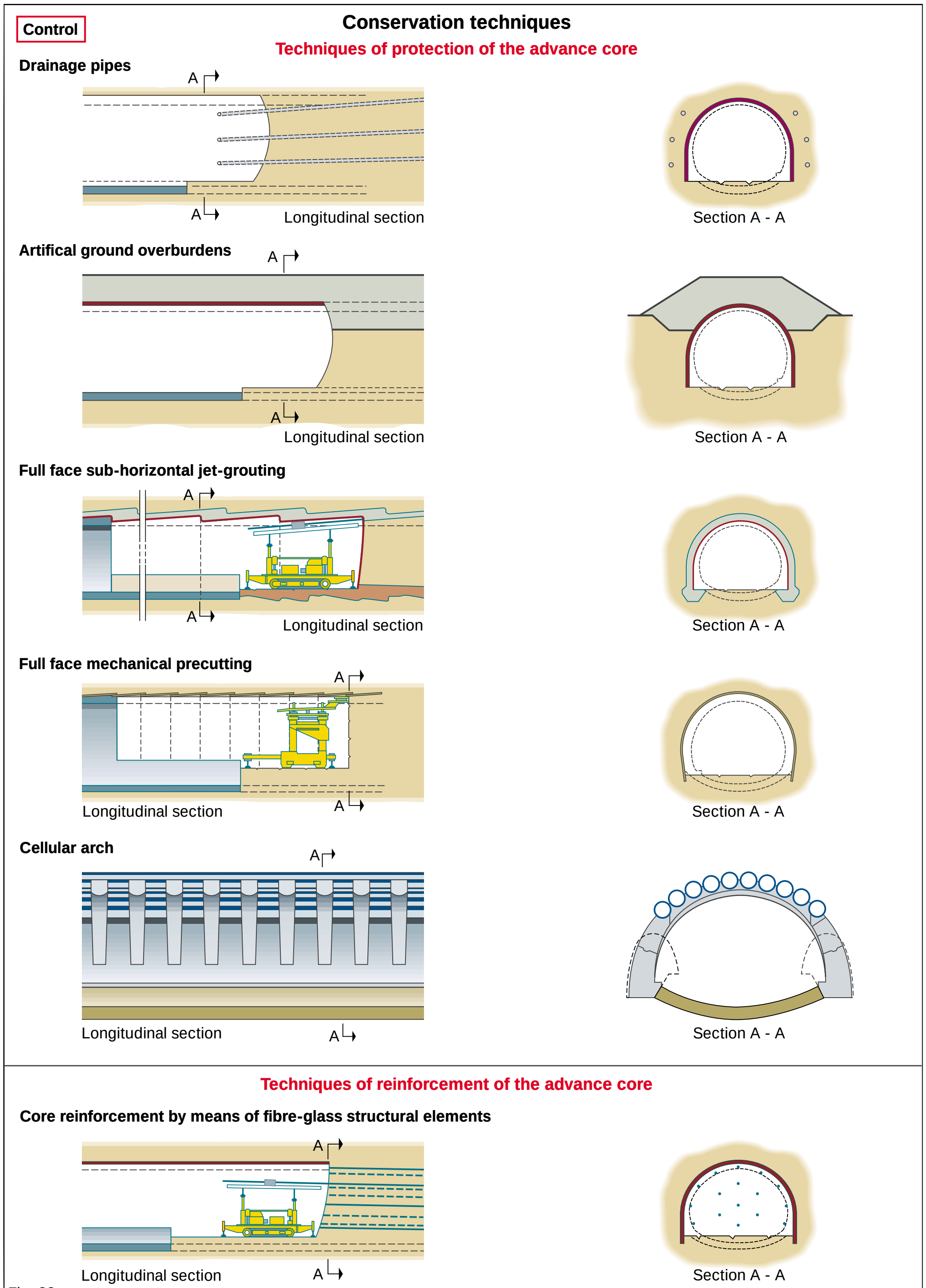


Fig. 22

6. PROPOSAL OF A NEW APPROACH

It seemed to us reasonable to state that the following is fundamental to the proper design and construction of an underground work:

at the design moment:

- to have a detailed knowledge of the medium in which one is to work, with particular regard to its strength and deformation properties;
- to make a preliminary study of the stress-strain behaviour (deformation response) of the medium to be excavated in the absence of stabilisation measures;
- to define the type of confinement or preconfinement action needed to regulate and control the deformation response of the medium to excavation;
- to select the type of stabilisation from those currently available among the existing technologies on the basis of the preconfinement or confinement action that they are able to provide;
- to design the longitudinal and cross section types on the basis of the forecast response of the medium to excavation, defining not only the most adequate stabilisation measures for the context in which it is expected to operate, but also the stages, paces and timing in which they are to be implemented;
- to use mathematical means to determine the entity of and to test the stabilisation measures that were selected in order to obtain the desired response of the medium to excavation and the necessary safety coefficient for the work;

at the construction moment:

- to verify that the response of the medium to excavation during construction corresponds to that which was predicted from studies at the design moment. Proceed then to fine tuning of the design balancing the intensity of the stabilisation measures between the face and the perimeter of the cavity.

It ensues from this that the design and construction of an underground work must necessarily be sequenced in the following chronological order:

1. a survey phase, for obtaining geological, geomechanical and hydrogeological knowledge of the medium;
2. a diagnosis phase, involving forecasting, by theoretical means, of the behaviour of the medium in terms of the deformation response in the absence of stabilisation measures;
3. a therapy phase, involving first, definition of the method of excavation and stabilisation of the medium employed to regulate the deformation response and then assessment using theoretical means of the effectiveness in this respect of the method chosen;
4. a monitoring phase, involving monitoring and experimenting with the actual response of the medium to excavation in terms of deformation response in order to fine-tune the excavation and stabilisation methods employed.

6.1 CONCEPTUAL FRAMEWORK ACCORDING TO THE A.DE.CO.-RS APPROACH

The approach based on the A.DE.CO.-RS differs in various important respects, some already mentioned in the previous parts of this report, from other methods used to date as a framework of reference:

1) the design and the construction of a tunnel are no longer seen as they were in the past but now represent two quite distinct moments with a clear and well defined physiognomy in terms of timing and practices;

2) the approach employs a new type of conceptual framework for underground works based on one single parameter common to all excavations: the stress-strain behaviour of the face-advance core system;

3) the approach is based on the prediction, monitoring and interpretation of the deformation response of the rock mass to excavation and this becomes the only reference parameter employed. First it is theoretically predicted and regulated and then it is experimentally measured, interpreted and experimented with as a means of fine-tuning the design of the construction that is being built;

4) the concept of preconfinement is introduced to complete the already well known concept of confinement; this addition makes it possible to drive tunnels in an orderly programmed fashion even under the most difficult statics conditions without having to resort to improvisation during construction;

5) the approach involves the use of conservation systems aimed at maintaining the geotechnical and structural properties of the ground, seen as the

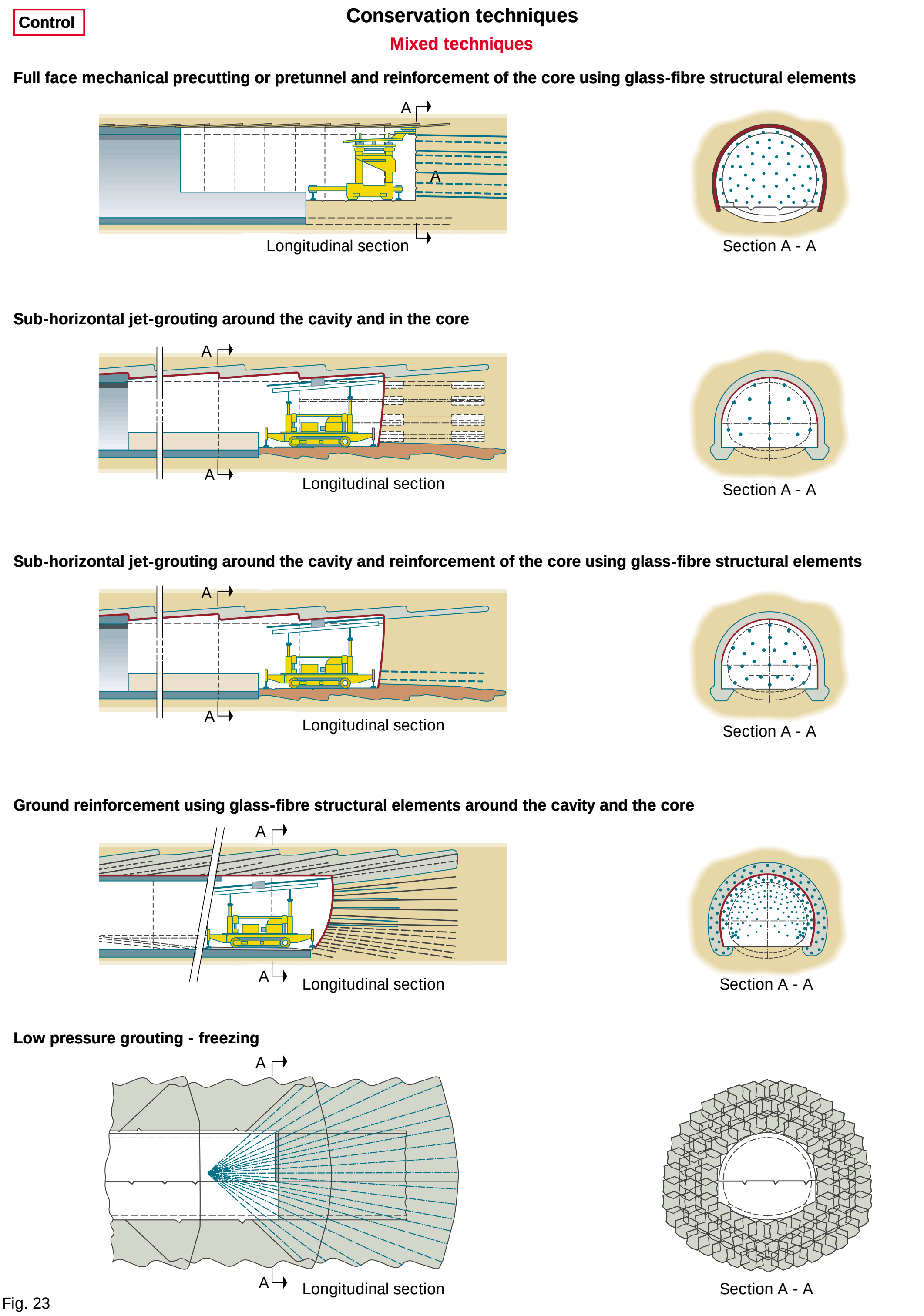


Fig. 23

"construction material", as unaltered as possible when these play a fundamental role in the speed and rhythm of tunnel advance.

As has been already mentioned, a particular characteristic of the approach is its introduction of a new conceptual framework for viewing underground works.

It has been observed that deformation of the ground during excavation and therefore the stability of the tunnel itself are dependent on the behaviour of the core of ground ahead of the face (advance core). Given this observation, the stability of the face-advance core system is employed as a basic element in the new conceptual framework. Consequently, by making reference to one single parameter valid for all types of ground (the deformation response of the advance core), the method overcomes the limitations of systems used until now, especially when soils with poor consistency are encountered.

As already illustrated, three fundamental behaviour categories can be identified *(fig. 14)*:

Category A: stable face, stony type behaviour;

Category B: face stable in the short term, cohesive type behaviour;

Category C: unstable face, loose ground type behaviour.

CATEGORY A

Category A is identified when the state of stress in the ground at the face and around the excavation is not sufficient to overcome the strength properties of the medium. The closer the excavated cross section of the tunnel is to the theoretical profile, the closer the "arch effect" will be to the walls of the tunnel.

Deformation phenomena develop in the elastic range, occur immediately and are measurable in centimetres.

The face as a whole is stable. Local instability only occurs due to the fall of isolated blocks caused by an unfavourable configurations of the rock mass. In this context a fundamental role is played by the anisotropic stress-strain state of the ground.

The stability of the tunnel is not affected by the presence of water even under hydrodynamic conditions unless the strength properties of the ground are mechanically or chemically affected by the water or unless the hydrodynamic gradient is so intense that washing away destroys the shear strength along the slip surfaces.

Stabilisation techniques are mainly employed to prevent deterioration of the rock and to maintain the profile of the excavation.

CATEGORY B

Category B is identified when the state of stress in the ground at the face and around the cavity during excavation is sufficient to overcome the strength of the ground in the elastic range.

An "arch effect" is not formed immediately around the excavation, but at a distance from it that depends on the size of the band of ground that is subject to plasticisation.

The deformation that occurs at normal advance rates is in the elastic-plastic range, is deferred and measurable in centimetres.

At normal advance rates the tunnel is stable in the short term and stability improves or worsens as advance speeds increase or decrease. Deformation of the advance core in the form of extrusion does not affect the stability of the tunnel because the ground is still able to muster sufficient residual strength.

Instability manifesting in the form of loose material breaking away is widespread at the face and around the cavity but allows sufficient time to employ traditional radial confinement measures after the passage of the face.

In some cases it may be necessary to resort to preconfinement of the face, balancing stabilisation measures between the face and the cavity so as to contain deformation within acceptable limits.

The presence of water, especially under hydrodynamic conditions reduces the shear strength of the ground, favours the extension of plasticisation and therefore increases the entity of instability phenomena. It must therefore be prevented, especially near the face, by channelling the water away from the advance core.

CATEGORY C

Category C is identified when the state of stress in the ground is considerably greater than the strength properties of the material even in the zone around the face. An "arch effect" can be formed neither at the face nor around the excavation because the ground does not possess sufficient residual strength. The deformation is unacceptable because it

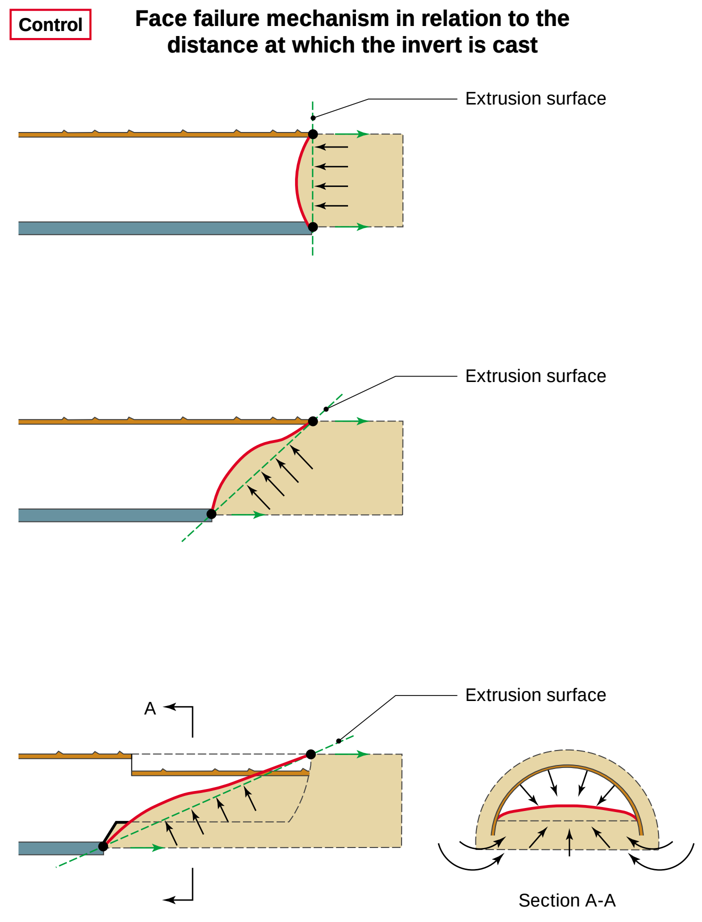


Fig. 24

develops immediately into the failure range giving rise to serious manifestations of instability such as failure of the face and collapse of the cavity without allowing time for radial confinement operations: ground improvement operations must be launched ahead of the face to develop preconfinement action capable of creating an artificial arch effect.

If due account is not taken of the presence of water under hydrostatic conditions, this favours the extension of plasticisation by further reducing the strength properties of the ground and basically increases the entity of deformation. Under hydrodynamic conditions it translates into the transport of material and siphoning which is absolutely unacceptable. It must therefore be prevented, especially near the face, by channelling the water out from the advance core.

It has been observed over twenty years of tunnel design and construction that all cases of underground works fall into one of these three behaviour categories.

6.2 MOMENTS AND PHASES OF THE A.DE.CO.-RS APPROACH

The approach based on the Analysis of COntrolled DEformation in Rocks and Soils suggests that in the logical development of the design and construction of a tunnel one should proceed according to the stages summarised in *fig 27*.

The design stage or moment consists of the following:

- the survey phase: during this phase the design engineer determines the nature of the ground affected by the tunnel in terms of rock and soil mechanics. This is indispensable for an analysis of existing natural equilibriums and for the success of the subsequent diagnosis phase;
- the diagnosis phase: during this phase the design engineer uses the information collected during the survey phase to divide the tunnel into sections with uniform stress-strain behaviour according to the category, A, B, or C, as described above, on the basis of the stress-strain behaviour that is expected. Details of the development of deformation and the types of loads activated by excavation are defined;
- the therapy phase: during this phase, the design engineer decides, on the basis of predictions made during the diagnosis phase, which type of action (preconfinement or simple confinement) to employ and the necessary means of implementing it to achieve complete stabilisation of the tunnel, naturally with reference to the three behaviour categories, A, B and C. He designs, therefore, the composition of the typical longitudinal and cross sections and verifies their effectiveness

Control

Calculation of the role of the invert in limiting deformation

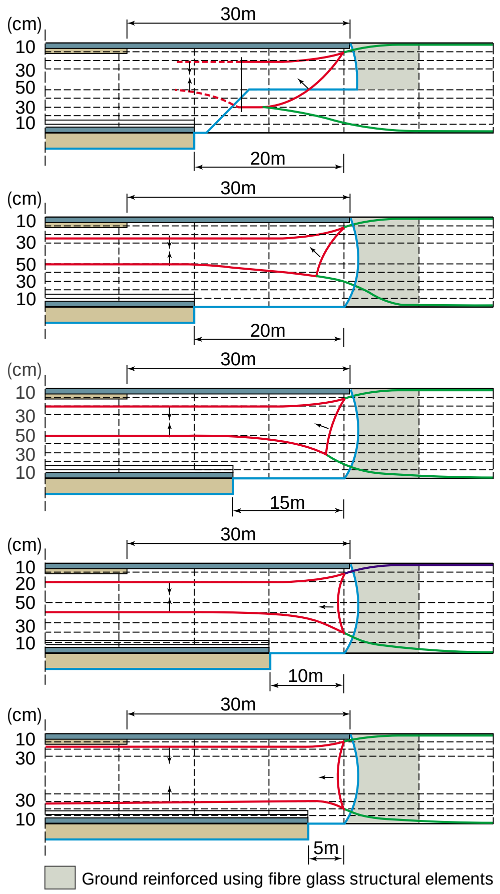


Fig. 25

Moment	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 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 face and the perimeter of the cavity

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

Fig. 27

using mathematical instruments.

The construction stage or moment consists of the following:

- the operational phase: during this phase the works for the stabilisation of the tunnel are carried out according to the design predictions. The various means employed are adapted in terms of confinement and preconfinement according to the actual deformation response of the rock mass and they are checked according to a quality control programme prepared in advance;
- the monitoring phase: during this phase deformation phenomena (which constitute the response of the medium to tunnel advance) is measured and interpreted firstly to check, during construction, the accuracy of the predictions made during the diagnosis and therapy phases and then consequently to add the final touches to the design in terms of the balance of stabilisation techniques between the face and the cavity. The monitoring phase does not end when the tunnel is completed, but continues during its whole life for constant verification of safety conditions.

Correct design of underground works, then, means knowing, on the basis of the original naturally existing equilibriums, how to predict the behaviour of the ground during excavation in terms of the triggering and development of deformation phenomena. Subsequently the predictions can be used to select the construction methods best suited to maintain these phenomena within acceptable limits and to establish construction times and pace as a function of tunnel advance and the position of the face.

Correct construction of an underground work, on the other hand, means working according to design decisions: first of all attentive reading of the response of the ground to the action of excavation and stabilisation operations in terms of extrusion and convergence of the face and the walls of the tunnel on the surface of the cavity and in the ground at distance from it; secondly, once the results of the various measurements have been interpreted, deciding actual length, speed and rhythm of advance, intensity, location and timing of stabilisation operations and the balance of these between the face and the perimeter of the excavation.

Rome to Florence railway line - Tasso tunnel

Chronology of observed deformation in a collapse during half-face advance

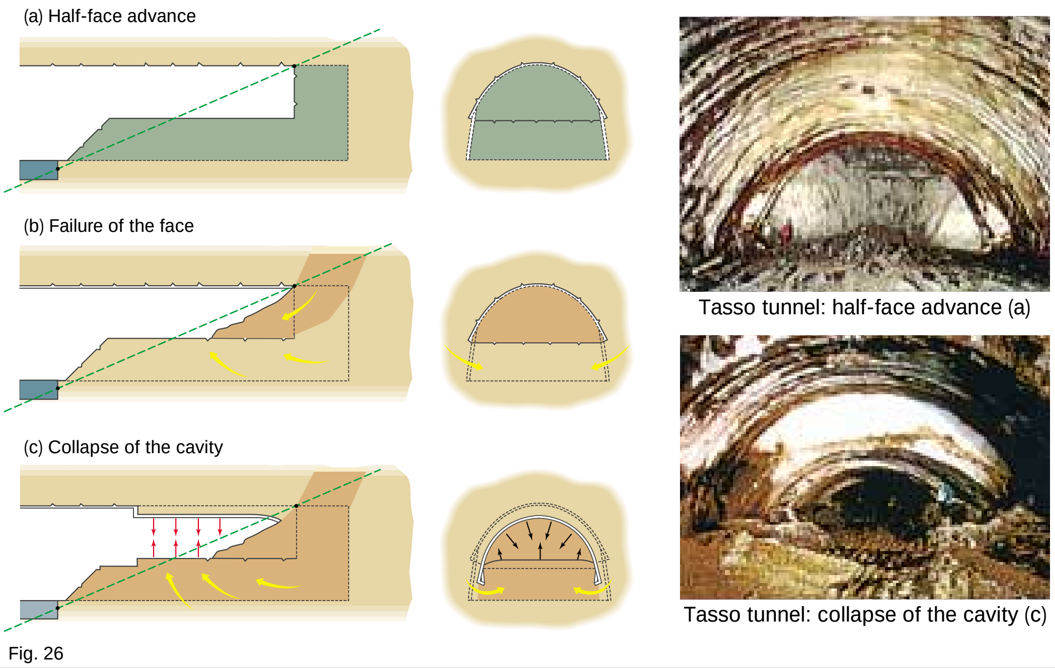


Fig. 26

6.2.1 THE SURVEY PHASE

Performing excavation underground means disturbing naturally existing equilibriums. Designing such an excavation with minimum disturbance to the medium in which one is working and therefore with a minimum deformation response means having, in advance, the fullest possible knowledge of the natural state of equilibrium of the ground before excavation begins.

Hence the design and construction of a tunnel must be preceded by a survey phase during which the nature of the medium is ascertained by acquiring information on the ground affected by the works, on its lithology, structure, stratigraphy, morphology, tectonics, hydrology, geotechnics, geomechanics and stress states, all of which is indispensable to the design engineer for analysing existing natural equilibriums and for carrying out the subsequent "diagnosis" phase correctly.

The survey phase consists of two consecutive stages.

The first stage involves a first estimate of the geological profile of the ground along the line of the route, developed on the basis of a 1:1,000,000 scale (circa) geological map, the existing literature and photogrammetrical surveys, all integrated with information from surface surveys including:

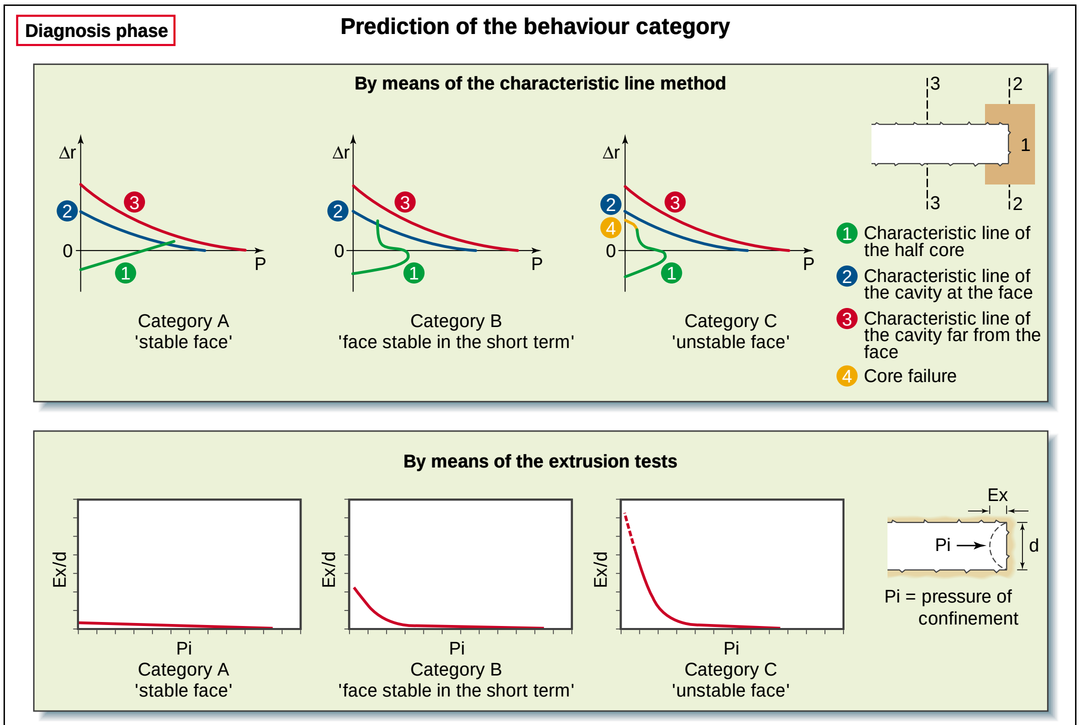


Fig. 28

- lithological surveys, with individuation of the main units;
- geomorphological surveys, with particular attention given to the stability of slopes;
- geological structure surveys, with the identification of the main lines of discontinuity;
- hydrogeological surveys, identification of the main hydrological system and a survey of the springs. The flow rates of the latter must be determined and monitored during construction to establish the drainage effect of the cavity on them.

The first estimate of the geological profile is accompanied by a series of lithological sheets giving information on the lithotypes that outcrop along the route and a summary of the surveys carried out.

Should the first stage of the survey phase result in the decision to construct a pilot tunnel, the final design may make advantageous use of geological and geomechanical survey information obtained from this tunnel [8], [9] and also of in situ tests designed to assess the strength and deformation properties of the rock mass.

The second stage is based on the results of the first stage. It involves the planning of the geological surveys

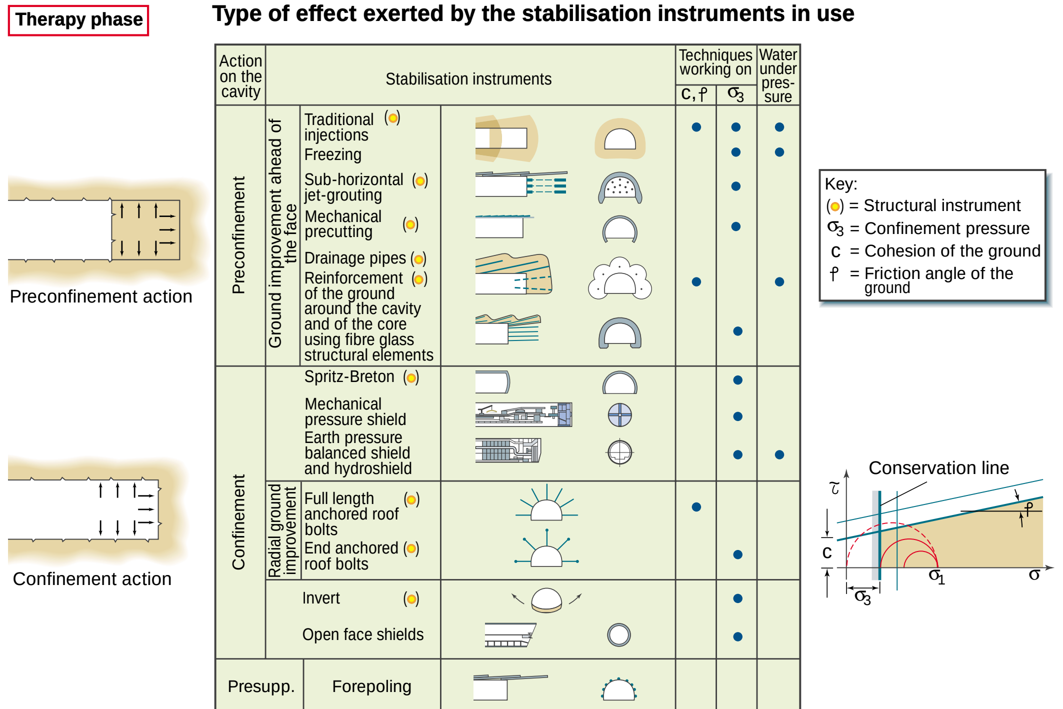


Fig. 29

including the definition of the indirect geophysical surveys, in situ tests and bore hole surveys with continuous core sampling for fine definition of the geotechnical parameters and recovery of undisturbed samples from the ground where the tunnel will pass.

For undisturbed samples it is indispensable that equipment designed to cause the least possible disturbance to the rock mass is used.

The samples obtained are used to assess the physical and chemical properties of the rock mass and also as these change over time and to assess geotechnical and geomechanical parameters.

The following are calculated:

- the intrinsic curve of the matrix of samples;
- the deformation parameters of the matrix of samples (initial elastic modulus and total deformation modulus calculated for levels of stress comparable to those that will develop following construction of the tunnel).

Where possible, it is important that the strength and deformation properties of structural discontinuities are calculated and that intrinsic curves and deformation parameters are derived from these on the basis of a detailed consideration.

The second stage is completed with an estimate of natural stress states based on the size of the overburden and the main tectonic structures involved.

It may be very advisable, according to the size of the tunnel to be constructed and the complexity of the tectonic structures involved, to carry out tests when possible to measure the natural stresses at the level of the cavity.

6.2.2 THE DIAGNOSIS PHASE

During the diagnosis phase the designer uses the information collected during the survey phase to divide the tunnel into sections with uniform stress-strain behaviour according to the categories, A, B, or C (stable face, face stable in the short term, unstable face). In order to do this he makes predictions, using theoretical methods, of the deformation response of the medium to the action of excavation, paying particular attention to deformation phenomena which, in the absence of stabilisation intervention, would manifest at the face and consequently in the band of ground around the cavity.

The analysis of <u>the deformation response of the face-advance core system and of the cavity</u> is conducted in terms of origin, localisation, development and size of probable deformation phenomena. Resort is made to mathematical tools such as characteristic lines, two and three dimensional finite elements models and so on which are capable, depending on the reliability of the geotechnical and geomechanical input data, of orienting the design engineer in deciding which of the three behaviour categories, A, B or C, to assign to a given section of tunnel.

Of these tools the characteristic line method [5], which can at present be used in most situations, seems to be particularly simple and easy to use for this purpose *(fig. 28)*.

Of the experimental methods available, for certain types of ground, extrusion tests on undisturbed samples in tri-axial cells can be used to simulate tunnel advance in the laboratory under different overburdens as well as changes in the stress state in the face-advance core system induced by the action of excavation to show how it behaves *(fig. 28)*.

The result of the analysis finally takes on concrete form in a longitudinal profile of the tunnel showing the division into sections with uniform stress-strain behaviour and the behaviour category (A, B, C) associated with each section.

Once it has been decided which of the three behaviour categories each individual section belongs to, it is also part of the diagnosis phase to do identify the following within each category:

a) the <u>types of deformation</u> that will develop around the cavity (extrusion, preconvergence and simple convergence);

b) the consequent and expected <u>manifestations of instability</u>, for example:

- fall of ground and spalling of strata at the face produced by advance core extrusion and preconvergence;
- fall of ground and spalling of strata around the cavity produced by convergence of the cavity;
- collapse of the cavity produced by failure of the face.

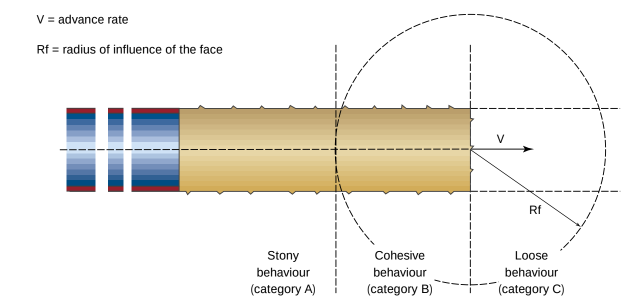
c) <u>loads mobilised by excavation</u> according to overarching weights and plasticised ring models.

6.2.3 THE THERAPY PHASE

During the theraphy phase the design engineer decides, on the basis of the behaviour categories assigned during the diagnosis phase, which type of action (preconfinement, confinement or presupport) to employ in order to achieve complete stabilisation of the tunnel (regulation of deformation phenomena).

From what was said previously concerning the importance of the rigidity of

Therapy phase — **Choice of stabilisation instruments for the composition of cross section types**



Face		Stable		Stable in the short term			Unstable				
		A_1	A_2	B_1	B_2	B_3	C_1	C_2	C_3	C_4	C_5
Stabilisation instrument	Radial roof bolts	•		•							
	Reinforced shotcrete *	•	•	•	•		•	•	•	•	•
	Reinforcement of the core using fibre glass structural elements				•			•		•	
	Reinforcement of the ground around the cavity using fibre glass structural elements									•	
	Tunnel invert			•	•	•	•	•	•	•	•
	Mechanical precutting					•		•			
	Radial ground reinforcement from pilot tunnel								•		
	Sub-horizontal jet-grouting						•				
	Injection in advance or freezing										•
	Drainage pipes	•	•	•	•	•	•	•		•	
	Forepoling		•								
Deformation phenomena		Centimetric		Decimetric			Unacceptable				

*Reinforcement by steel ribs and/or mesh and/or fibres

Fig. 30

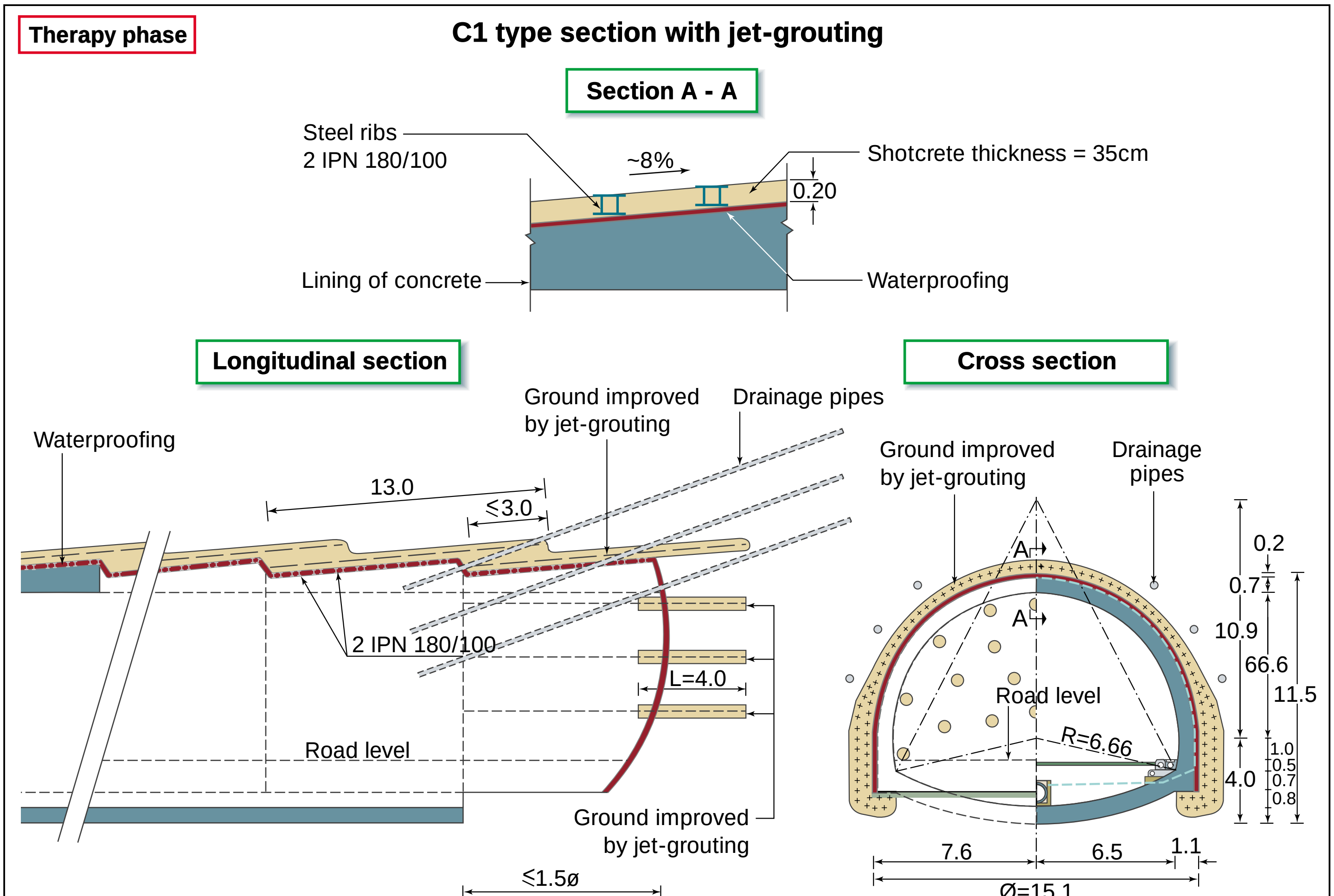


Fig. 31

the core of ground ahead of the face with regard to the stress-strain behaviour of the face and the cavity and therefore to the stability of the whole tunnel, a designer has three basic courses of action open to him:

- he may limit himself to simple confinement action in the case of tunnels with stable face stress-strain behaviour (category A);
- he has to think in terms of producing vigorous preconfinement action - in addition of course to confinement action - in the case of tunnels with unstable face stress-strain behaviour (category C);
- he may choose between preconfinement or simple confinement of the cavity as a function of the rate of advance he estimates he can achieve in the case of tunnels with the face stable in the short term (category B).

Once the type of action to be exerted has been decided, the action must be perfected in terms of systems, rates, excavation stages and above all stabilisation methods and tools. For the latter, how and where they are to be employed with respect to the position of the face and according to the behaviour category, A, B or C, involved, must be established so that the desired action is produced.

In order to actually produce the type of action desired in practice, a design engineer has a number of tools available with which to implement all the necessary types of stabilisation.

Keeping in mind that stabilisation may be of the following types:

- conservation when the main effect is that of containing the relaxation of the minor principal stress;
- improvement when the main action is that of increasing the shear strength of the medium;

of those instruments available to the design engineer **that produce preconfinement action on the cavity** [1] *(fig. 29)*, the following exert an essentially conservation effect:

- fibre reinforced shotcrete shells created by means of mechanical precutting along the profile of the tunnel using the precutting itself as formwork [3], [10];
- reinforcement of the core ahead of the face to a depth not less than the diameter of the tunnel by means of fibre glass tubes or structural elements fixed to the ground with cement mortar; the entity of this operation will depend on how much the shear strength of the core is to be increased [3], [10], [11], [12], [13];
- truncated cone umbrellas, consisting of subhorizontal columns of ground, side by side, improved by jet-grouting [10], [14].

A mainly improvement action is exerted by:

- truncated cone umbrellas of ground improved by traditional injections or freezing;
- truncated cone drainage umbrellas, when working under the water table.

Of those instruments available to the design engineer **that produce confinement action on the cavity**, the following exert a mainly conservation effect:

- a primary lining shotcrete shell capable of producing confinement pressure as a function of its thickness;
- full-face mechanised excavation by means of pressurised shields, capable of producing confinement pressure on the face and on the cavity (lining ring in prefabbricated concrete segments);
- mechanised excavation by means of open shields, that furnish radial confinement pressure on the ground during excavation;
- radial roof bolting performed by means of end anchored bolts, which apply "active" confinement pressure on the walls of the tunnel, to an extent that is preset by the tensioning of the bolts;
- invert, which creates a closed lining structure and multiplies the capacity of the primary and secondary linings to produce very high levels of confinement pressure around the cavity.

A mainly improvement action is exerted by:

- a ring of reinforced ground around the cavity, created by means of roof bolts anchored along their full length capable of increasing the shear strength of the ground involved and raising its intrinsic curve.

Those tools which fall into neither of these two categories because they produce neither preconfinement nor confinement action are known as presupport or support methods according to whether they act ahead of the face or not. They have no effect on the formation of an "arch effect" and are not able to either contain in any appreciable manner the relaxation of the minor principal stress or to improve the shear strength of the ground.

An example of one of these presupport methods is forepoles which, although constituting beams resting on ribs installed after excavation and forming a cylindrical configuration running along the line of the tunnel, are incapable of producing an arch effect due to the absence of any reciprocal transverse action between them.

6.2.3.1 COMPOSITION OF TYPICAL LONGITUDINAL AND CROSS SECTIONS

In the preceding paragraphs we have seen that the stability of face-advance core system plays a fundamental role in the deformation response of the medium to the opening of an underground cavity and consequently in the long and short term stability of the tunnel itself. We have also seen that the stability of this system can be described in terms of three fundamental behaviour categories that characterise the type of tunnel to be excavated for a given section in terms of a conceptual framework and that it is completely logical to refer to these categories when deciding on stabilisation methods designed to guarantee the stability and safety of the works.

Taking account of the above, figure 30 shows, schematically, the range of applicability of the individual stabilisation instruments available to a design engineer in the context of the proposed framework. The assembly of these will determine the typical longitudinal and cross sections designed to guarantee the feasibility of the excavations and the long and short term stability of the tunnel. The cases involved are as follows:

- in sections of tunnel with a stable face (behaviour category: A, stresses: in the elastic range, typical manifestations of instability: fall of ground), the methods of stabilisation proposed have above all a protective function and are determined by the geostructural configuration of the ground and by the possible presence of water;
- in sections of tunnel with a face stable in the short term (behaviour category: B, stresses: in the elastic-plastic range, typical manifestations of instability: spalling of strata due to extrusion of the core, preconvergence and convergence of the cavity), the methods of stabilisation must guarantee the formation of an arch effect as close as possible to the profile of the excavation. Instruments are therefore proposed that are capable of preventing the ground from losing its strength and deformation properties with particular reference to the face-advance core system. Confinement or preconfinement action is developed that is sufficient to counter the onset of plasticisation of the ground or at least limit the extent of this;
- in sections of tunnel with an unstable face (behaviour category: C, stresses: in the failure range, typical manifestations of instability: failure of the face, collapse of the cavity), the methods of stabilisation must guarantee the formation of an artificial arch effect in advance ahead of the face. Instruments for preconfinement of the cavity are therefore proposed which, by ensuring the stability of the face-advance core system, prevent the minor principal stress (σ_3 from falling to zero, de facto, when deformation phenomena can still be controlled.

The table in *fig. 30* can therefore be used by the designer as a reference for the design of typical longitudinal and cross sections.

Fig. 31 shows an example of the composition of C1 type, longitudinal and cross sections.

6.2.3.2 DESIGN AND ANALYSIS OF TYPICAL LONGITUDINAL AND CROSS SECTIONS. SUMMARY OF THE THERAPY PHASE

Having decided the type of action to

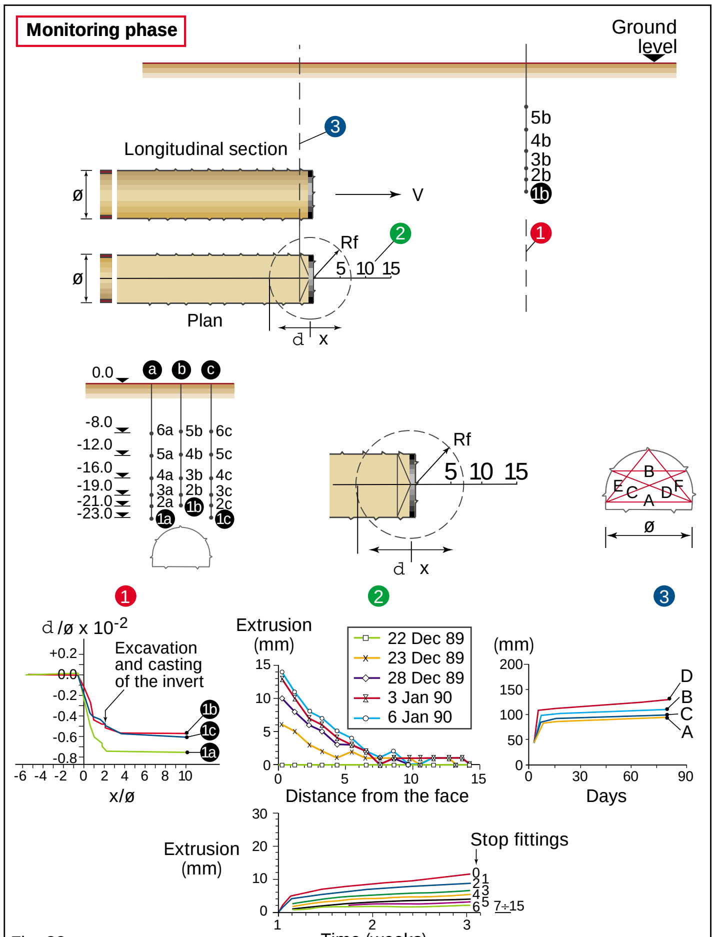


Fig. 32

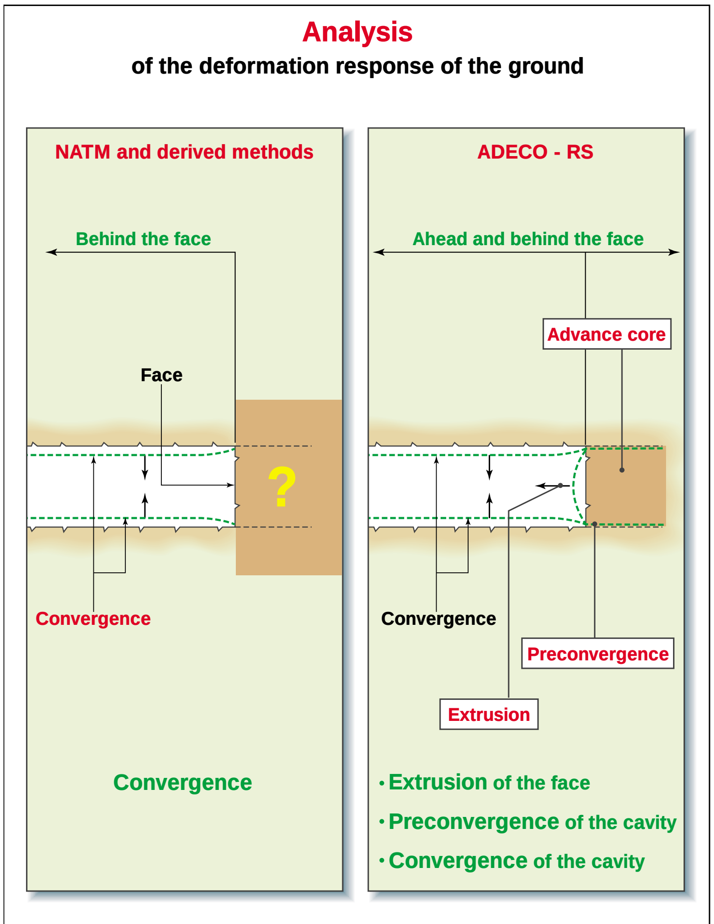


Fig. 33

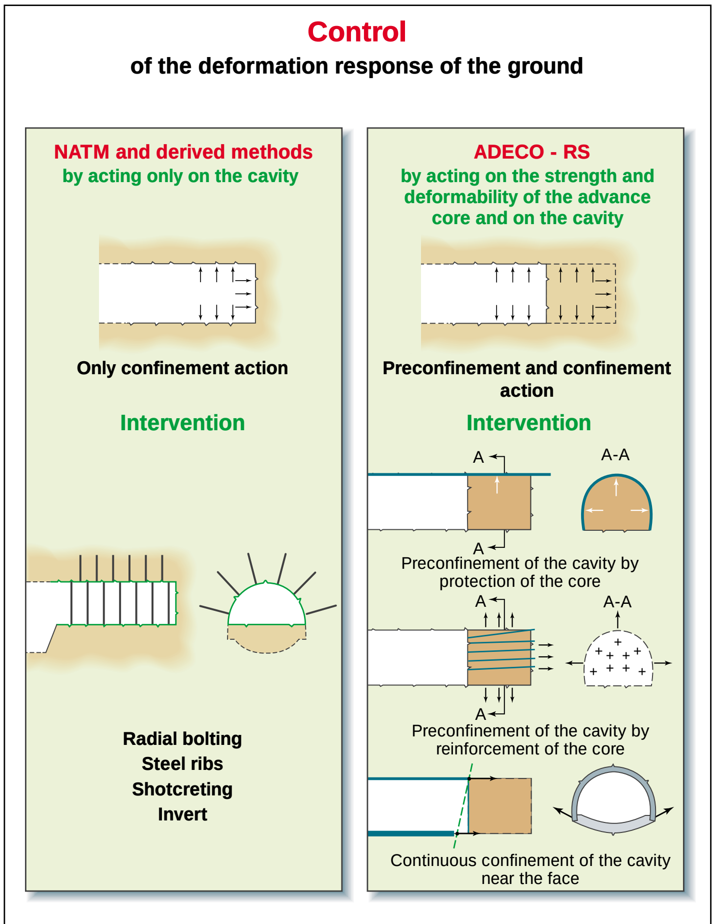


Fig. 34

exert, designed the methods to be employed to achieve it and composed the typical longitudinal and cross sections, the design engineer still has the task of setting the dimensions of the latter and analysing them using the mathematical methods already employed during the diagnosis phase. Analysing the balance of intervention between the face and the perimeter of the excavation is of particular importance as is assessing their effectiveness on the basis of the acceptability of the stress-strain behaviour predicted for the tunnel once stabilisation measures have been implemented. Naturally the calculations can be carried out resorting to simple convergence-confinement models or, on the other hand, to more complex extrusion-confinement or extrusion-preconfinement models according to the specific stress-strain situation that is assumed.

In addition to the main (or prevalent) design section types, some derived section types may also be designed for use in statistically probable situations, where, however, the precise location could not be predicted on the basis of the available data.

The prevalent and derived design sections are in any case defined unequivocally in the sense that for each type not only are the type, intensity, stages and sequence of the construction operations described in detail, but the geological and geomechanical conditions in which the design section must be placed are also clearly identified.

The introduction of derived design sections now allows tunnels to be constructed in Quality Assurance regime, conforming to ISO 9000 rules [15].

The result of the therapy study is then summarised on the geomechanical profile of the tunnel showing the typical longitudinal and cross sections to be adopted for each section of the tunnel having uniform stress-strain behaviour.

6.2.3.3 THE ADVANCE CORE UNDER THE WATER TABLE

As is known, water under hydrostatic conditions, and even more so under hydrodynamic conditions reduces the strength and deformation properties of the ground considerably. It is also known that a tunnel that advances under the water table acts like a huge drain: a movement is set up whereby water filters towards the face which affects the advance core first. Since as we have seen the advance core plays a key role in determining the short and long term stability of a tunnel, it is important that water is prevented from circulating inside the core. This can be achieved, depending on the specific conditions involved (feed to the water table, gradients, etc.) by acting systematically to water-proof the core and the walls of the cavity (advance under hydrostatic conditions) or by intercepting the water three or more tunnel diameters ahead of the face with special drainage pipes placed in an umbrella-shaped configuration around the future tunnel (advance under hydrodynamic conditions).

In the latter case, in order ensure that the treatment ahead of the face does not have the opposite effect to that desired, it is extremely important that great care is paid to the correct implementation of the drainage pipes. These must never be inserted into the ground from the surface of the face. They must always be inserted, in a truncated cone configuration, into the ground from the side walls of the tunnel, or at least from the perimeter of the face so that the core is never intersected. If this were not done, then the water drawn into them would soak into the advance core with disastrous effects on its stability and on that of the entire tunnel. To prevent this from happening, it is important to make sure that the drainage pipes are free from perforations for a few meters at the end closest to the tunnel.

Similarly, for the same reason, great care must be taken to ensure that ground improvement treatment involving the drilling and then the insertion of reinforcement elements into the advance core is performed correctly. It is important that the holes are drilled one at a time and that they are immediately filled and perfectly sealed with cement mortar. This is the only way to prevent the holes bored from draining water into them with devastating consequences for the advance core which once water-logged and weakened, would not be able to perform its stabilising action effectively.

6.2.4 THE MONITORING PHASE

Once the design moment is complete, the start of construction work (construction moment) coincides with that of monitoring to check the reliability of predictions made during the diagnosis and therapy phases in terms of stress-strain behaviour.

This monitoring (which assumes great importance, having based the entire design on these predictions) is carried out by measuring and checking the real "response" of the medium to the action of excavation. The response manifests in the form of deformation phenomena:

- inside the cavity, at the face and on the walls of the excavation;
- on the surface, along the route of the tunnel.

To this end, appropriate measurement stations are installed ahead of, at and back from the face *(fig. 32)*.

In fact when it is predicted that tunnel advance will occur in short term stable or unstable face conditions, it is particularly interesting and advisable when the tunnel overburden permits, to install multibase vertical instruments for a given cross-section before the arrival of the face that are designed to measure radial deformation that precedes its arrival (preconvergences).

For the face-advance core system then, sliding longitudinal micrometers are used to measure extrusion and multibase radial rod extensometers for convergence both at the surface of the cavity and at varying distances from it, inside the mass of the ground. Special tape extensometers are used to check perimeter convergence back from the face.

The more systematic and thorough the monitoring is the more reliable and useful the information will be for the design engineer whose task will be more or less complex according to the range in which the deformation phenomena develop.

If tunnel advance occurs in a medium with stony or loose type behaviour (categories A or C respectively), where forecast deformation is so slight that it does not cause any worry (cases of lithoid ground under weak to medium overburdens) or so great as to be unacceptable and to require preconfinement intervention (non cohesive ground under any overburden, clayey and lithoid ground under large overburdens), the importance of monitoring is reduced because deformation phenomena will develop rapidly and be limited in entity. Consequently the task of the design engineer is much easier once appropriate decisions have been made for adequate control of the real situation.

On the other hand, however, the design engineer must pay much more attention and greater care must be taken with the analysis of deformation of the face-advance core system and of the convergence of the cavity both at the cavity surface and inside the ground at a distance from it. Their development over time and in space should be followed carefully when advance occurs in a medium with cohesive type behaviour (category B).

In this case, deformation phenomena will be slow, progressive, and deferred and of ever increasing entity and the design engineer will only be able to obtain the information necessary for optimising the intensity and distribution of intervention between the face and the cavity on the one hand and calibrating stages, rates and excavation systems on the other, by continuous interpretation of monitoring results.

Consequently, it cannot be emphasised too much just how important correct interpretation of the results of monitoring is, because the detailed fine-tuning of the design of the excavation under construction depends on correct interpretation of these results.

During construction, the results of monitoring will guide the design engineer and the director of work in deciding whether to proceed with the design section type as specified or modify construction quantities (according to criteria specified in the design) adopting a derived design section type allowed for the considered tunnel section in the design specifications or to proceed with the design of a new type of design section type in the presence of particular conditions not detected at the survey phase and therefore not provided for by the design.

It is also important to point out that the monitoring phase does not end when the tunnel is complete, but on the contrary must continue with systematic monitoring aimed at ensuring the safety of the tunnel during the whole of its service life.

7. CONCLUSIONS

If the deformation that is normally observed inside a tunnel while it is advancing is interpreted in terms of a process of cause and effect, it would seem perfectly reasonable to identify the cause as the action that is exerted on the medium and the effect as the deformation response of the medium, consequent to the action.

While on the basis of this assumption the cause until only a few years ago was not felt worthy of attention nor of in-depth analysis, remaining only apparently determined, the effect was immediately identified as convergence of the cavity and it is this that was studied *(fig. 33)*. These studies produced theories, design approaches and construction systems that assumed that all the problems connected with tunnel construction could be solved by the use of simple radial confinement action *(fig. 34)*.

Among the former, the "theory of characteristic lines" developed by Lombardi and the "convergence-confinement method" developed by Panet [4], [5] are very well known. Although they recognised for the first time the beneficial effect of the presence of the core on the stability of the cavity, they nevertheless did not furnish any effective suggestions on how to exploit the effect nor on how to tackle instability of the face.

Among the latter, approaches like the NATM, based on geomechanical classifications (often used for purposes other than that for which they were intended) undoubtedly constituted considerable progress with respect to the past at the time when they were introduced. The principal merits of the NATM were:

- to consider the ground as a construction material for the first time;
- to introduce the use of new technologies of simple active confinement of the cavity such as shotcrete and roof bolts;
- to underline the need to systematically measure and interpret the deformation response of the rock mass.

Today, however, having considered the statics problems of tunnels exclusively as two dimensional problems and concentrated all attention on convergence of the cavity alone, it (and all the approaches derived from it) shows the following important limitations:

- it is an incomplete and partial classification system, because it is not applicable to all types of ground and all stress-strain conditions;
- it completely overlooks the importance of the advance core and the need

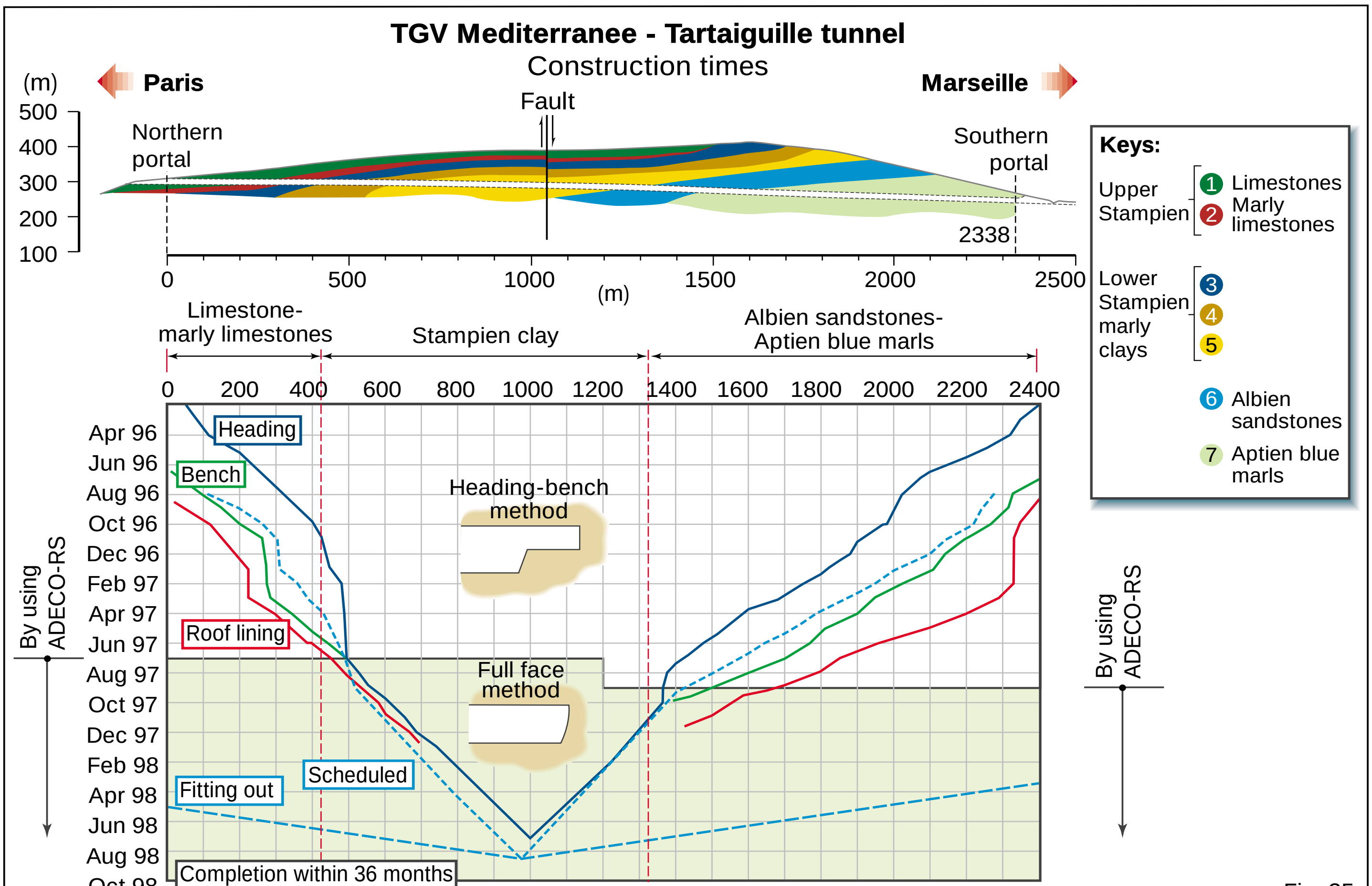


Fig. 35

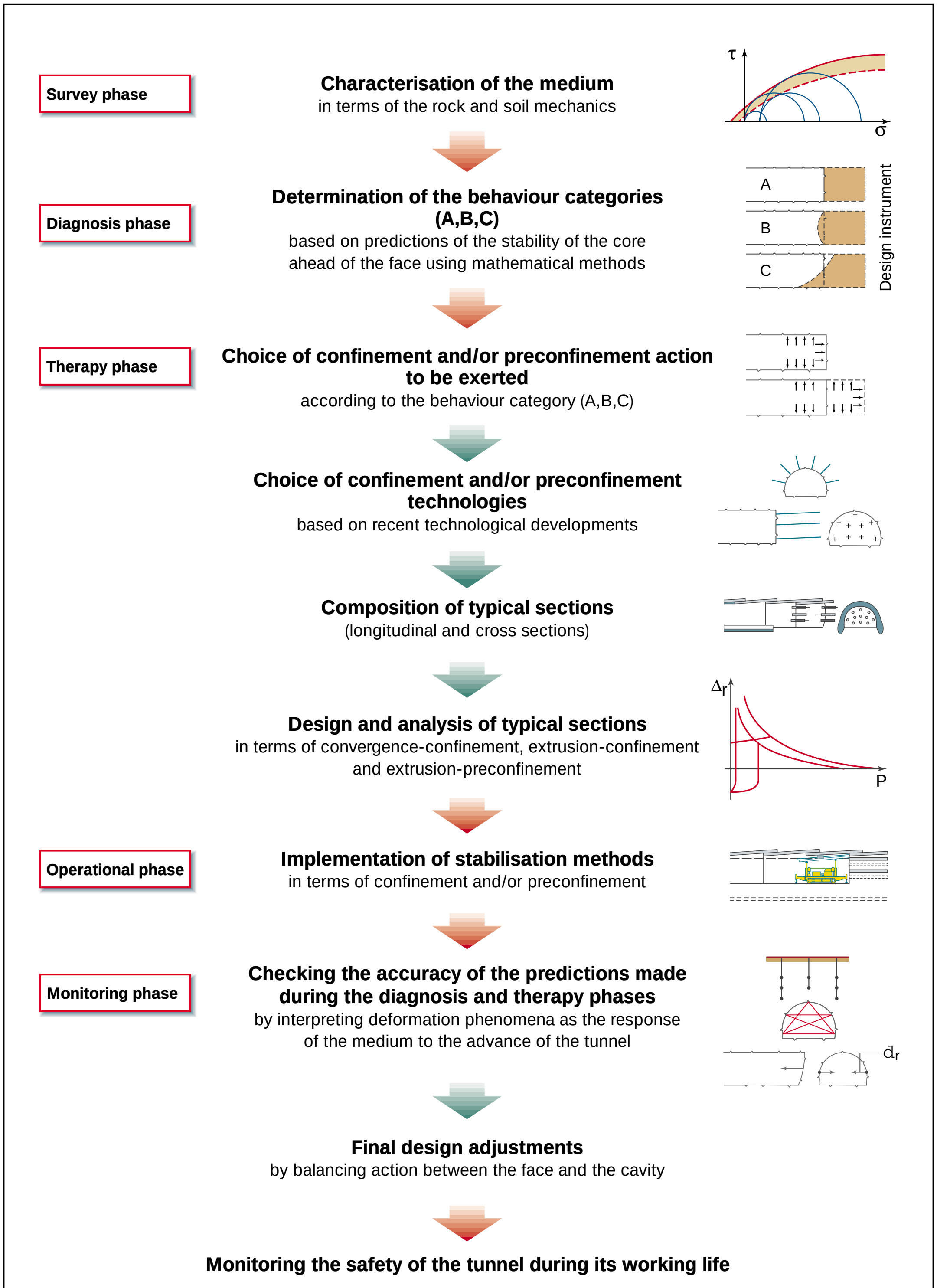


Fig. 36

to use it as a stabilisation instrument under difficult stress-strain conditions;

- it ignores new technologies, continuing to propose simple confinement techniques alone for the stabilisation of tunnels;
- it does not provide a clear distinction between the design moment and the construction moment of a tunnel;
- it solves the problem of monitoring the adequacy and correct dimensions of the design solutions adopted in an indisputably non scientific manner, quite happily comparing the geomechanical classes with the size of the deformation response of the ground.

The erroneous conviction that the effect consequent to the action that is exerted on the medium during tunnel excavation was identifiable only in the convergence of the cavity has led entire generations of engineers down the wrong path. What was taught by the design and construction approaches fashionable at the time (NATM and derived methods which still today act according to this erroneous conviction) induced them to concentrate on dealing with the effects (containing convergence of the cavity by simple confinement action) instead of on the causes of tunnel instability [16], [17].

This way of tackling the problem was successful for driving tunnels under low to medium difficulty stress-strain conditions but showed its limitations when faced with very or extremely difficult conditions because:

1. of the inability to make reliable forecasts of tunnel behaviour during advance and therefore of the absence of a diagnosis phase in the design procedure;
2. measures to confine deformation that was not forecast beforehand were improvised;
3. of a lack of effective stabilisation systems, capable of dealing with the cause of instability (deformation of the core) and not just the effect (convergence);
4. of the inability to make a preliminary assessment of a project in terms of forecasting risks, time schedules and advance rates.

Given this situation, the rapid and constantly growing demand for tunnels of all types, including those under high and extreme stress-strain conditions urgently required the formulation of theories and procedures capable of controlling the deformation response of the medium <u>under all possible stress-strain conditions and not just under “not difficult” conditions</u>.

In order to get out of this stalled situation it was necessary to bring the problem back to reality and treat it as a <u>three dimensional problem</u>, which in effect it is, and to consider the entire dynamics of the tunnel advance process and not just the last phase of the process.

It was with this philosophy that theoretical and experimental research studies commenced which provided the foundations for a new approach based on the Analysis of COntrolled DEformation in Rocks and Soils. It has been successfully employed over the last 10 - 15 years in an extremely wide range of types of ground and stress-strain conditions including the most difficult and has been used to solve problems in numerous extremely difficult situations *(see table 1, overleaf)* where the application of old concepts (NATM and derived methods), which don’t show their limitations or intrinsic defects in less difficult situations, had furnished disappointing and at times even catastrophic results.

In this respect, it is perhaps important to consider, in the conclusion, the events that occurred in France during the construction of the Tartaiguille tunnel for the new “TGV Méditerranèe”, Marseilles to Lyons high speed rail line.

Tunnel advance with a 180 sq. m. cross section, began in February 1996 and proceeded with varying success according to NATM principles until September of the same year when the heavily swelling “argile du Stampien” formation (75% montmorillonite) was encountered and increasing difficulties were such that continuation of the works was practically impossible. In order to solve the problem at the start of 1997 the SNCF (Societè National du Chemin de Fer) set up a study group (“Comité de Pilotage”) consisting of engineers from the French Railways, engineers from the G.I.E. Tartaiguille consortium, the consulting engineers Coyne et Bellier and CETU, consulting geotechnical engineers from the Terrasol and Simecsol consortium. This group in turn consulted major European tunnelling experts inviting them come up with a design solution to cross the clayey formation safely and on schedule.

After examining several proposals, none of which offered the guarantees of

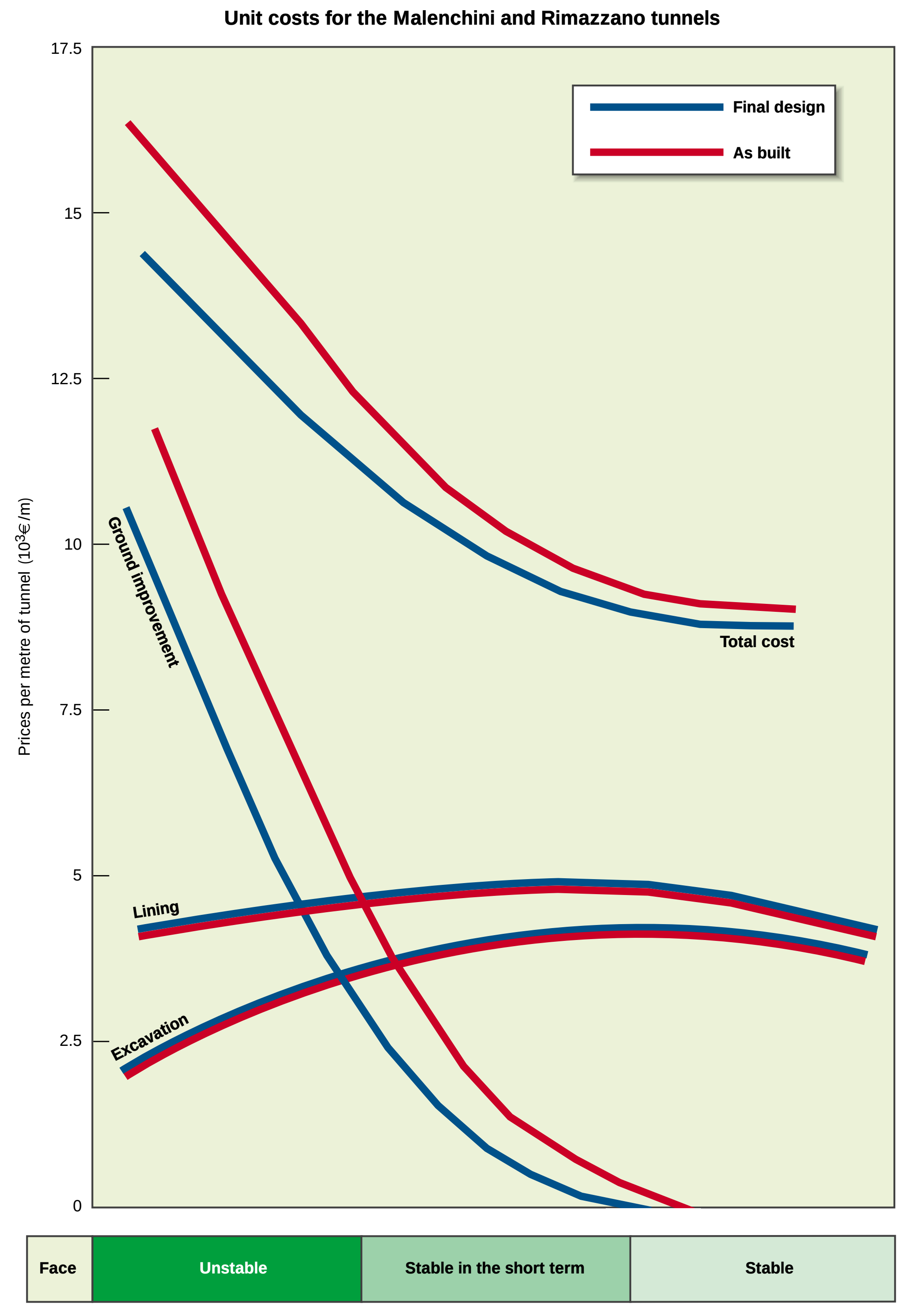


Fig. 37

safety and reliability requested by the client, above all with regard to completion times, the SNCF were attracted by the proposal put forward by myself containing hypothesised construction times and costs guaranteed on the basis of similar cases successfully solved. In March 1997, Rocksoil S.p.A. was awarded the contract for detailed design of the 860 m. of tunnel still to be completed.

Tunnel advance resumed in July 1997 after radical revision of the design according to A.DE.CO.-RS principles (full face tunnel advance, *see photo 4*) and was finally able to continue without interruptions and with growing success as the site operators gradually gained confidence in the use of the new technologies. Exceptionally constant average advance rates were recorded *(fig. 35)*, which as guaranteed by the consulting engineers were higher than 1.4 m. per day allowing the tunnel to be completed in July 1998 after only one year since work began with the new system and one month ahead of schedule [18], [19], [20].

In the light of the considerable experience acquired over the last ten years [3], [12], [13], [20], [21], it can be confidently stated that the A.DE.CO.-RS approach to the design and construction of tunnels can be used to produce virtually linear advance rates independently of the type of ground tunnelled and the contingent stress-strain conditions. It follows that while it was once only possible to talk of mechanisation under conditions that could be dealt with by simple confinement of the cavity or of the face (shields, TBMs), today mechanisation can be spoken of even under more complex and difficult conditions which require preconfinement action. Tunnel excavation can finally be industrialised (constant advance rates, accurate forecasting of times and costs) independently of the type of ground and the size of the overburden involved.

To summarise, by making full use of the most recent knowledge, calculating power and advance technologies *(fig. 36)* the A.DE.CO.-RS approach offers design and construction engineers a simple guide with which tunnels can be classified in one of three fundamental behaviour categories. To do this, reference is made to the stability of the face-advance core system which is predicted by means of in-depth stress-strain analysis performed theoretically using mathematical tools. For each section of tunnel with a uniform deformation behaviour identified in this manner, the design engineer decides the type of action (preconfinement or simple confinement) to produce in order to control deformation and, as a consequence, selects the type of stabilisation technique and the longitudinal and transverse tunnel section type most appropriate to each given situation, by using the suited instruments to develop the necessary actions. Tunnel section types are available for all types of ground and stress-strain condition. Costs and construction times (per linear metre) can be automatically calculated for each of these.

By using this approach:

- importance is given to stabilisation techniques as indispensable instruments for controlling and regulating deformation, and hence as “structural elements” for the purposes of the final stability of a tunnel (tunnels are seen in terms of, and paid for in proportion to, how much they deform). In this respect it is worth noting that on the cost side of income statements for underground projects, stabilisation and ground improvement works are the only items which vary significantly as compared to excavation and lining items which increasingly tend to remain constant for all types of ground *(fig. 37)*;
- with a complete and reliable design the main contractor is induced to industrialise tunnel advance operations in all types of ground, even the most difficult;
- given the ability to plan construction times and costs, disputes which until very recently arose between the Clerk of Works and contractors are avoided;
- by employing one single reference parameter, common to all types of ground (the stress-strain behaviour of the face-advance core system), that can be easily and objectively measured during tunnel advance, the problem of the clearest and most evident defect of previous classification systems (comparing geomechanical classes with the deformation response of the ground), which until today fuelled disputes between Clerks of Works and contractors, is solved.

As a result of these important characteristics, the A.DE.CO.-RS approach has aroused considerable interest and rapidly established itself as an advantageous alternative to those employed to date. In this respect the decision to use it for drawing up the design specifications on the basis of which contracts were awarded and then the construction design of the new Bologna to Florence High Speed Rail line was particularly significant. At the moment it certainly constitutes the largest tunnel construction project in the world: approximately 84.5 km. of tunnel with a cross section of 140 sq. m. on a route with a total length of 90 km through notoriously difficult ground due to its variability and often very poor geomechanical properties. Despite the difficult conditions, contracts for the works were awarded on a "turnkey" basis in which the contractor evidently felt that the design was sufficiently complete and reliable to accept all the risks, including geological risks. At present more that 30% of the works, which began in July 1998, are now complete and full face advance *(see photo 5)* is proceeding simultaneously on 32 faces at an average advance rate of approximately 1,600 m./month of finished tunnel [21] [22].

While the art of designing and constructing tunnels has perhaps lost some of its fascination with the demand to plan, it has certainly gained with the introduction of the A.DE.CO.-RS approach, in efficiency and functionality, while neither restricting nor conditioning the imagination. ■

Photo 4: Tartaiguille Tunnel (France, Marseille to Lyon TGV Méditerranèe, ground: swelling clay (75% montmorillonite), overburden: 100 m, tunnel diameter: 15 m.). View of the face (180 m^2) reinforced with fibre-glass structural elements.

Photo 5: Pianoro Tunnel (Italy, Bologna to Florence High Speed Rail Line, ground: cemented silty sand, overburden: 150 m, tunnel diameter: 13.30 m.). View of the face reinforced with fibre-glass structural elements.

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Table 1st. The application of design and construction criteria stated by ADECO-RS approach enabled us to achieve considerable successes, including the salvage of tunnels whose excavation by other means proved to be impossible. Some of the above mentioned tunnels are listed below:

TUNNEL	YEAR	Ø [m]	GROUND	OVERBURDEN max [m]	ADVANCE RATES [m/day] average ÷ max
"Tasso" (Florence to Arezzo Railway Line)	1988	12,20	Sandy silts	50	2,0 ÷ 3,2
"Targia" (Bicocca to Syracuse Railway Line)	1989	12,00	Hyaloclastites	50	2,0 ÷ 3,3
"San Vitale" (Caserta to Foggia Railway Line)	1991	12,50	Scaly clays	100	1,6 ÷ 2,4
"Vasto" (Ancona to Bari Railway Line)	1993	12,20	Silty sand and clayey silt	135	1,6 ÷ 2,6
"Tartaiguille" (Marseille to Lyon TGV Méditerranèe)	1996	15	Swelling clay	110	1,4 ÷ 1,9
"Appia Antica" (Rome Outer Ring Motorway)	1999	20,65	Sandy gravelly pyroclastites	18	2.3+3.3

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