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FIBRE-GLASS TUBES TO STABILISE THE FACE OF TUNNELS IN DIFFICULT COHESIVE SOILS

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ABSTRACT: When the engineer specialized in underground works is faced with the problem of driving a tunnel under difficult stress-strain conditions he must pay careful attention to the stability of the face; the stability of the face and the core ahead of it will have a determining effect on the stress strain response of the ground around the tunnel and this in turn will affect the long and short term stability of the tunnel.

The employment of conservation methods, in advance of the face, as researched and developed by the author over the last few years have solved, for the first time, the problems of face stability. This paper describes the study and in particular the subsequent practical application of one of these methods: reinforcement of the core of ground ahead of the face by means of fibre-glass tubes.

1. INTRODUCTION

When one talks of civil engineering, the picture that immediately comes to mind is that of a surface construction. From the engineering view point, this is a construction that is created by assembling materials (steel, reinforced concrete, etc.) selected by a design engineer. Various stress-strain states will develop within this assembly when it is acted on in a predetermined manner and the engineer must design and test the shape and dimensions of the structure accordingly.

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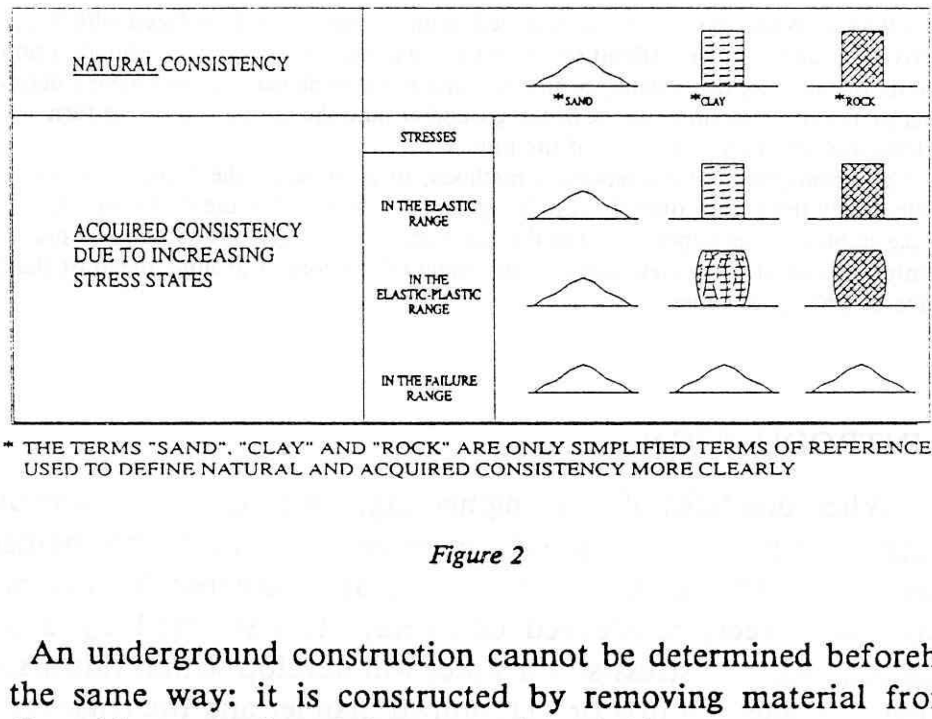
	SURFACE CONSTRUCTION	UNDERGROUND CONSTRUCTION
MEDIUM (construction material)	●	○
ACTION (loads)	●	○
REACTION (stresses and strains)	●	○

● Determined beforehand ○ Not predetermined

Figure 1



* THE TERMS "SAND", "CLAY" AND "ROCK" ARE ONLY SIMPLIFIED TERMS OF REFERENCE USED TO DEFINE NATURAL AND ACQUIRED CONSISTENCY MORE CLEARLY

Figure 2

An underground construction cannot be determined beforehand in the same way: it is constructed by removing material from a *medium* (the ground) and the properties of this are not easy to ascertain. This medium is already naturally subject to states of stress which excavation and construction *action* will modify irreversibly by triggering *reaction*. The success of an underground construction depends on controlling this reaction (fig. 1).

Anyone who sets out to design and construct an underground con-

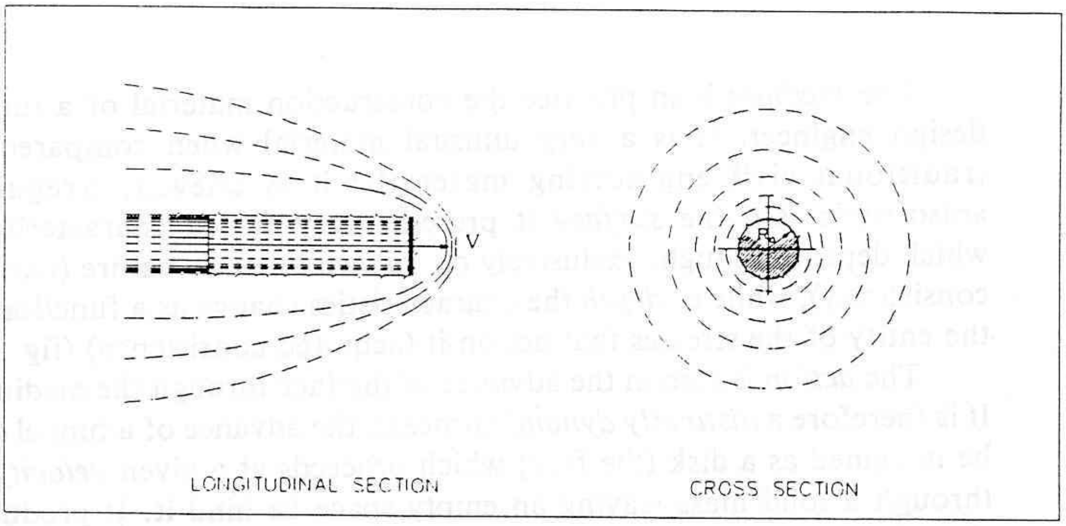


Figure 3 - The action of tunnel advance on a medium

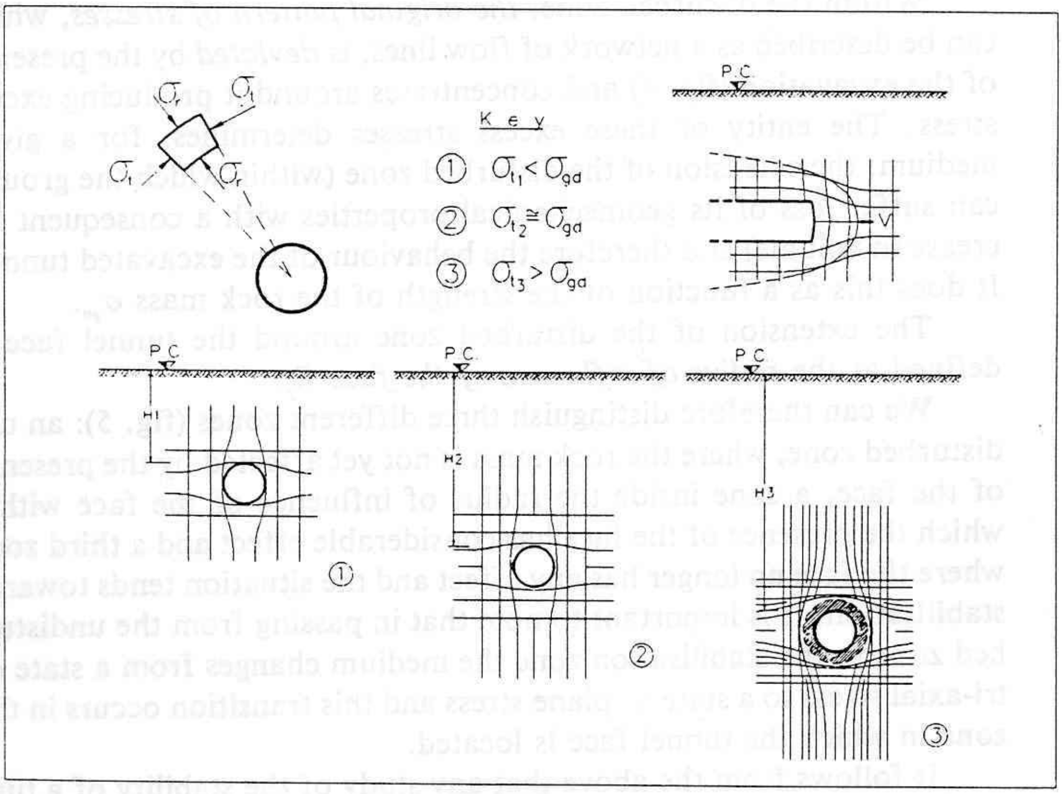


Figure 4 - The deviation of the flow of stresses due to the effect of excavation

struction cannot afford not to have a knowledge of:

- the *medium* in which he is operating;
- the *action* carried out to effect excavation;
- the *reaction* that then follows the excavation.

The *medium* is in practice the construction material of a tunnel design engineer. It is a very unusual material when compared to traditional civil engineering materials: it is uneven, irregular, anisotropic. On the *surface* it presents very varied characteristics which depend, though, exclusively on its own intrinsic nature (natural consistency), while *at depth* the characteristics change as a function of the entity of the stresses that act on it (acquired consistency) (fig. 2).

The *action* is seen in the advance of the face through the medium. It is therefore a *distinctly dynamic process*: the advance of a tunnel can be imagined as a disk (the face) which proceeds at a given *velocity* V through a solid mass leaving an empty space be hind it. It produces *disturbance in the medium*, both in a longitudinal and a transverse direction and this alters the stress states (fig. 3).

Within the disturbed zone, *the original pattern of stresses*, which can be described as a network of flow lines, is *deviated* by the presence of the excavation (fig. 4) and concentrates around it producing excess stress. The entity of these excess stresses determines, for a given medium, the extension of the disturbed zone (within which the ground can suffer loss of its geomechanical properties with a consequent increase in volume) and therefore the behaviour of the excavated tunnel. It does this as a function of the strength of the rock mass σ_{gd}.

The extension of the disturbed zone around the tunnel face is defined as the *radius of influence of the face* R_f.

We can therefore distinguish three different zones (fig. 5): an undisturbed zone, where the rock mass is not yet affected by the presence of the face, a zone inside the radius of influence of the face within which the presence of the face has considerable effect and a third zone where the face no longer has any effect and the situation tends towards stabilisation. It is important to note that in passing from the undisturbed zone to the stabilisation zone the medium changes from a state of tri-axial stress to a state of plane stress and this transition occurs in the zone in which the tunnel face is located.

It follows from the above that any study of the stability of a tunnel cannot be limited to *plane diagrams but must be three dimensional.*

The *reaction* is the *deformation response* of the medium to the action of excavation. It is generated down stream from the face, within the disturbed zone, as a consequence of the excess stress generated in the medium and it manifests first with *extrusion phenomena* at the face and then, down stream from the face, with *convergence around the excavated tunnel.*

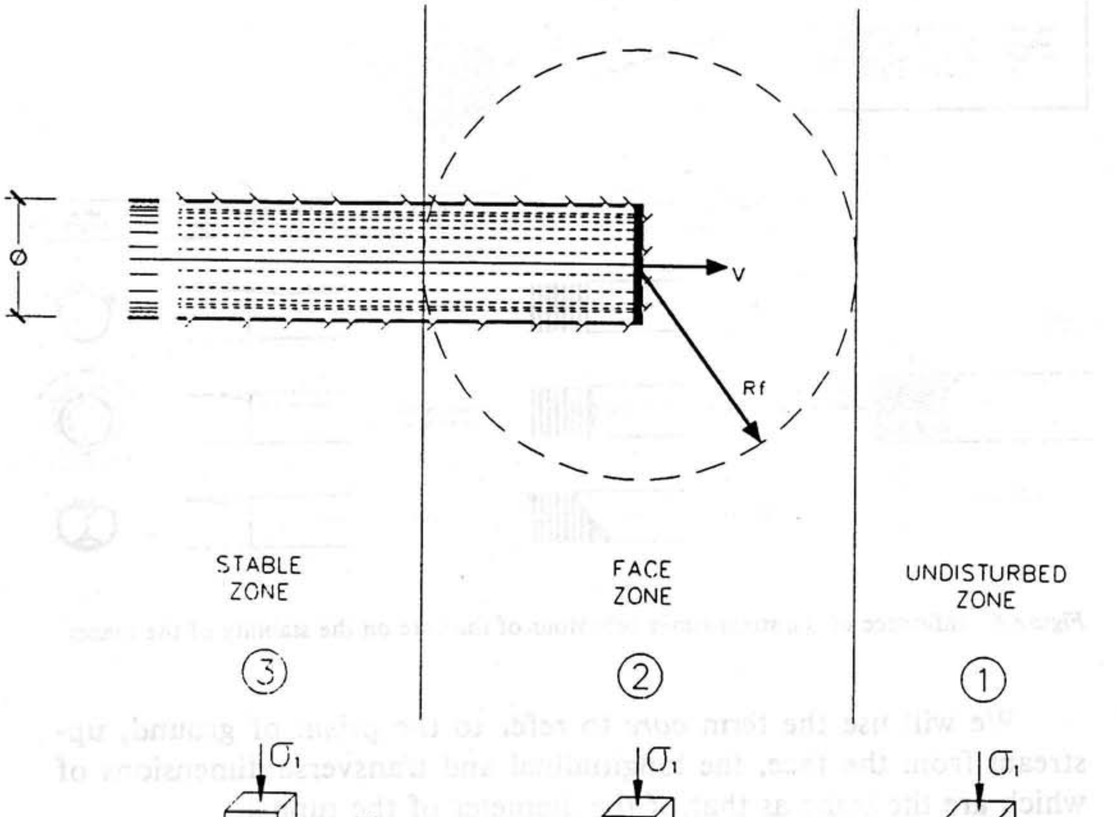


Figure 5 **- Advance of the face and characteristic zones**

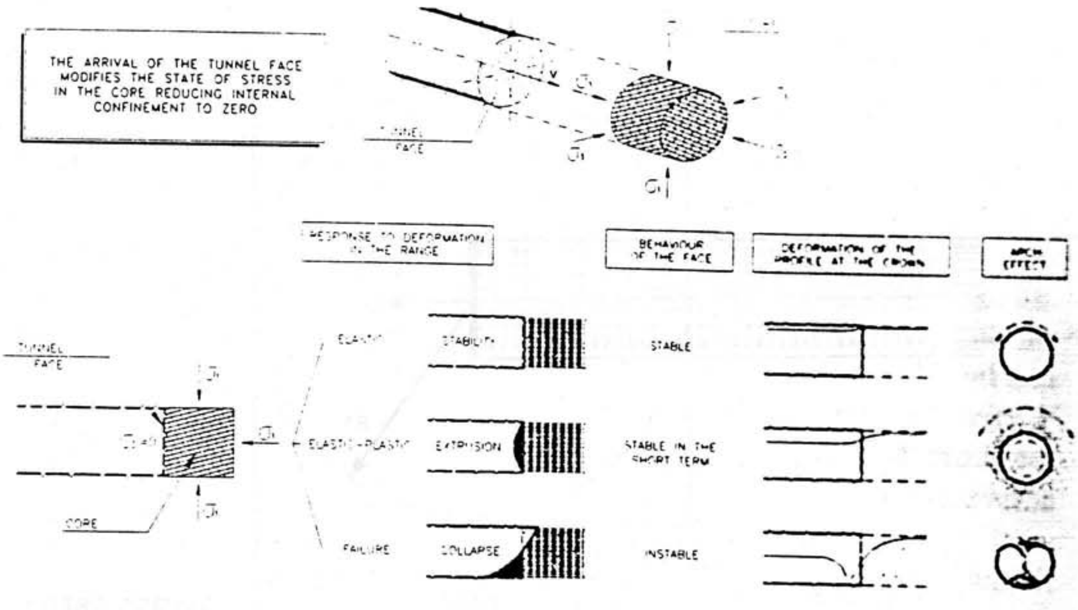


Figure 6 - Influence of the stress-strain behaviour of the core on the stability of the tunnel

We will use the term *core* to refer to the prism of ground, upstream from the face, the longitudinal and transverse dimensions of which are the same as that of the diameter of the tunnel.

Naturally, in passing from a state of tri-axial stress to one of plane stress the material in the core ahead of the face behaves differently, as it loses its strength, according to the entity of the stresses to which it is subject and to its own strengthand deformation properties (fig.6).

If the reduction to zero of the state of stress at the face ($\sigma_3 = 0$) occurs with the core stressed in the elastic range, the wall that is freed (the tunnel face) will remain *stable* with limited and absolutely negligible deformation. In this case the channelling of stresses around the tunnel («arch effect») occurs naturally, close to the profile of the excavation.

If, on the other hand, the reduction to zero of the stress state at the face ($\sigma_3 = 0$) occurs when the core is stressed in the elastic-plastic range, then the reaction will be critical and the freed wall of the core (the tunnel face) will be subject to elastic/plastic deformation towards the inside of the tunnel (extrusion) resulting in a situation of *short term stability*. A situation is triggered in which plasticisation is propagated longitudinally and radially from around the excavation and this phenomenon shifts the «arch effect» farther into the rock mass: this process can only be halted by appropriate action designed to stabilise the mass of the plasticised ground.

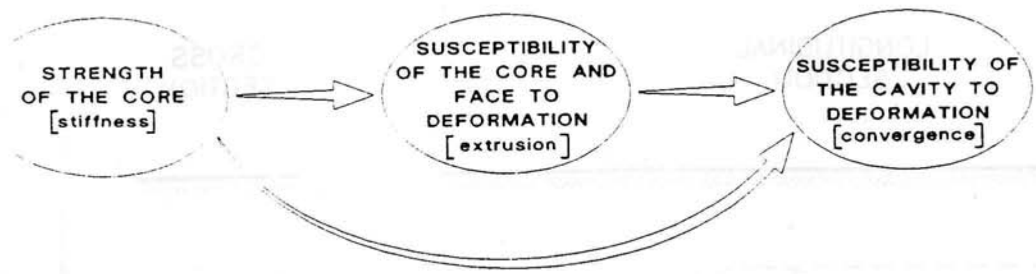


Figure 7

Finally, if the reduction to zero of the stress state at the face ($\sigma_3 = 0$) occurs when the core is stressed in the failure range, then the deformation response will be unacceptable and the core ahead of the face will become *unstable* making the creation of an «arch effect» impossible: this occurs in non-cohesive or loose soils and consequently as the «arch effect» does not occur naturally then it must be produced artificially.

It follows, then, that the *strength and deformation properties* of the core ahead of the face play a determining role as far as the original causes and development of deformation phenomena are concerned.

The radial deformation (convergence) that manifests around the excavated tunnel is a direct consequence of deformation phenomena already triggered upstream from the face, or of design and construction decisions made to regulate the deformation response up stream from the face (fig. 7).

Regulating the stiffness of the core in particular, has the effect of controlling the *deformation response* at the face (extrusion) from its onset and consequently also of controlling its development down stream from the face (convergence).

Experimental measurements of the development of deformation in the medium as related to the position of the face confirm what has been described above. They also show that the smaller the radius of influence of the face R_f, the more rigid and further into the elastic range the core will remain (fig. 8).

In order to transform the core ahead of the face into an instrument for controlling and regulating the deformation response, intervention is required upstream from the face itself in the form of advance ground improvement operations that provide an action that we will call *pre-confinement* to distinguish it from simple confinement that acts around the cavity downstream from the face (fig. 9).

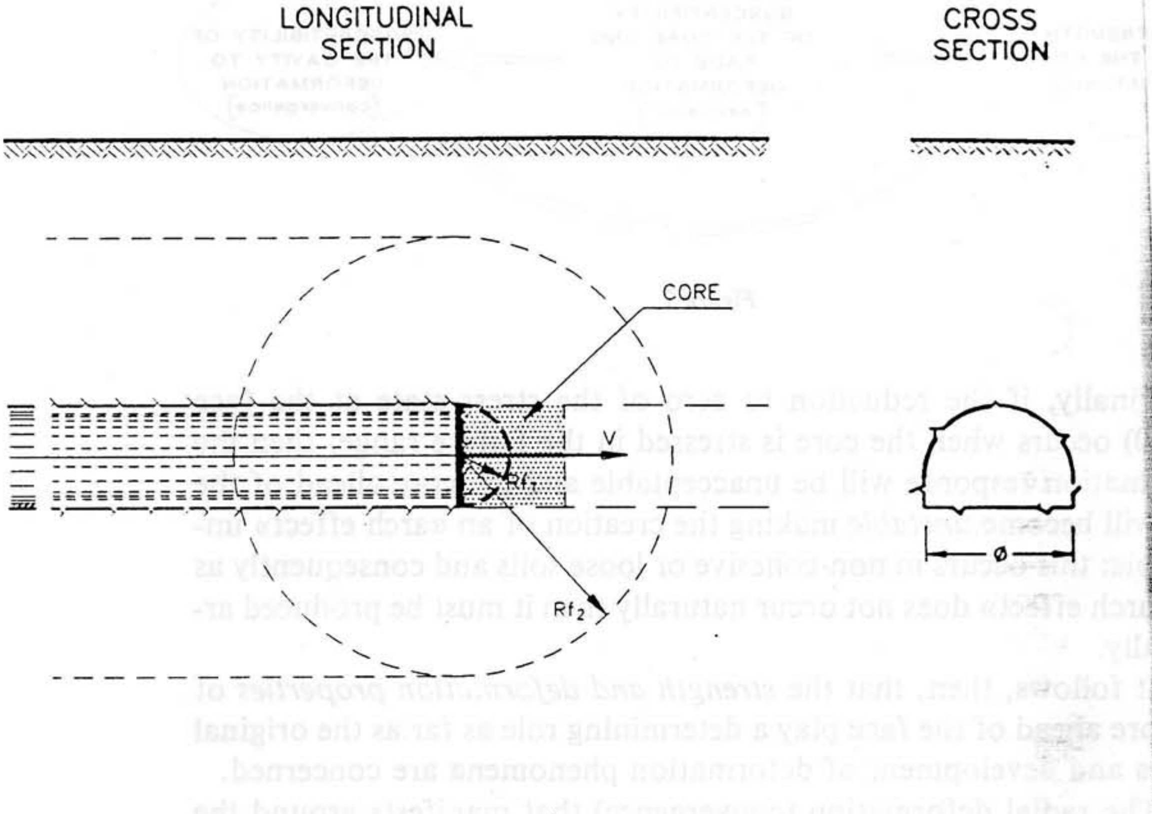


Figure 8 - **Variation in Rf as a function of the stiffness of the core**

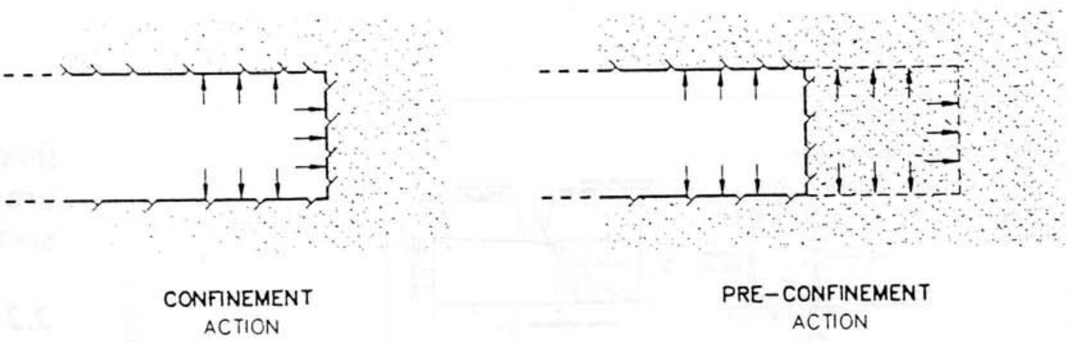


Figure 9 - Confinement action and pre-confinement action on a tunnel

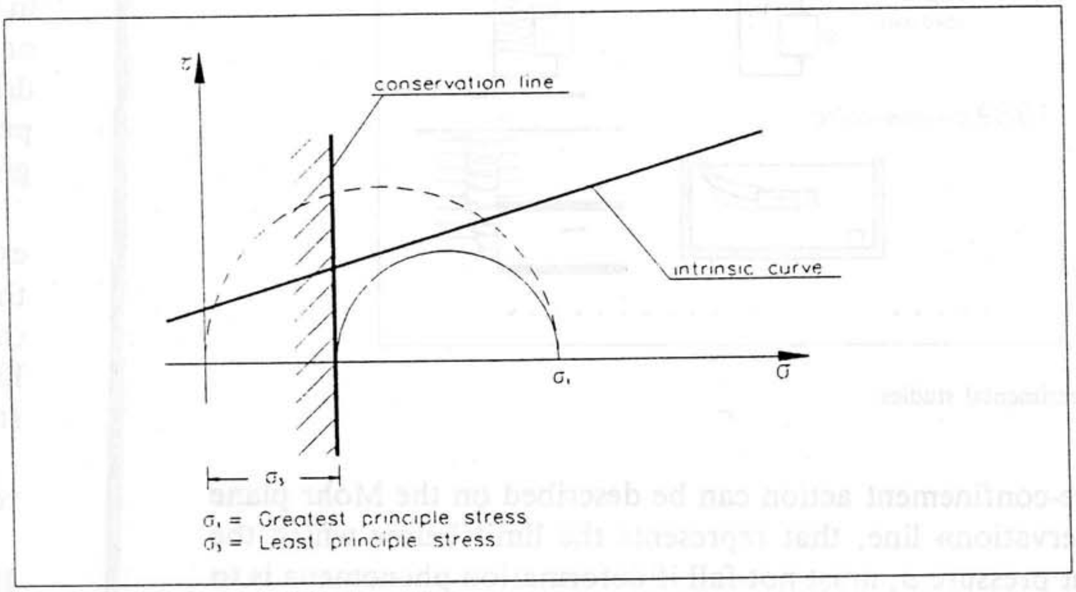


Figure 10 - Conservation line

Pre-confinement action, by acting to counteract the start of the deformation response upstream from the face, as a consequence limits its development downstream from the face making it easier to control and regulate. Therefore, in accordance with the most recent standards of the major contract awarding authorities for underground works in Italy (Autostrade S.p.A., ANAS, ITALFERR S.p.A.), the design engineer must focus his attention on the core and its strength and deformation properties: if the core is rendered sufficiently stiff, it is possible to maintain the ground inside the radius of influence of the face on the verge of the elastic range and obtain only limited deformation phenomena as a result. In doing this the principal problem of long and short term stability of underground constructions is solved.

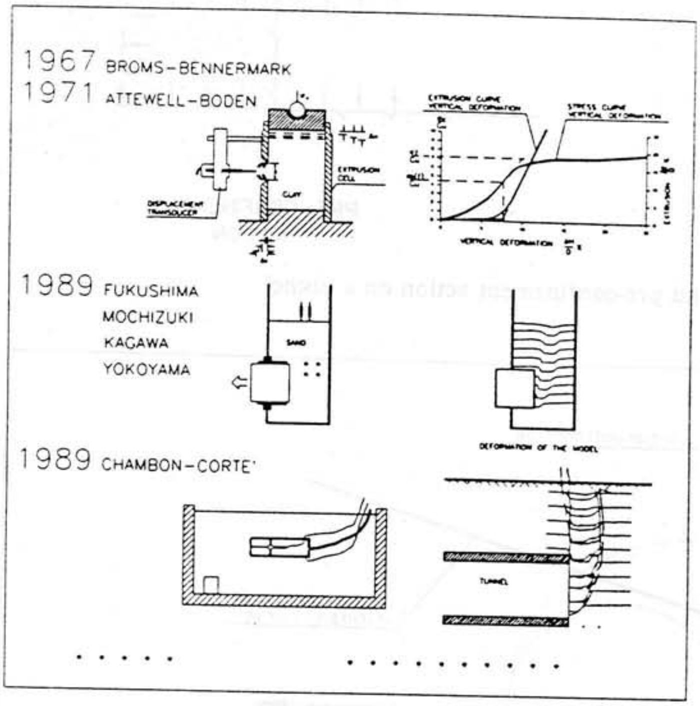


Figure 11 - Experimental studies

The pre-confinement action can be described on the Mohr plane by a «conservation» line, that represents the limit below which the confinement pressure σ_3 must not fall if deformation phenomena is to be controlled acceptably (fig. 10).

It is perhaps worthwhile, before describing the new approach to face stability, making a brief but exhaustive examination of the work of those research engineers who have in some way examined the problem of the stability of the face of a tunnel.

2. THE STUDY OF FACE STABILITY IN THE TRADITIONAL LITERATURE

2.1 Introduction

From a historical view point, the first studies of the stability of a tunnel face can be mainly summed up as the search for a reliable method of predicting possible conditions of face instability in tunnels to be driven through soft clays.

At the same time, as we will see below, *operational methods* developed in the past consisted of simply applying confinement action around the excavated tunnel down stream from the face.

For purposes of clarity, we will distinguish, in this paper, between theoretical contributions, either experimental or mathematical, and practical operational contributions in terms of instruments and specific construction systems.

2.2 Theoretical and experimental methods

On looking back at the first *experimental studies* of models of extrusion phenomena, the work carried out by Broms and Bennermark in 1967 seems worth recalling. This research showed that undisturbed or remodelled samples of clay will extrude through a vertical hole drilled in a hollow metal cylinder when the total vertical pressure is approximately six times greater than the undrained cohesion of the ground at the depth of the hole.

Clearly there are obvious similarities between laboratory obtained extrusion and that observed at the cutting face of a tunnel driven through clay. On the basis of these similarities the authors concluded that in order to maintain stability at the face, the ratio between vertical lithostatic pressure and undrained cohesion should be less than the stability ratio Ns, equal to a value of between 6 and 8.

Following these findings, Attewell and Boden (1971) took up the research carried out by Broms and Bennermark and developed it further.

A re-examination of the failure concepts on which Broms and Bennermark's conclusions were based suggested that a ratio derived from the maximum acceleration of extrusion would be more appropriate for defining that critical depth which is of interest in the practical business of tunnel construction. With this method they calculated a critical value for the stability ratio of 4.5.

Following this work numerous studies have been carried out by other researchers. Amongst these, the experiments of Fukushima, Mochizuki, Kgawa and Yokoyama (1989) should be mentioned. These were designed to study the behaviour of a tunnel face in sand reinforced with steel bolts arranged in various ways. Also to be mentioned is the work carried out by Chambon and Cortè (1989) using centrifuge tests to study the yield behaviour of a tunnel face in sand at shallow depths (fig. 11).

These last experiments in particular had little follow up as they relate to very specific operational conditions (tunnels in loose ground at shallow depths) difficult to apply to many real situations.

As far as *theoretical studies* are concerned, the work by Lombardi (1974) is of major importance. Lombardi was perhaps the first resear-

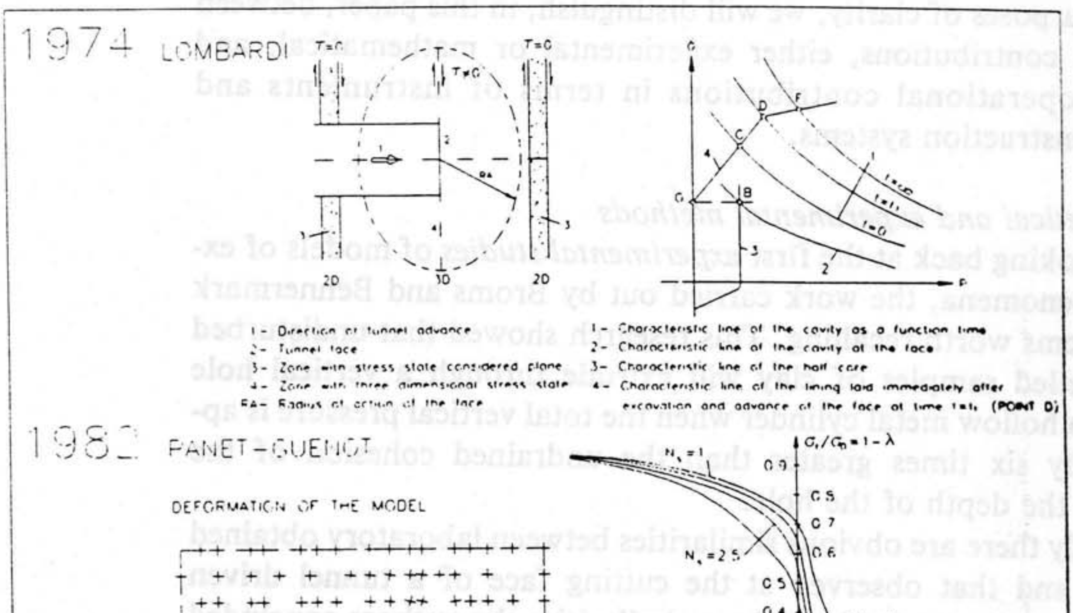


Figure 12 - Theoretical studies - stress-strain analysis

cher to realise the importance of a three dimensional approach to the statics of tunnel construction. He furnished predictions not just in terms of stress behaviour but also in terms of deformation.

Lombardi has plotted the characteristic lines that describe the behaviour of the core ahead of the face, of the face itself, of the cavity and of lining structures. The result is a powerful analytical and graphical calculation instrument for the study of stress-strain behaviour of the cavity and the core of ground ahead of the face (fig. 12).

Though it does not provide practical indications concerning methods for regulating the deformation response at the construction stage, it does have the great merit of clarifying the real terms of the problem for the first time.

Subsequently, Panet and Guenot (1982) developed stress-strain study of the face and the cavity further using an axial symmetric finite element model in the elastic-plastic field obtaining convergence-confinement curves conceptually analogous to the characteristic lines.

On the other hand it must be said that the characteristic lines instrument is not really sufficient to deal with the greater part of the geometries of the problem due to the simplified nature of the hypotheses on which it is based.

Another line of research is that of yield calculation begun in 1980 by Davis, Gunn et al. and taken up again in 1988 by the French school (Panet, Leca, etc.) and more recently by Chambon Cortè (1990). This method can be used to provide information on the stability of a tunnel face by means of plasticity theory (fig. 13).

These latter methods define the lower and upper limits of potentially bearable load systems by taking into account both statically and plastically admissible load systems and kinematically admissible yield point mechanisms respectively. The difference between the limits obtained using these two criteria is, however, very large due to the small number of load systems and yield point mechanisms considered and no precise definition in this sense has yet been made of the range of loads that can be considered as constituting safe conditions.

The authors have studied both purely cohesive materials and materials with cohesive and frictional properties. In the latter case approximation through the upper limit has not yet been thoroughly established.

Finally other researchers (Ellstein (1986), Tamez and Cornejo (1988), etc.) followed a third type of approach essentially based on the limit equilibrium method. Using this method it is possible to define a failure point safety coefficient from the equilibrium of the unstable masses of the ground involved in the phenomenon. (fig. 14).

The difference between the formulas produced by different researchers can be essentially brought down to the type of yield surface hypothesised as the definition of this is the main limit of these methods.

2.3 Operational methods

Until just a few years ago the problem of the stability of the face of a tunnel was always strictly correlated with the onset of operational difficulties that hindered normal excavation work during tunnel advance. It was never suspected that it might have repercussions on the long and short term statics of the tunnel. The methods adopted in conditions of instability such as these were aimed at providing an action of gravitational confinement around the cavity, with confinement pressure in the range of a few tenths of a kPa. This was insufficient to

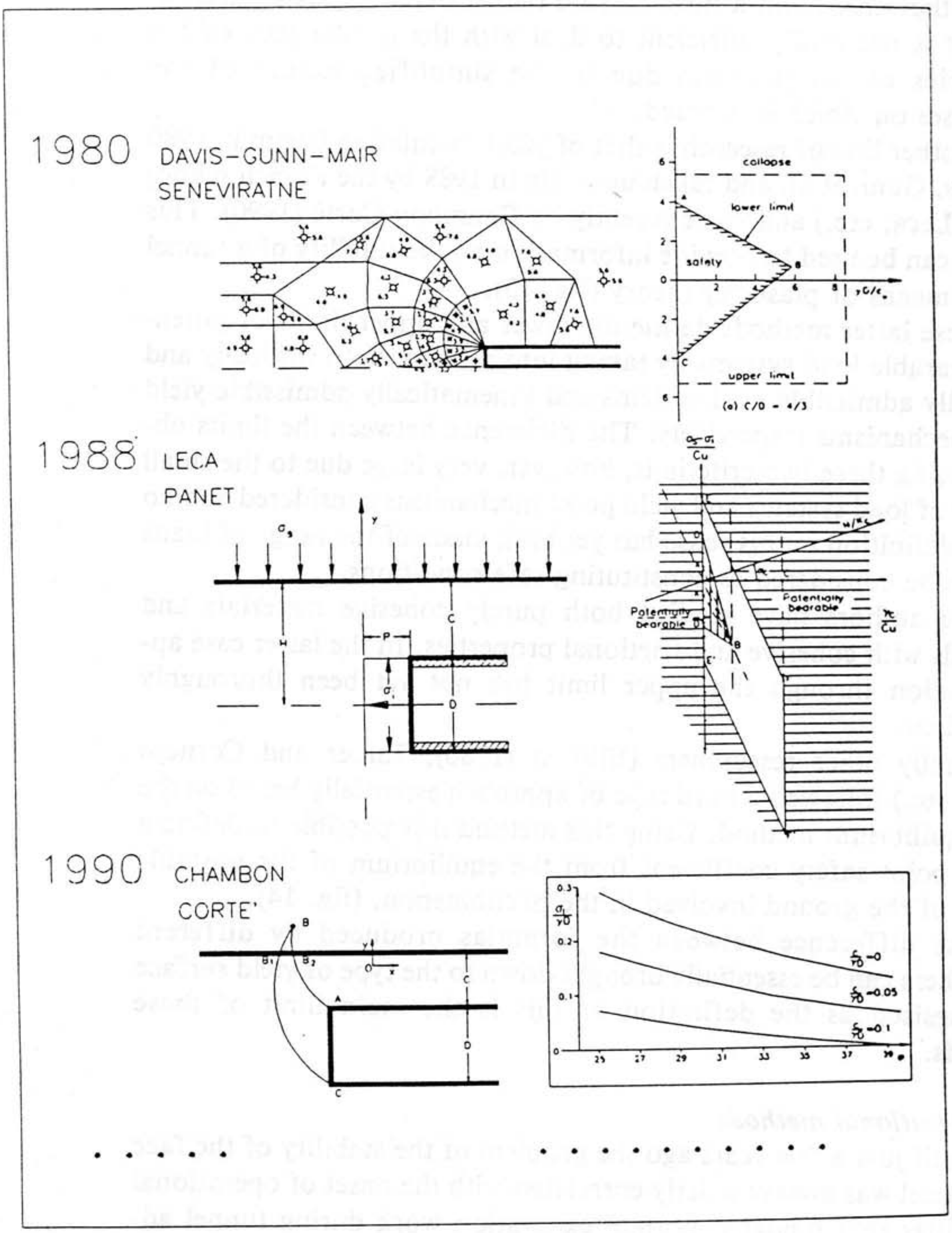


Figure 13 - Theoretical studies - failure point calculation

prevent loosening of the ground and consequent loss of its strength properties. Such methods, the most well known of which was the NATM method, involved dividing up the face cross section and ex-

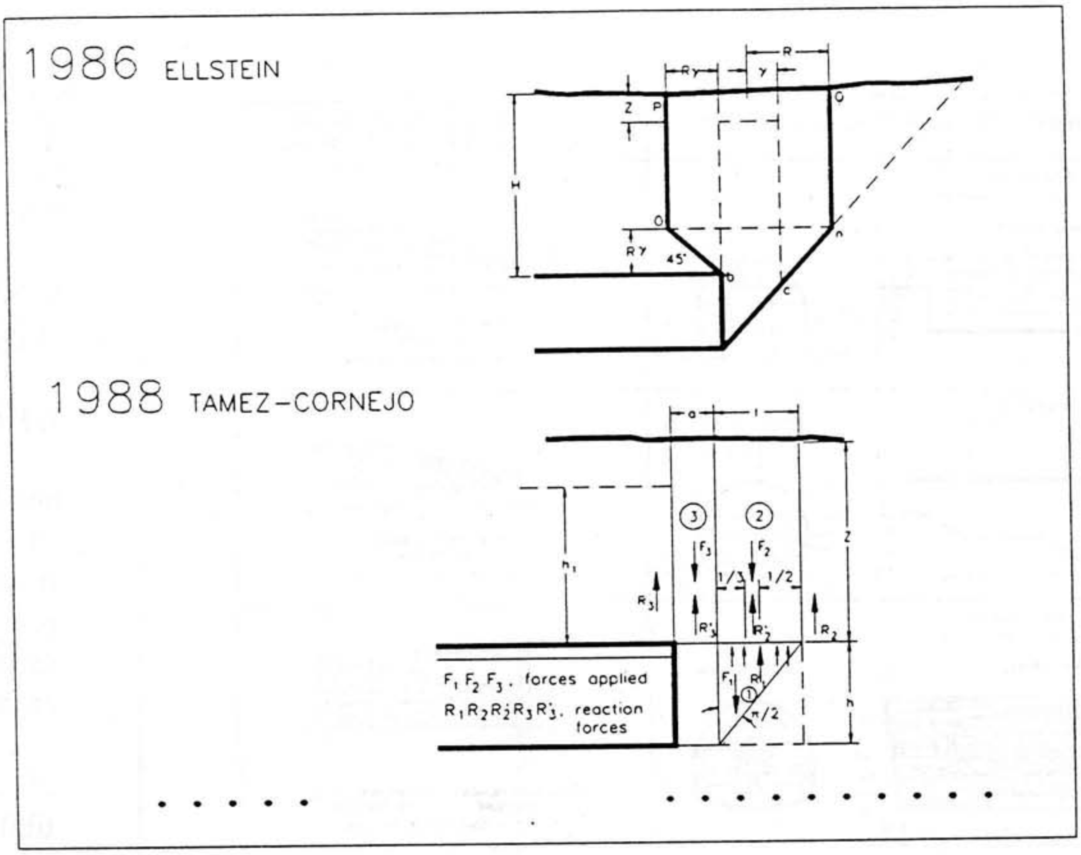


Figure 14 - Theoretical studies - stability analysis

cavating in stages (fig. 15). These operational methods were further complicated and made more difficult in cases where tunnel advance was through loose ground where confinement of deformation was of prime importance especially when excavation interfered with existing structures.

Subsequently special technologies were developed for safe tunnel advance in loose soils. Under these conditions the problem of the stability of the face and walls becomes of prime importance for the short and long term statics of a tunnel. These technologies involve (fig. 15) confinement of the face using mechanised shields (partial or total mechanical confinement) or, in the presence of a water table, shields with pressurised fluids (full hydraulic confinement). The latter in particular are capable of providing confinement of the face of some kilopascals by means of air under pressure (compressed air shield) or of fluids under pressure (hydro-shields and E.P.B.s). They provide a confinement action on the rock mass that is sufficient to limit convergence of the cavity and to prevent loosening of the ground surrounding it.

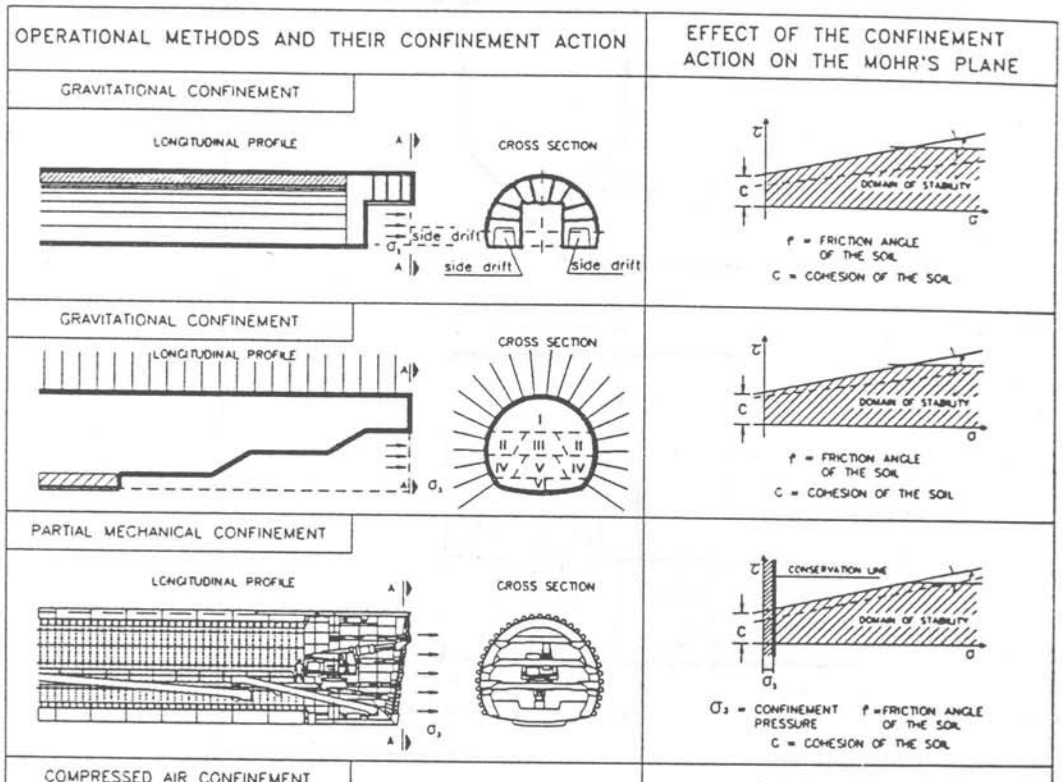


Figure 15 - Operational methods used to stabilise a tunnel face

The employment of these construction systems, however, assumes a very limited field of use due to the difficulty of adapting them to unexpected variations in geology and because they cannot be used in swelling and thrusting ground. Consequently they are extremely inflexible systems.

3. NEW APPROACH TO THE STUDY OF FACE STABILITY (ADECO-RS)

3.1 Introduction

In the past, studies carried out on the stability of the face of a tunnel have only dealt with the problem in terms of maintaining the equilibrium of the face by providing a «confinement» action on the face itself from inside the cavity. Such confinement action can be provided, as we have seen, by methods that employ gravitational, mechanical or hydraulic confinement, but these can only generate rather modest pressures in the kPa range.

This is due to the fact that importance of the core of ground ahead of the face was not understood sufficiently. The stiffness and susceptibility to deformation of this core has an effect on the initial causes and development of deformation phenomena in the tunnel. The real innovation introduced by the new approach to the study of face stability is that the latter is no longer studied passively as a core of ground to be supported by confinement action from within the cavity, but is studied actively as a structural element the stiffness of which will determine the deformation response at the face (extrusion) and around the cavity (convergence).

It is only recently that new methods have been studied by the author and developed by the engineers at Rocksoil S.p.A. in Milan designed to stabilise the face and *conserve* the integrity of the core of ground ahead of it. First insight was needed into the role and influence of the core on long and short term stability of a tunnel and secondly it had to be realised that the core could become a genuine structural tool for regulating deformation in all types of ground.

These methods work by acting inside the rock mass ahead of the face, when this is still affected by triaxial forces, and by creating a preconfinement effect (fig. 16) that can be of three types:

- direct (*direct conservation methods*) when the method acts on the ground surrounding the core by creating resistant shells of improved ground or steel fibre-reinforced shotcrete that create an artificial arch effect lightening the load of excess stresses on the core;

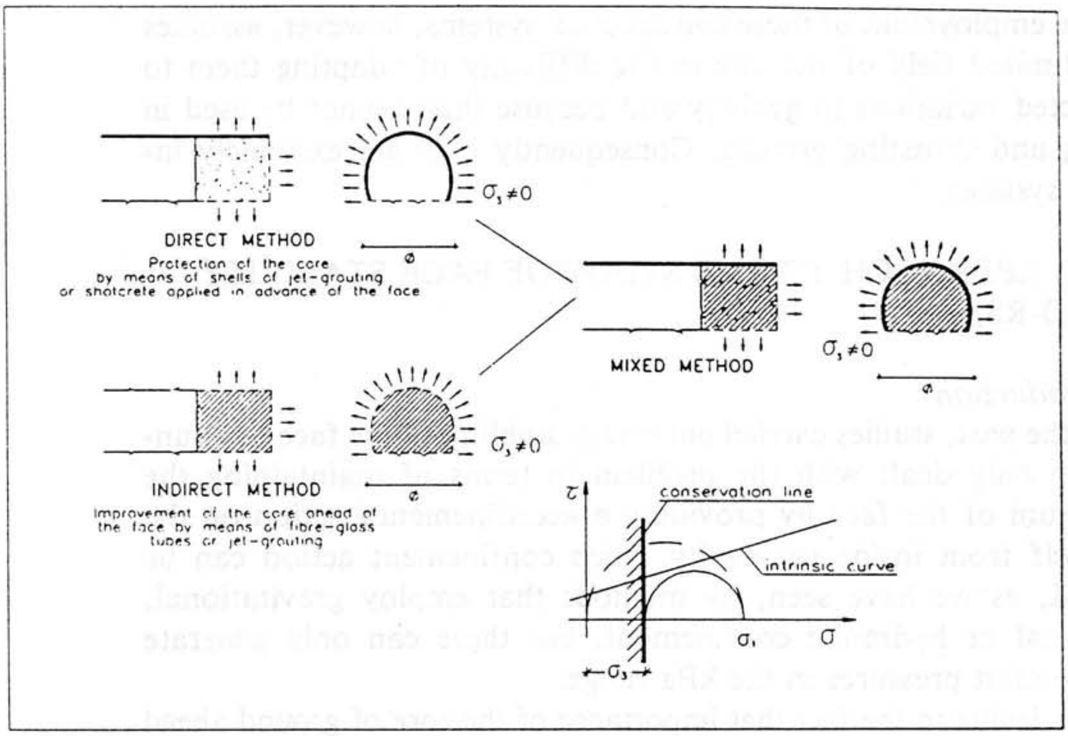


Figure 16 - Conservation methods

- indirect (*indirect conservation methods*) when the method acts directly on the core improving it in advance;
- mixed (*mixed conservation methods*) when the method acts both directly on the core and also around it;

The availability of conservation excavation technology has led to a true revolution in the approach to underground excavation in soft ground and to the consequent development of an innovative method of design and construction, the ADECO-RS (Analysis of COntrolled DEformation in Rocks and Soils) method. In the last few years, the use of this method in Italy has produced exceptional results that until recently would have been thought impossible and this has actually been done *in the field of the most difficult grounds* where other theories and other methods generally find their most serious limits.

Figure 17 collects together newly conceived instruments for stabilisation that can be termed «conservation»:

- horizontal jet-grouting systems, for the excavation of tunnels in non cohesive or poorly cohesive soils (employed for the first time in the world in Italy in 1983 for the «Campiolo» tunnel on the Pontebba to Tarvisio railway line - Rocksoil S.p.A. of Milan were the design consultants);

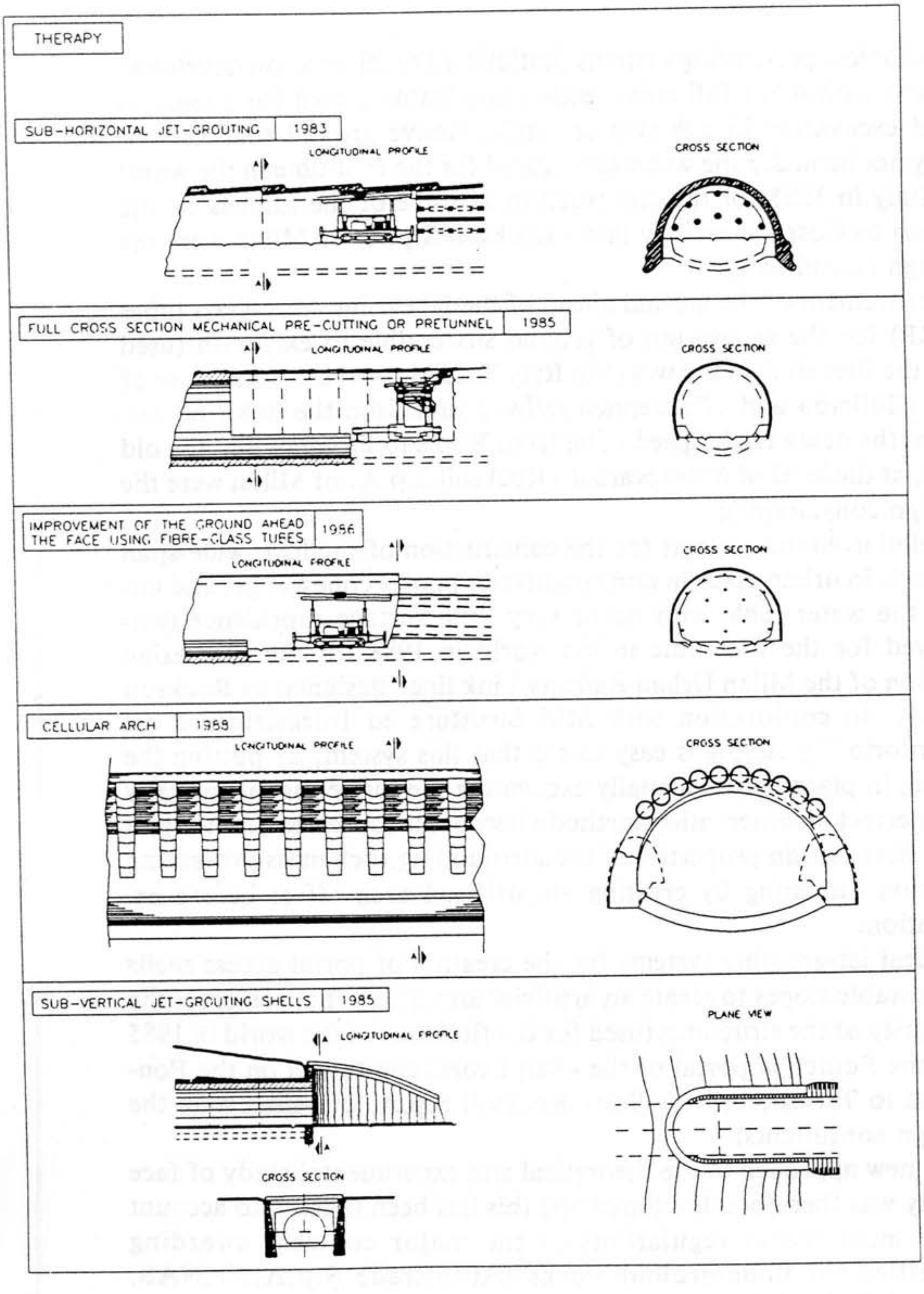


Figure 17 - Conservation methods by pre-confinement action

- mechanical pre-cutting systems, initially a French method developed by the author for full cross section application, used for programmed excavation in cohesive or semi-cohesive ground that may or may not be under the water table (used for the first time in the world in Italy in 1985 for the construction of some of the tunnels on the Sibari to Cosenza railway line - Rocksoil S.p.A. of Milan were the design consultants);
- improvement of the ground ahead of the face using fibre-glass tubes (FRP) for the excavation of ground susceptible to extrusion (used for the first time in the world in Italy in 1988 for the construction of the «Talleto» and «Caprenne» railway tunnelson the link line between the new «High Speed» line from Rome to Florence and the old line, at the level of Montevarchi - Rocksoil S.p.A. of Milan were the design consultants);
- «Cellular Arch» systems for the construction of shallow, wide-span tunnels in urban areas in non cohesive or poorly cohesive ground under the water table with no or very little surface subsidence (employed for the first time in the world in 1988 for the «Venezia» station of the Milan Urban Railway Link line - designed by Rocksoil S.p.A. in conjunction with MM Strutture ed Infrastrutture del Territorio S.p.A.); it is easy to see that this system, by putting the lining in place before actually excavating the tunnel, is intrinsically the perfect «Conservation method» because it conserves the strength and stress-strain properties of the surrounding rock mass preventing it from loosening by creating an artificial arch effect before excavation.
- vertical jet-grouting systems for the creation of portal access shells in unstable slopes to create an artificial arch effect that preserves the integrity of the structures (used for the first time in the world in 1985 for the Pontebba portal of the «San Leopoldo» tunnel on the Pontebba to Tarvisio railway line - Rocksoil S.p.A. of Milan were the design consultants).

A new approach to the theoretical and experimental study of face stability was therefore developed and this has been taken into account by the most recent regulations of the major contract awarding authorities for underground works (Autostrade S.p.A., ANAS, ITALFERR S.p.A.).

Properties concerning the strength of the rock mass to be excavated are ascertained from «in situ» and laboratory tests (**survey phase**) and these are used to make predicitons of the stress-strain response of the face (diagnostic phase), that can be classified as being in one of three states: stable, stable in the short term, unstable. The most appropriate ground improvement method, which may be ahead of the face, must then be decided and checked **(therapy phase)**. The operational phase then follows with actual construction work in the tunnel during which monitoring of the construction methods chosen is carried out **(monitoring phase)**.

3.2 Diagnosis

«In situ» surveys and laboratory tests provide information on the geology, geotechnics, rock and soil mechanics and hydrogeology of the ground through which the tunnel is to pass and this information is used to make predictions of the deformation response of the face using theoretical and experimental methods. The aim is to establish the confinement and pre-confinement operations necessary and which type of tunnel advance to employ.

With the stress-strain, «characteristic lines» method (Lombardi 1974), given a circular cavity and hydrostatic type stress-strain conditions (tunnels at great depth) it is possible to ascertain whether deformation of the core ahead of the face will occur in the elastic range (stable face) or in the elastic-plastic range (face stable in the short term or unstable).

As can be seen from fig. 18, if the characteristic line of the half core intersects that of the cavity at the face maintaining elastic conditions then the behaviour of the future tunnel will be that of a stable face (category A).

If the characteristic line of the half core intersects that of the cavity at the face under elastic-plastic conditions, then there will be a *face stable in the short term* (category B).

If the characteristic line of the half core does not intersect that of the cavity at the face because elastic-plastic deformation has reached failure point, then there will be an *unstable face* (category C).

For very shallow tunnels the stability of the face can be predicted using the laboratory extrusion tests designed by Broms and Bennermark (1967). These tests are valid for clayey soils and give a ratio related to the undrained cohesion of the rock mass to be bored.

Alternatively, theoretical methods based on the determination of a safety coefficient for the equilibrium stability of the face can be used. These methods hypothesise particular slip surfaces.

New construction methods mean that full face tunnel advance is

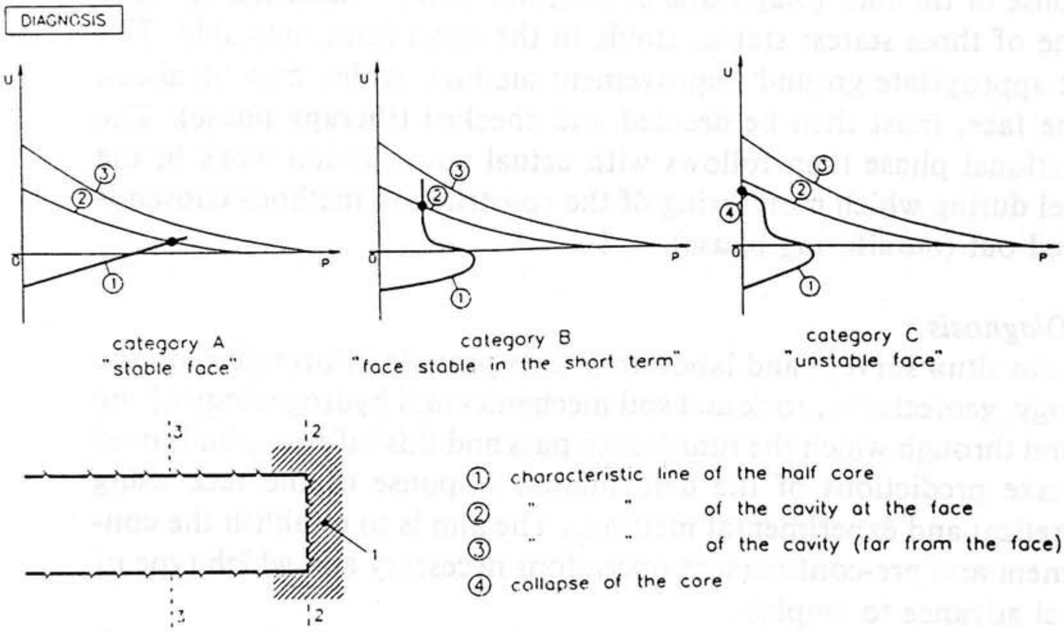


Figure 18 - **Prediction of the behaviour category by means of characteristic lines**

possible even in soft cohesive ground and consequently a better modelling of extrusion problems at the face was needed. As a result laboratory tests were designed to consider:

- the presence of a lined cavity;
- a triaxial stress-strain state;
- the natural state of consolidation of the ground (this varies according to the size of the overburden);
- the possibility of stiffening the core of ground ahead of the face.

Recently, integrated methods have been developed based on physical and mathematical modelling of the extrusion of samples of clayey ground. These methods are used to evaluate the deformation response of the face and also to rate the soil mechanics parameters (c, φ, E) for subsequent use in finite element calculations.

The triaxial extrusion cell test

Physical modelling of the excavation of a tunnel at any depth is carried out by inserting a sample of the ground into a triaxial cell that provides the natural conditions of confinement of the mass. The pressure in a cavity made in the sample itself is then reduced from inside the cavity.

It is possible to calculate extrusion at the face of the cavity as a

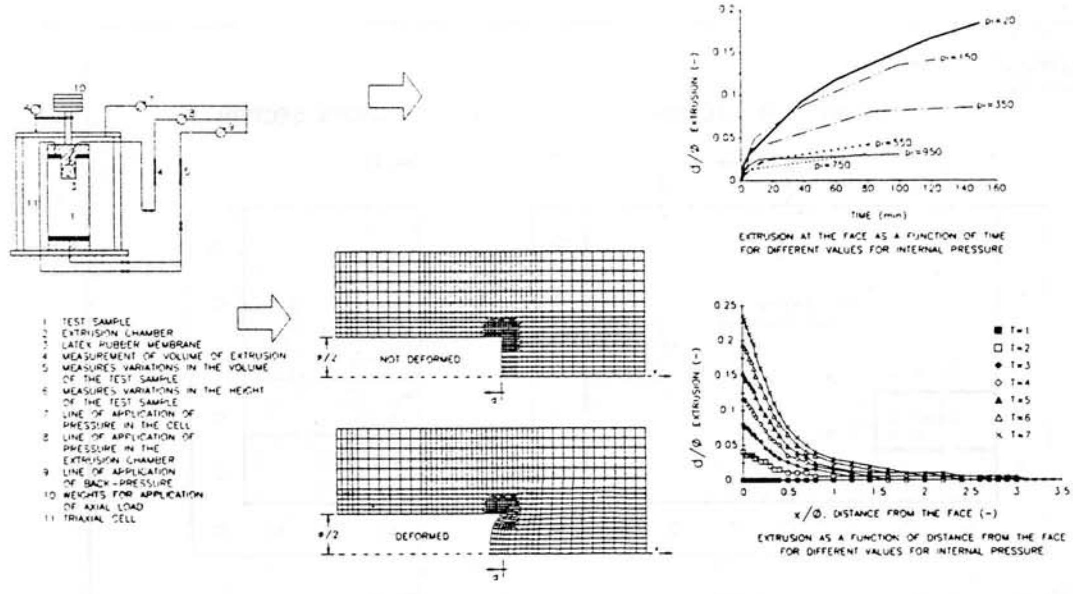


Figure 19 - Triaxial extrusion cell test

function of time, by simulating tunnel advance.

What is more finite element mathematical modelling of the laboratory testing has been carried out. This is done to rate the soil mechanics parameters (c, φ, E), furnished by the survey phase, with respect to the face extrusion values.

Curves are then plotted which give extrusion as a function of distance from the face, as the internal pressure of the cavity decreases (fig. 19).

The centrifuge extrusion test

While the study of the construction phases of a tunnel using numerical models reveals on the one hand the considerable power of the finite element method, on the other it also reveals its considerable limits. As in fact was remarked in the introduction, the medium in which a tunnel is constructed possesses properties that are not known with certainty and in any case this medium is almost never uniform: the objective difficulty of reproducing all the phenomena that manifest in reality also derives from this.

It is therefore logical to conclude that the rating of a numerical model must involve an experimental stage; in practice experiments were carried out using a centrifuge for accurate simulation of the system of external forces acting in reality.

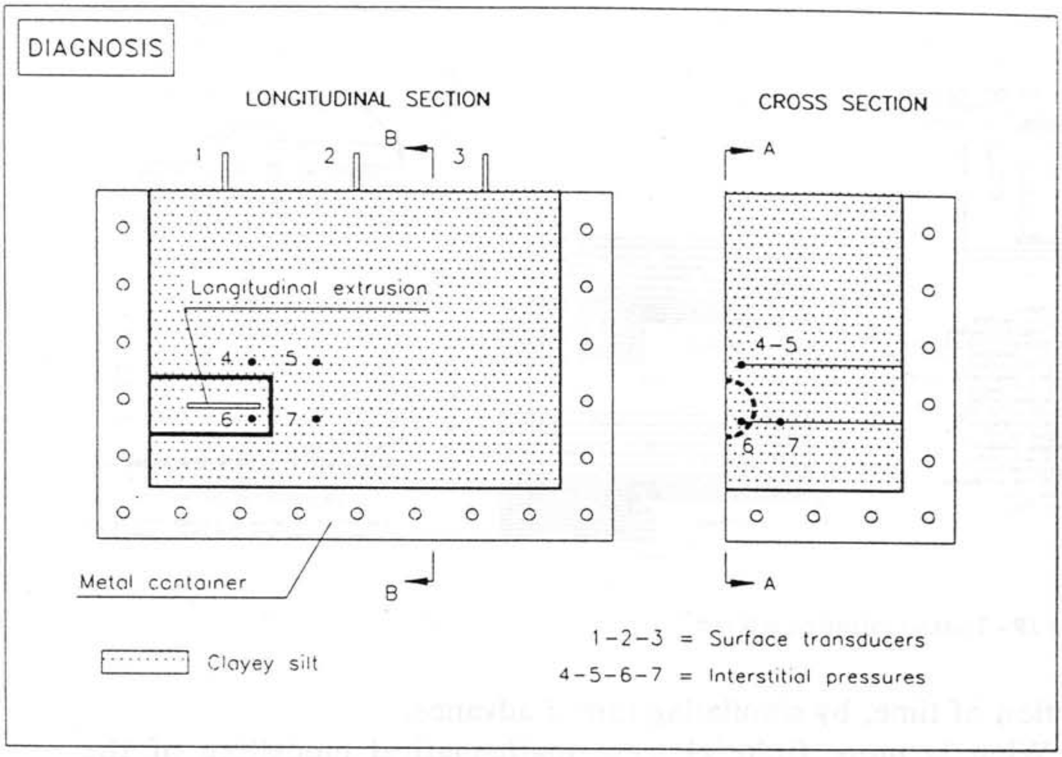


Figure 20 - Centrifuge extrusion test: scheme of the model

A description of one of these tests is given below. It was designed to simulate an overburden of 60 m, with face advance in the absence of ground improvement. A model 42x16x62 cm in size was constructed with dry clay, in water, and subjected to consolidation to create a state of stress in the ground equivalent to that of the desired depth.

The tunnel cavity is first created, with due precautions, in the test sample and then a steel tube is inserted to represent a first approximation of the primary lining, the secondary lining and the tunnel invert. The ground near the transparent wall is marked to make it easier to read the movements at the end of the test. Photo 1 shows the model before the test.

Figure 20 shows how the surface movement transducers (1-2-3), the extrusion and interstitial pressure transducers (4-5-6-7) are placed.

The geostatic pressures for an overburden of 60 m are reproduced in the centrifuge with rotation reaching 175 revolutions per minute.

Analysis of the extensometer and interstitial pressure readings, as taken at the positions shown in Figure 20, lead to some interesting considerations: qualitatively they correspond quite well to the readings

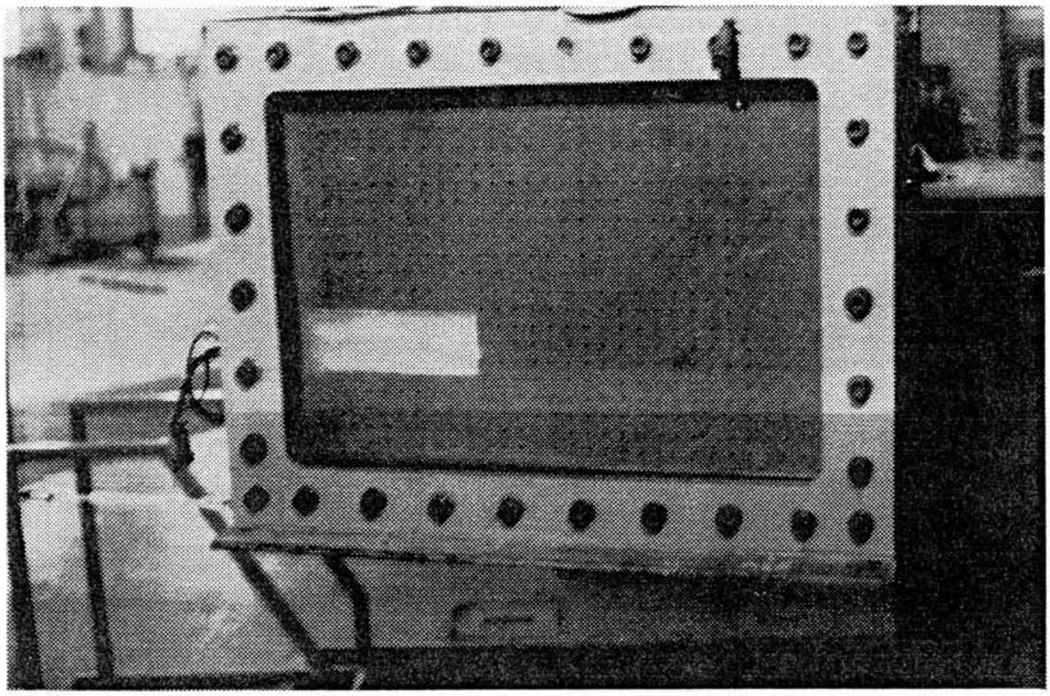


Photo 1 - Centrifuge extrusion test: the model before the test

taken daily on the actual construction site; in particular, Figure 21a shows how extrusion of the face occurs very quickly during the transient state when the load is removed or in other words during the brief interval when the cell pressure is reduced. To be more precise this velocity increases as the core is relaxed or as the face approaches the measuring station positioned on the cross section in question. The non linearity of the response is more evident in the graph in Figure 21b where, by placing side by side, extrusion deformations without the viscous component are shown. It should be noted, with respect to this, that the load removal steps are all equal (-2 bar). Figure 21c shows the extrusion of the face as it accumulates over all the intervals of time at constant load: the viscous component can be clearly seen, and at the end of the test is equal to 50% of the total. It is felt that the results of this test are strongly affected by the water content.

The interstitial pressure during the first load removal steps changes very rapidly indicating rapid increase in drainage resulting from an equally rapid relaxation of the ground. This is also confirmed by a simultaneous increase in the velocity of extrusion of the face.

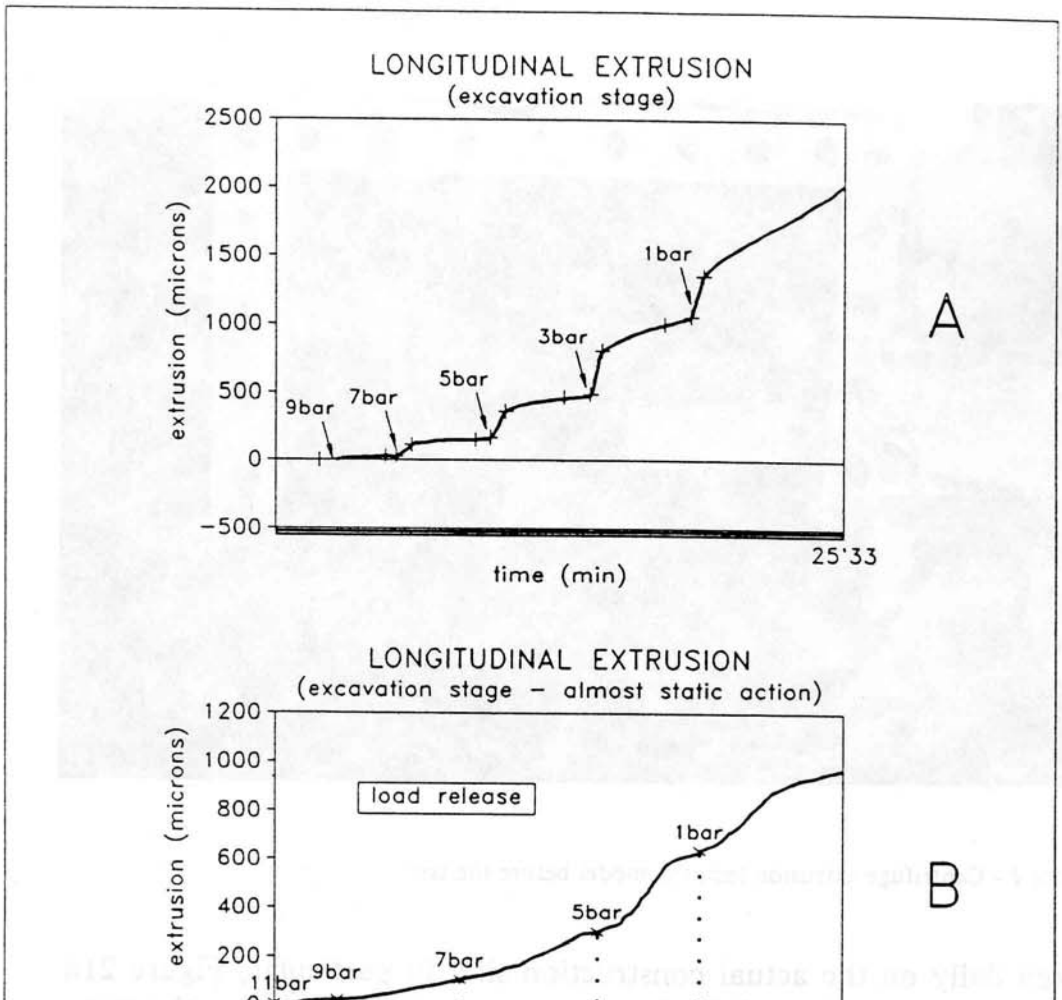


Figure 21 - **Centrifuge extrusion test: the results**

As far as the time function is concerned it should be noted that a quadratic scaling factor makes the duration of the centrifuge tests equal to the average time actually employed (3 weeks) on site to advance more than two tunnel diameters; during this period the stress levels pass from initial levels (practically geostatic) to those existing near the face.

3.3 Therapy

This phase follows the predictions of the entity and type of deformation that may occur during tunnel advance, made during the previous phase. It consists of deciding operational methods that will guarantee stability of the works and above all of deciding the balance of stabilisation instruments to be applied either on the face or around the tunnel.

Finally the dimensioning of the structural elements to be adopted and the statics of these is decided with the definition of the geometry and strength properties of the materials.

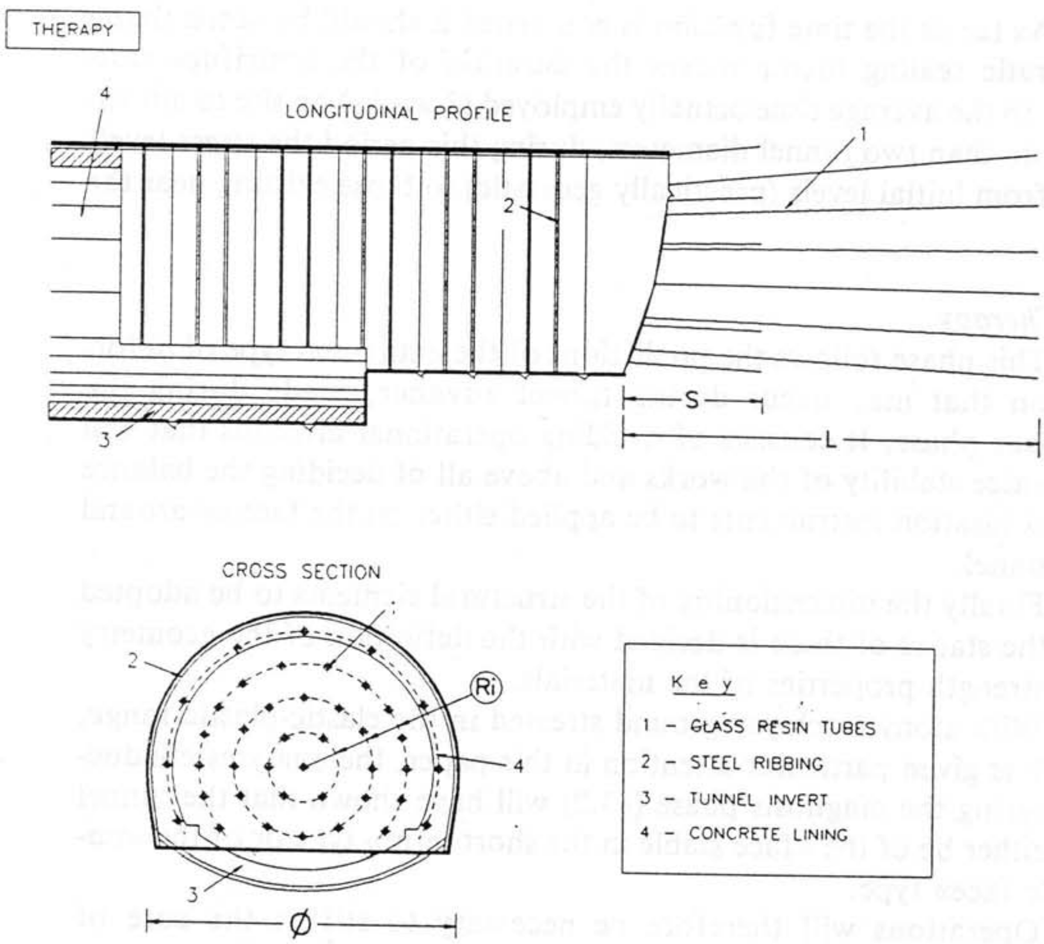
With stony or clayey ground stressed in the elastic-plastic range, which is given particular attention in this paper, the analyses conducted during the diagnosis phase (§3.2) will have shown that the tunnel will either be of the «face stable in the short term» type or of the «unstable face» type.

Operations will therefore be necessary to stiffen the core of ground ahead of the face providing therefore a *pre-confinement action*.

In these types of ground (fragmented rocks, silts, clays) the most suitable and effective method of construction is with improvement of the ground ahead of the face by means of fibre-glass tubes and carried out before excavation. They are inserted to a depth of not less than one tunnel diameter in a direction sub-parallel to the longitudinal axis of the tunnel. If necessary, this method can be combined with other conservation methods.

3.3.1 Operational proposals: fibre-glass (FRP) tubes for the stabilisation of tunnel faces

The method consists of starting by drilling a series of holes, dry, evenly distributed over the face, sub-parallel to the longitudinal axis of the tunnel. Fibre-glass tubes are inserted into these and immediately injected with cement mortar (Fig. 22). The best method for creating the holes is helical cutting.



PARAMETERS THAT GOVERN THE METHOD :

- GEOMETRIC DISTRIBUTION (Ri)
- TUBE LENGTH (L)
- TUBE DIAMETER ($\emptyset_t$)
- INTENSITY (No. per sq m)
- OVERLAP (S)

$L \geqslant \emptyset$

$S \geqslant \emptyset/2$

$\emptyset_t = 4 \div 6$ cm

Figure 22 - **Advance ground improvement of the face with fibre-glass tubes: typical scheme**

The fibre-glass tubes are manufactured with thermosetting polyester resin reinforced with glass fibre which general constitutes more than 50% of the content by weight. Adherence of the external surface of the tube is generally improved by cutting spiral grooves to

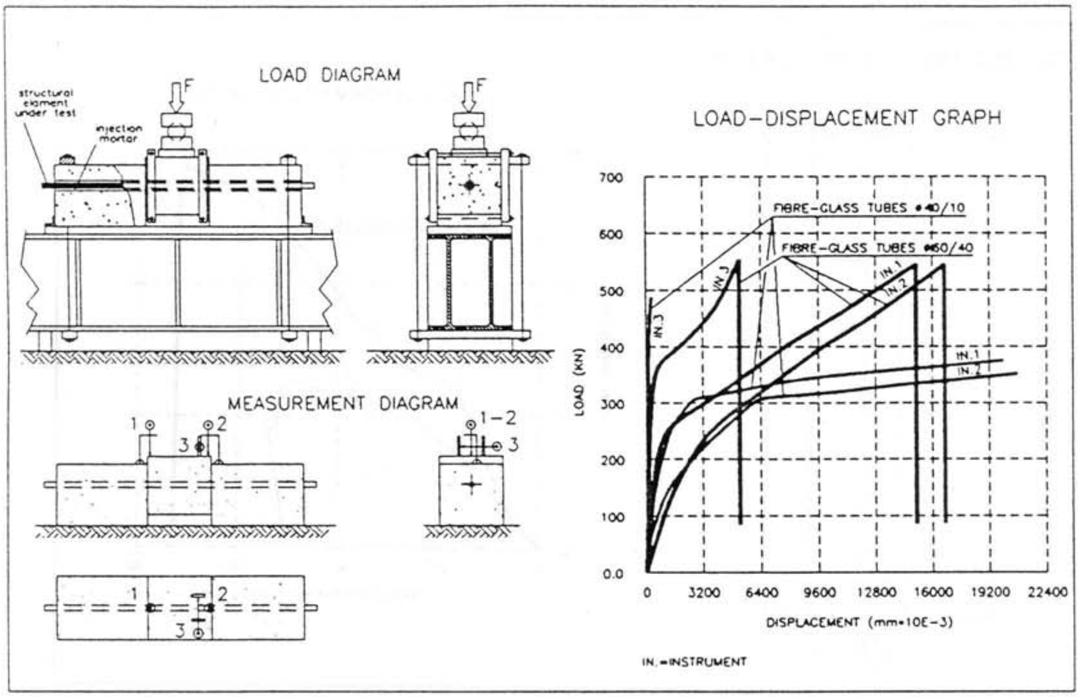


Figure 23 - Shear test

a depth of at least 1 mm. This makes it highly resistant to forces that may push it out of its hole.

As far as the length of the tubes is concerned this will be determined by design requirements (they should not, however, be less than the diameter of the tunnel). Should the tubes be supplied in pieces, they must be joined with fibre-glass sleeves by gluing with epoxy resin as well as by screwing.

The most recent specification standards (Autostrade S.p.A., ANAS, ITALFERR S.p.A.) specify mechanical properties which the fibre-glass must satisfy. A typical example is given below:
- density (UNI 7092) ≥ 1.7 g/cu cm
- resistance to tensile tress (UNI 5819 ≥ 200 MPa
- ultimate elongation = 2%
- resistance to bending stress (UNI 7219 ≥ 350 MPa
- shearing strength (ASTM D732) ≥ 80 MPa

When using tubes or bars, whether smooth surfaced or with improved adherence for specific applications, it is always wise to carry out specific shear, tensile and where necessary burst tests for valved tubes. The results of these tests for a tube of Ø 60/40 mm and Ø 40/10 mm are given in Figures 23, 24 and 25. These tests can provide strength

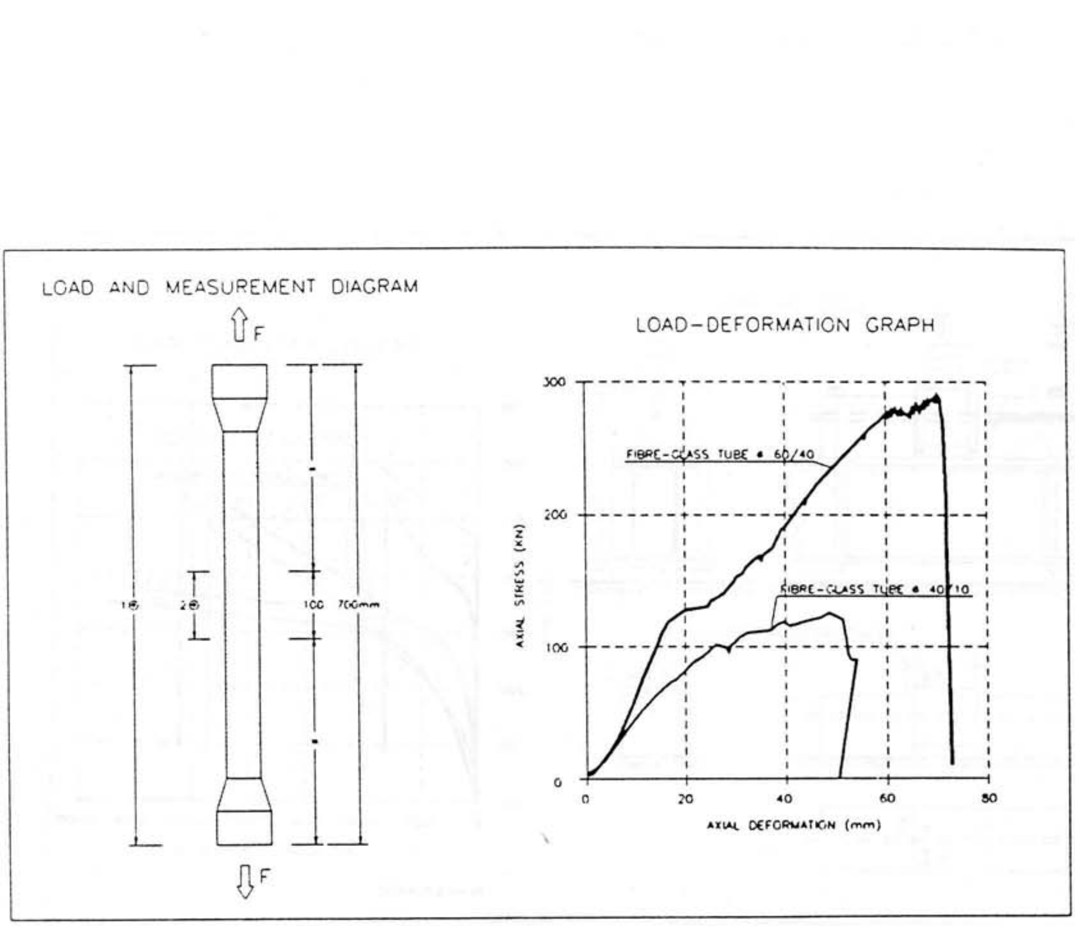


Figure 24 - Tensile test

parameters for direct use by design engineers. The products supplied on site should be checked.

Fibre-glass reinforcement will be supplied with a series of accessories, in accordance with its specific function.

As far as the heads of tubes are concerned they will be fitted with:

- a special plug made of plastic or similar material, cylindrical or conical in shape, fitted with a small breather tube. This is to prevent cement mix from escaping, specially with upwardly inclined tubes;
- an ABS ball valve with a rapid bayonet fitting for the injection pump.

The length, number, diameter and geometrical distribution of the tubes to be inserted in the face of a tunnel are all parameters that concern advance ground improvement.

The depth of the operation (length of the tubes) will be determined by the diameter of the tunnel but also by logistical considerations of limiting the incidence of these operations to standard unit lengths of tunnel on the one hand and by the impossibility of working with excessively long tubes on the other.

The intensity of this improvement (number and diameter of tubes) and geometrical distribution will however be determined by the natural

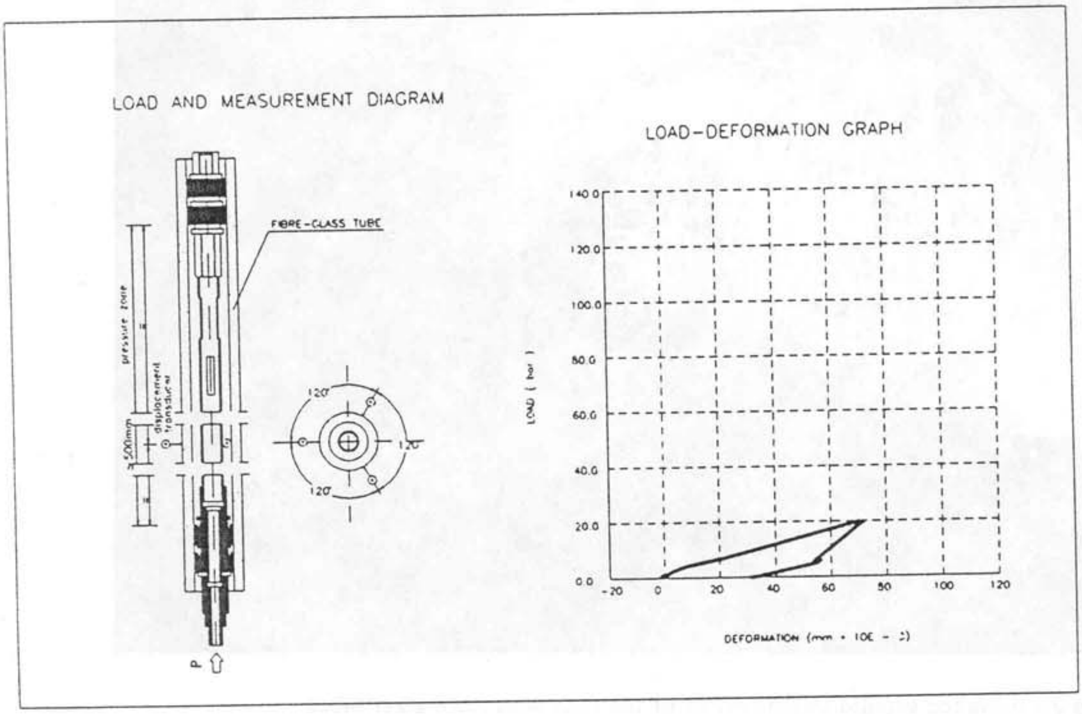


Figure 25 - Burst test

shear strength of the ground, by lithostatic pressures involved and the size of the cross section to be excavated as well as, of course, by the size of the effect that is desired.

The idea of using fibre-glass tubes was determining for the success of this type of advance ground improvement because it combines high strength properties with great fragility. Consequently they can be broken with the same bucket that is used for excavation of the ground.

When, following tunnel advance, the length of the tubes is less than at least one tunnel radius, a fresh series of tubes is inserted.

In addition to inserting fibre-glass tubes it is also advisable to give the face a concave shaped profile so as to help to channel stresses to the sides of the excavation thus further improving safety.

This method can be used in cohesive and semi-cohesive ground and this is generally of a clayey nature. If the method is well designed and carried out, appreciable improvement of the strength and susceptibility to deformation of the ground in the core ahead of the face is obtained and consequently extrusion is considerably contained as is instability (photo 2).

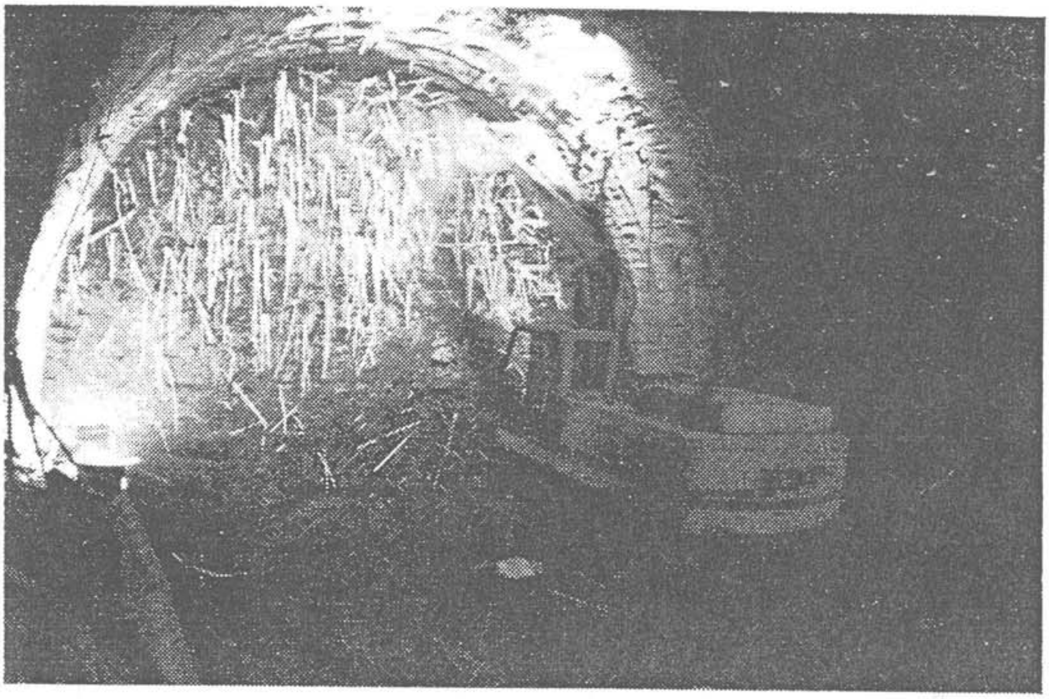


Photo 2 - Advance ground improvement of the face with fibre-glass tubes

This translates into:

- reduced transverse plastic deformation at the tunnel face and therefore contained, if not negligible, convergence (the effect of pre-confinement of the cavity);
- conservation of the intrinsic strength and deformation properties of the ground surrounding the next portion of material to be excavated and therefore considerable reduction of the thrusts, in the short and long term, on the primary and secondary linings;
- stiffening of the ground in the core and therefore zero or considerably reduced extrusion of the face.

3.3.2 Dimensioning of the operation and numerical verification with a mathematical model

The stress-strain method using «characteristic lines», already used in the diagnosis phase (§3.2), can be used in cases of homogeneous ground, a circular tunnel and a natural stress state of the ground of hydrostatic type (valid for tunnels at considerable depth). Its purpose is to assess the *pre-confinement action* to be applied and therefore the

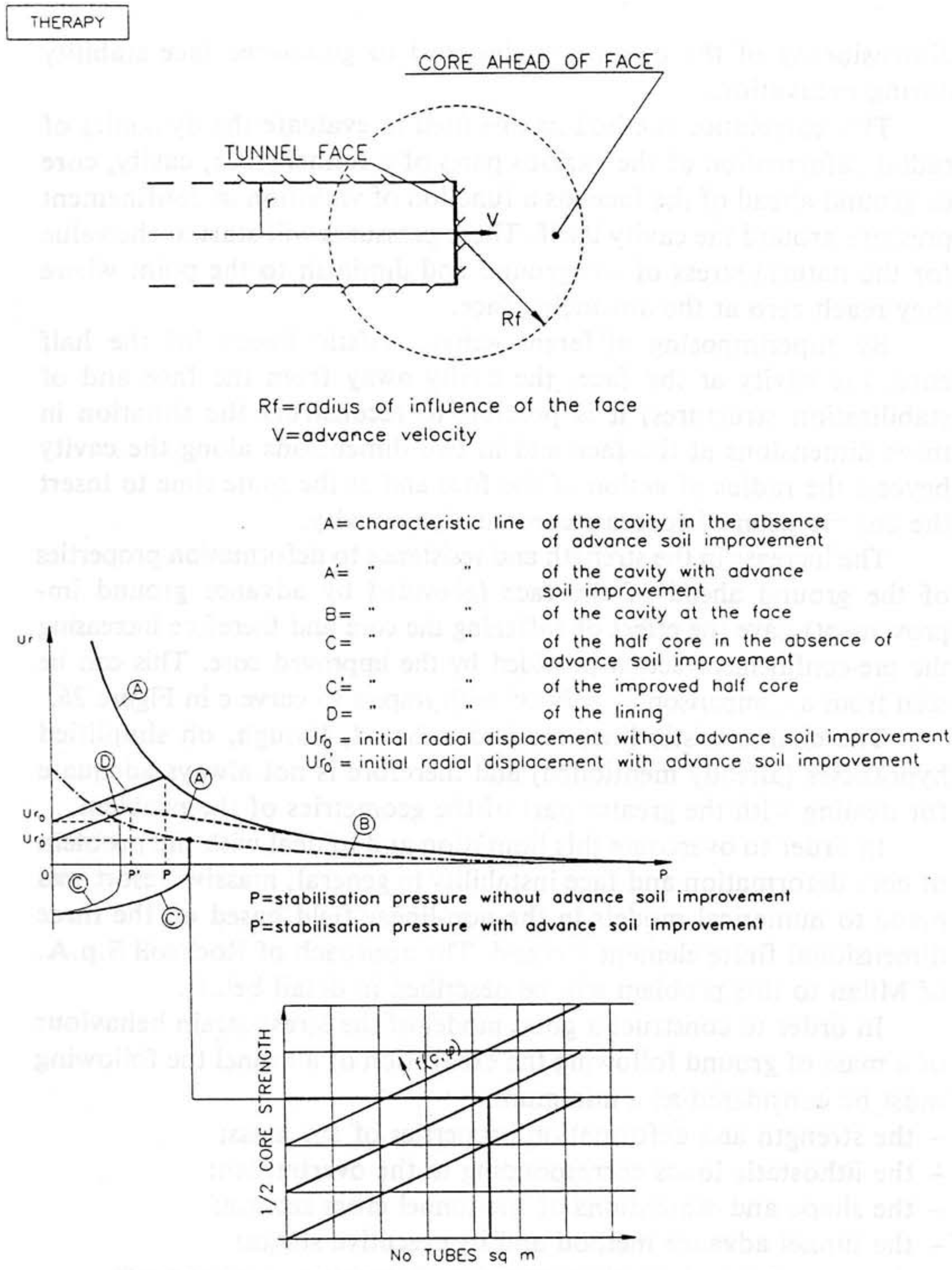


Figure 26 - **Advance ground improvement of the core by means of fibre-glass tubes at the face: the effects**

dimensioning of the operations designed to guarantee face stability during excavation.

This calculation method can be used to evaluate the dynamics of radial deformation of the various parts of a tunnel (face, cavity, core of ground ahead of the face) as a function of variation in confinement pressure around the cavity itself. These pressures will start at the value for the natural stress of the ground and diminish to the point where they reach zero at the advancing face.

By superimposing different «characteristic lines» (of the half core, the cavity at the face, the cavity away from the face and of stabilisation structures) it is possible to reconstruct the situation in three dimensions at the face and in two dimensions along the cavity beyond the radius of action of the face and at the same time to insert the contribution of forecast construction activity.

The increase in the strength and resistance to deformation properties of the ground ahead of the face (provided by advance ground improvement) have the effect of stiffening the core and therefore increasing the pre-confinement action provided by the improved core. This can be seen from a comparison of curve c' with respect to curve c in Figure 26.

The characteristic lines method is based, though, on simplified hypotheses (already mentioned) and therefore is not always adequate for dealing with the greater part of the geometries of the problem.

In order to overcome this limitation and to deal with the problem of core deformation and face instability in general, massive resort was made to numerical models in the non-linear field based on the three dimensional finite element method. The approach of Rocksoil S.p.A. of Milan to this problem will be described in detail below.

In order to construct a good model of the stress-strain behaviour of a mass of ground following the excavation of a tunnel the following must be considered as a minimum:

- the strength and deformation properties of the mass;
- the lithostatic loads corresponding to the overburden;
- the shape and dimensions of the tunnel cross section;
- the tunnel advance method and its executive stages;
- the geometry and elastic properties of structural components.

The calculation system that gives the best model of the effect of the tunnel advance method - of pre-confinement and confinement operations - on the development of the stress-strain state of the face and the cavity of a tunnel is three dimensional finite element modelling in the elastic-plastic range.

A three dimensional model is needed to simulate the three dimensional nature of the problem of face advance for the reason that this cannot be evaluated adequately with plane models.

With this method it is possible to follow the activation of the various structural components (fibre-glass tubes at the face, primary lining, secondary lining) and the disactivation of the ground as various calculations are run on the computer. Consequently the development of the stress-strain states of the ground both ahead of the face and around the tunnel is obtained.

Given the importance of the strength and deformation properties of the mass for the reliability of the results of the finite element modelling, a series of integrated theoretical and experimental systems were developed (described in §3.2). These made it possible to rate the parameters obtained from laboratory tests with double physical and mathematical modelling of the extrusion that occurs following the excavation of a cavity in clayey ground (Fig. 27).

The parameters obtained from the rating are then inserted in the three dimensional finite element model of the tunnel and the stress-strain states of the various structural components and the ground surrounding the tunnel are obtained for all stages of face advance.

3.4 Monitoring during construction work

The stability of the face is monitored during construction of the tunnel by siting appropriate survey stations along the route. Figure 28 describes the following measurements:

1) measurements of convergence of the cavity in the vicinity of the face using distance measurement spuds positioned as illustrated in diagram **1**;
2) measurements of extrusion in the ground ahead of the face are made using an incremental subsidence metre positioned at the centre of the cross section the perimeter of which coincides with that of the walls of the advancing tunnel, as can be seen from the topographical plans. These are used to evaluate the extension of the radius of influence of the face;
3) subsidence measurements of the mass above the cavity, to provide a complete map of the stress-strain state of the ground surrounding the tunnel before, during and after the passage of the face along the surveyed section.

Advance ground improvement of the core ahead of the face using fibre-glass tubes in combination with mechanical pre-cutting was first

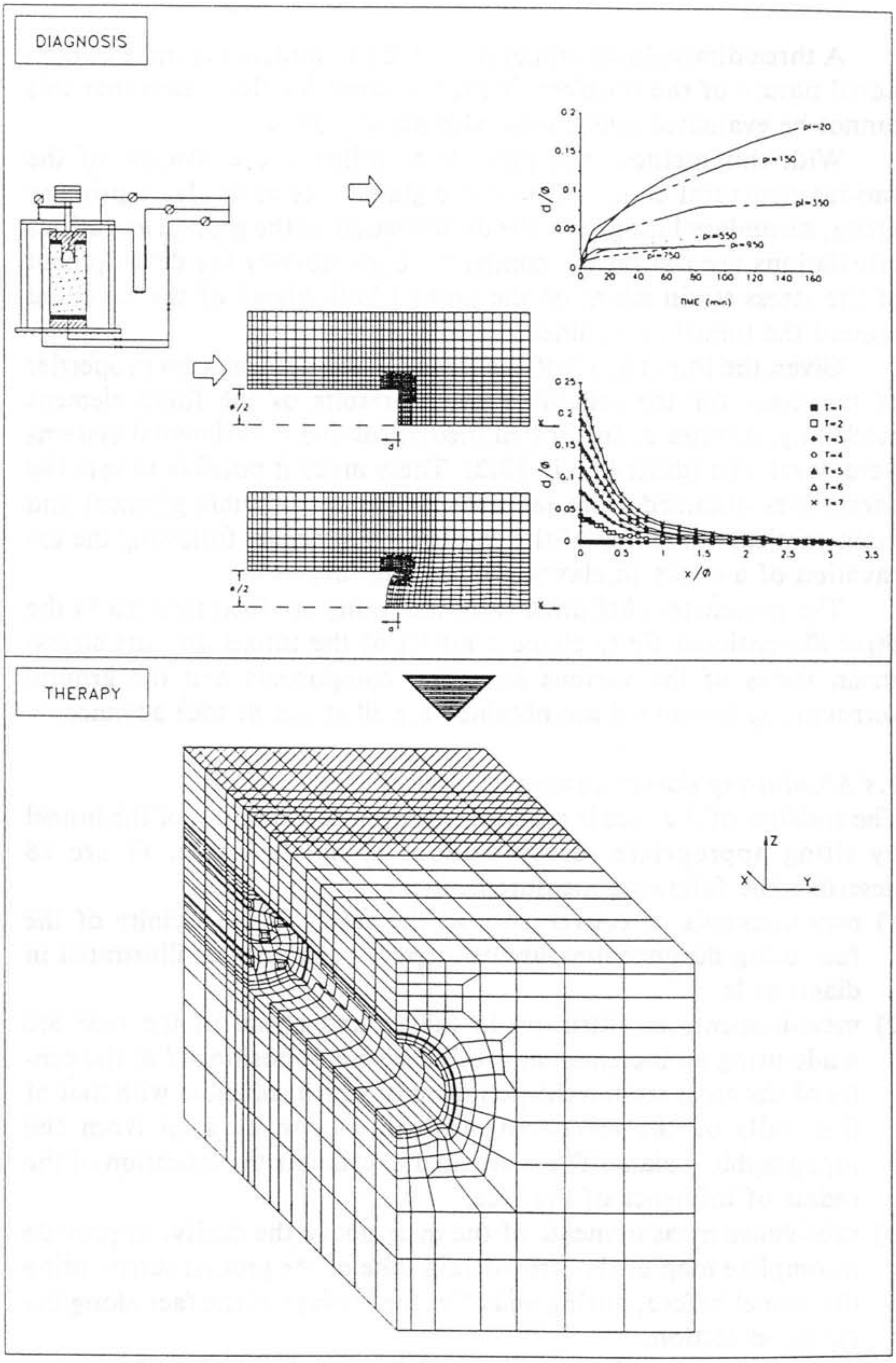


Figure 27 - Three dimensional finite element model

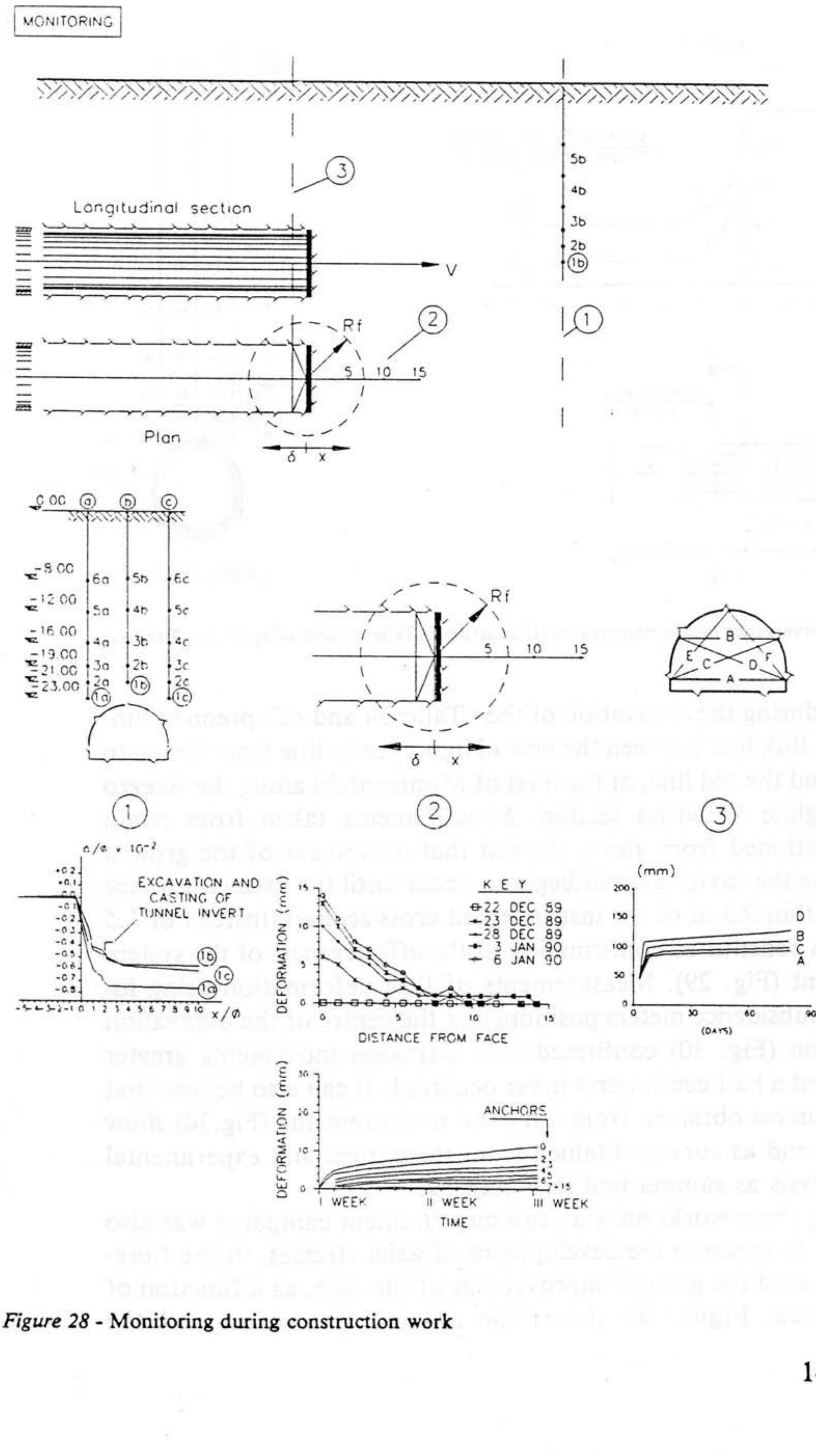


Figure 28 - **Monitoring during construction work**

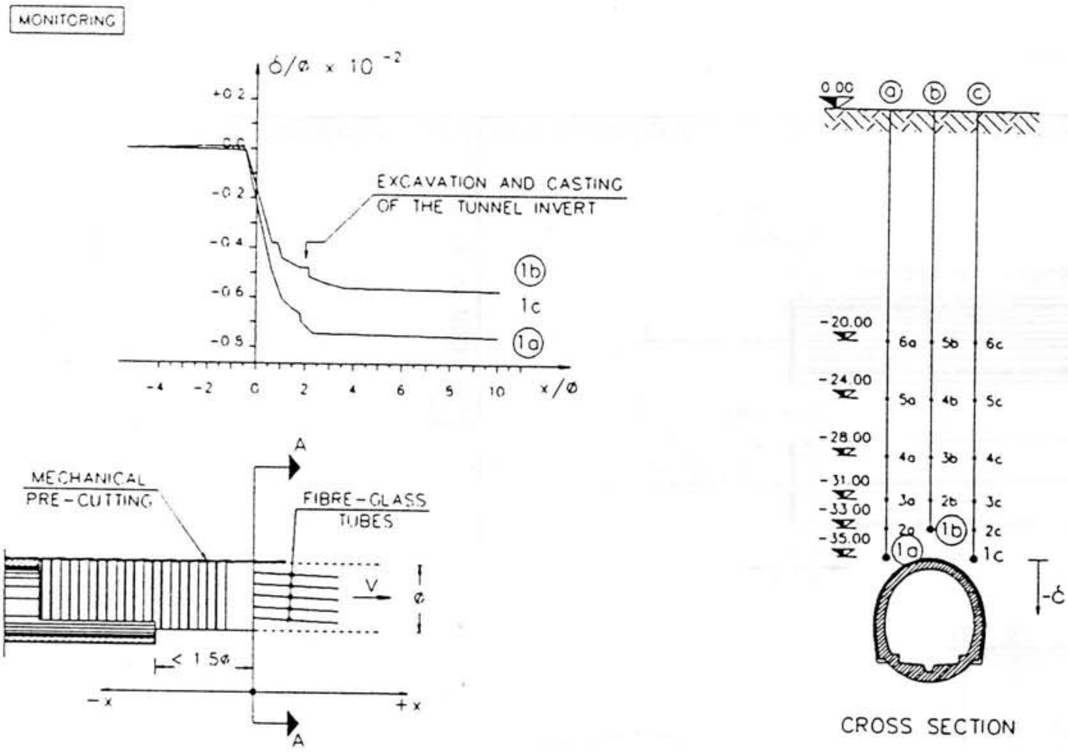


Figure 29 - Convergence measurements from the surface («Talleto» and «Caprenne» Tunnels)

employed during the excavation of the «Talleto» and «Caprenne» tunnels on the link line between the new «High Speed» line from Rome to Florence and the old line, at the level of Montevarchi along the Arezzo Sud to Figline Valdarno section. Measurements taken from strain gauges positioned from above showed that movement of the ground surrounding the cavity did not begin to occur until the excavation face came to within 0.5 m of the instrumented cross section (instead of 1.5 m) and this constituted confirmation of the effectiveness of the system of treatment (Fig. 29). Measurements of face deformation using incremental subsidence meters positioned at the centre of the excavation cross section (Fig. 30) confirmed that extrusion movements greater than one and a half centimetres never occurred. It can also be seen that extrusion curves obtained from «in situ» measurements (Fig.30) show the same trend as curves obtained from theoretical and experimental model analysis as summarised in Figure 19.

During these works an «in situ» measurement campaign was also carried out to measure the development of axial stresses, in the fibre-glass tubes used for ground improvement of the face, as a function of face advance. Figure 31 shows the axial deformation and the

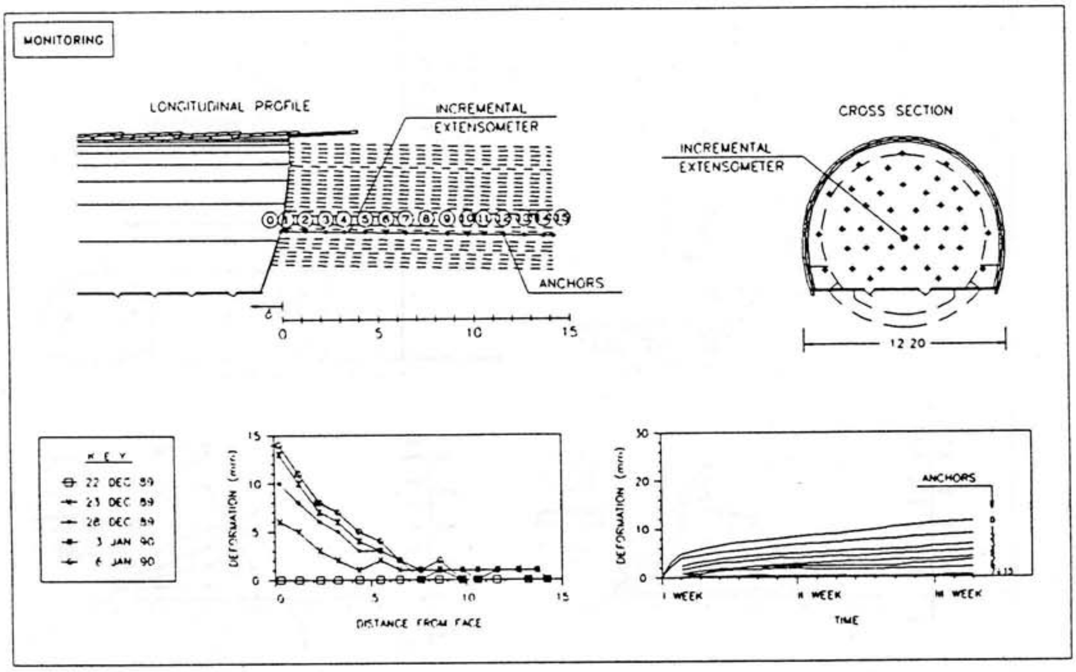


Figure 30 - Measurements of deformation at the tunnel face («Talleto» and «Caprenne» Tunnels)

corresponding stresses as a function of the distance of measurement marks on the tubes from the face and also as a function of the position of the face itself. The measurements were carried out using strain gauges.

4. THE SAN VITALE TUNNEL

The «San Vitale» tunnel is 4,200 m in length and forms part of the project for converting the Caserta to Foggia railway line from single to twin track and generally improving the line. The tunnel lies on the outskirts of Benevento between the Apice and Vitulano stations. The tunnel passes through ground belonging to two lithostratigraphical formations: the «Unità di Altavilla» and the «Unità delle Argille Varicolori» (cf.. profile in fig. 32).

The «Unità di Altavilla» has been subject to medium tectonisation and consists of alternating levels of sands at times only poorly cemented and levels of clays, marly and silty clays.

The underlying «Unità delle Argille Varicolori» (mottled clays unit) consists of two different lithotypes: one mainly clayey-marly and the other predominantly calcareous-marly. Both had been subject to intense tectonisation giving them a scaly, disordered and chaotic struc-

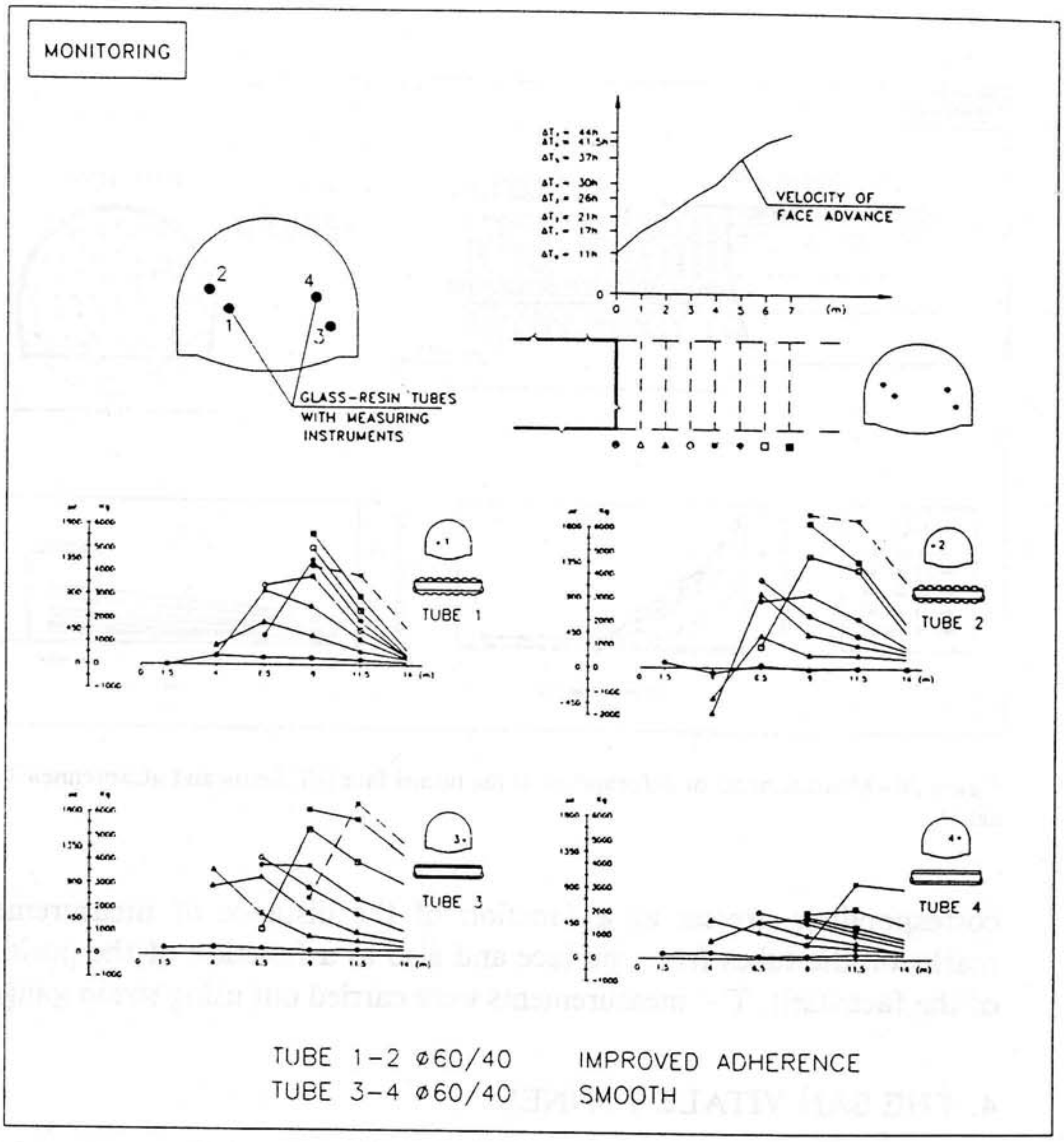


Figure 31 - In situ measurements of the stress-strain behaviour of the fibre-glass tubes

ture making them extremely subject to disaggregation and giving rise to minute fracturing of the most consistent lithoid elements. These are now found in the form of isolated masses immersed in a clayey or marly clay matrix and not in continuous strata as they probably were originally.

Photograph 3 illustrates this ground very well. It is reduced to minute fragments that tend to make it behave like a loose mass, with curls of stony material folded by tectonic forces and dispersed in the clayey matrix. The scaliness of this ranges from metric to millimetric making it extremely sensitive to the action of humidity in the air.

Photo 3 - San Vitale Tunnel: ground at the face (Argille Varicolori)

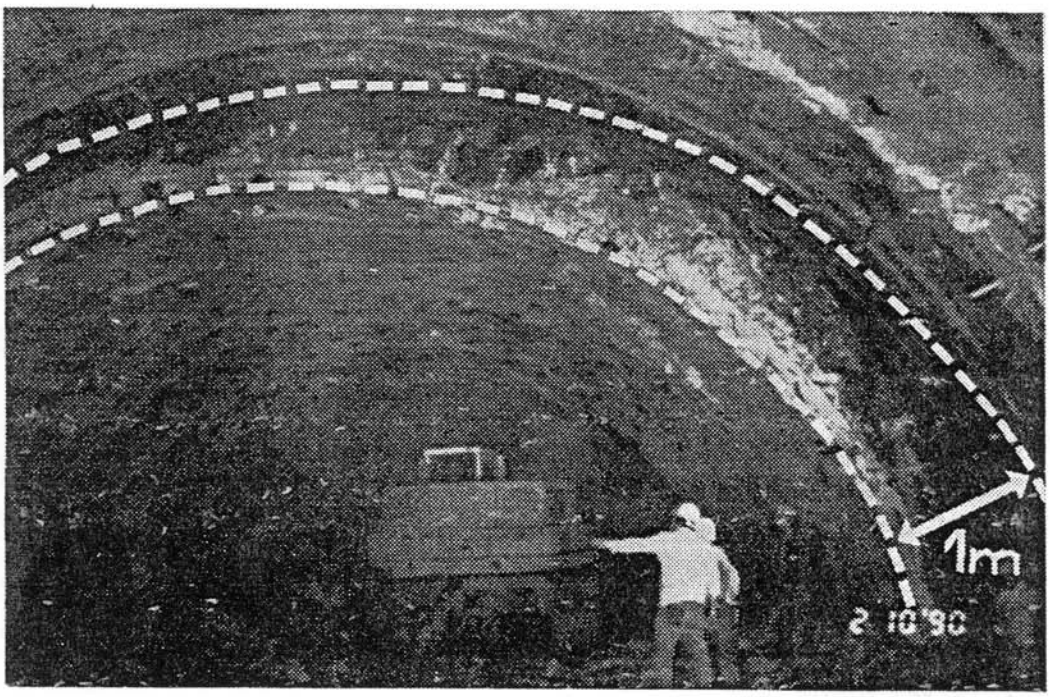


Photo 4 - San Vitale Tunnel: phenomena of convergence of the cavity during partial face excavation

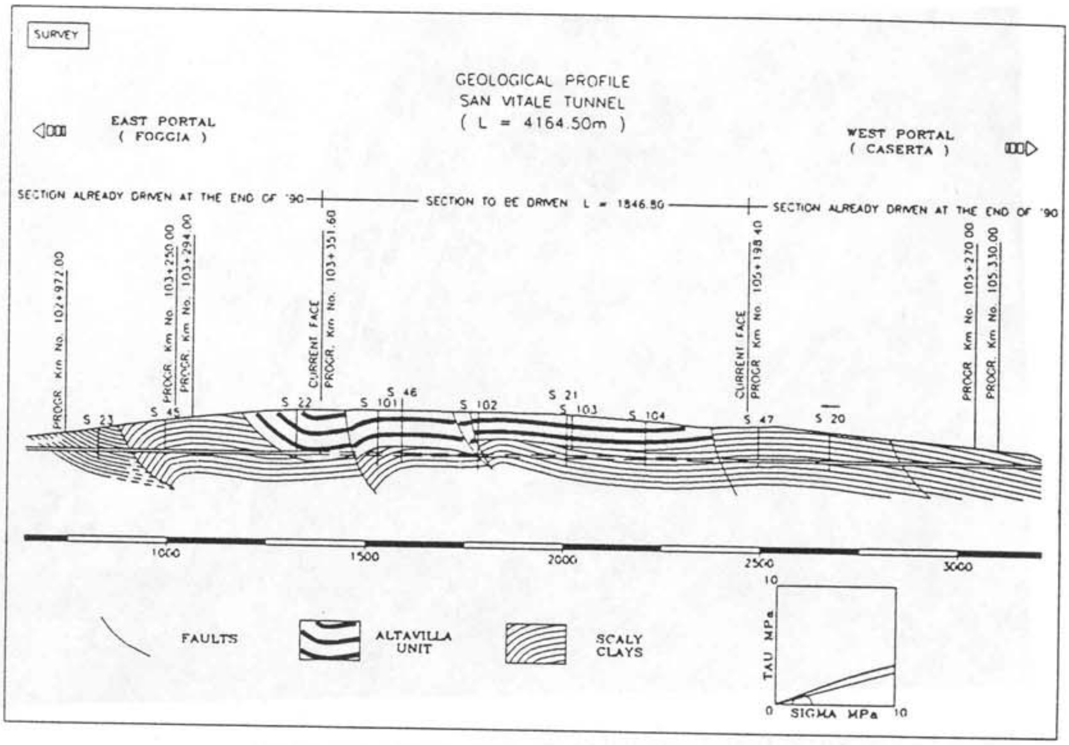


Figure 32 - San Vitale Tunnel: geological profile

The construction of the tunnel carried out according to the dictates of the New Austrian Method, was driven from both ends until it met the «Argille Varicolori». Here enormous problems were encountered all of which could be brought down to instability of the cutting face which, despite partial face excavation as prescribed for class Vb ground (cf.. Fig. 33), extruded tremendously causing convergence of the cavity of up to a metre and more (cf.. photo 4).

Attempts had been made to prevent collapse of the face by ground improvement with fibre-glass tubes, but the operation had not had the desired effect as it was wrongly applied on the half cross section of the crown (the improved core extruded rigidly into the tunnel).

Since all attempts to advance using normal methods were in vain, the contractor, in agreement with the customer, decided call in the consultants Rocksoil S.p.A. of Milan to design the tunnel. They had already dealt with difficult cases in similar types of ground.

4.1 Survey phase

The geotechnical character of the ground was examined employing the usual laboratory tests, the results of which were, however,

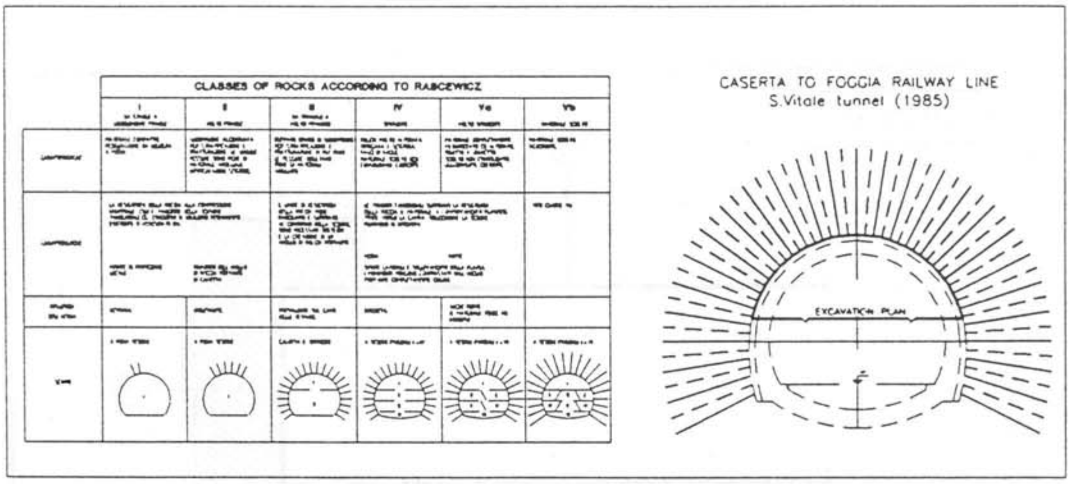


Figure 33 - San Vitale Tunnel: partial face excavation by NATM

heavily affected by the extremely uneven nature of the clayey matrix. This examination made it possible to construct the intrinsic curve of the mass (fig. 32).

Various extrusion tests furnished extremely small stability factors that predicted the appearance of extrusion phenomena with overburdens as shallow as 20 m.

The tectonic action to which they had been subjected made it impossible to calculate the natural stress state; there was no precise and continuous tensor of the force; each packet of ground possessed its own stress state which therefore varied continuously along the profile of the excavation.

4.2 Diagnosis phase

In the diagnosis phase, study of the characteristic lines showed that, in the absence of any intervention to regulate deformation phenomena, the face was of the unstable type (fig. 34). The situation appeared further compromised by the interstitial pressures which produced rapid changes in the water content of the clay on loosening of the ground with a consequent drop in the stability coefficient and extrusion.

4.3 Therapy phase

In the therapy phase, given the nature of the ground to be excavated (decidedly sensitive to variations in humidity as a result of the ability of the ground to absorb water on decompression) it seemed ab-

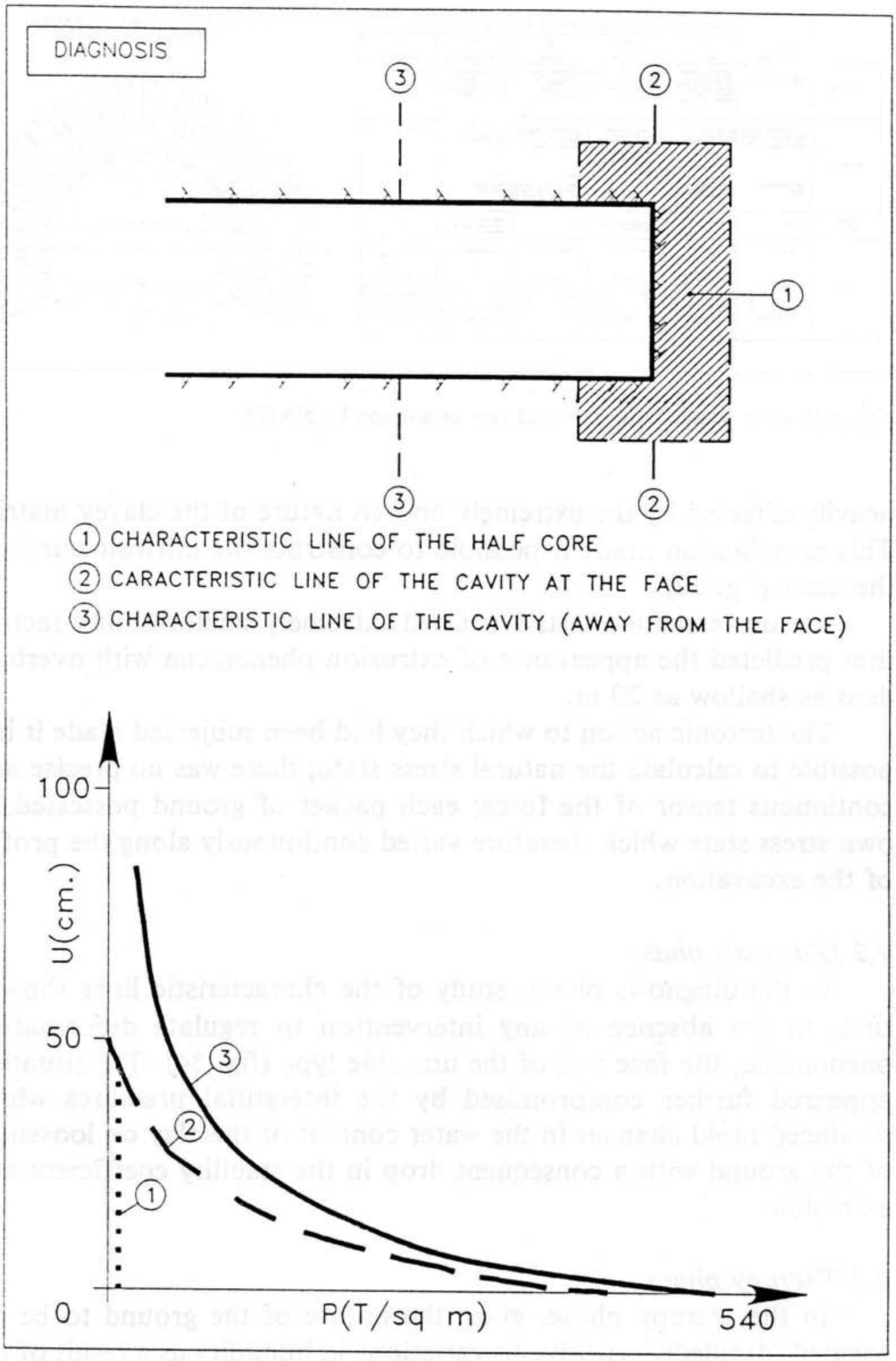


Figure 34 - San Vitale Tunnel: study of the characteristic lines

solutely necessary to act decisively ahead of the face, anticipating and preventing deformation before it could start. If this were done there would be no need to operate in collapsed and completely disaggregated ground incapable of opposing any resistance.

In practice the operations prescribed consisted of mixed type conservation involving improvement of the ground ahead of the face, with fibre-glass tubes with improved adherence and advance improvement of the ground surrounding the tunnel using very thickfibre-glass tubes fitted for recompression injections and sub-horizontal drainage in advance (Fig. 35).

Construction was completed with a primary lining consisting of steel ribbing and steel fibre reinforced shotcrete, a reinforced tunnel invert immediately behind the face and finally the secondary lining in concrete.

A three dimensional finite element analysis in the non linear field was carried out to simulate all the phases of tunnel advance for the design, dimensioning and rating of all these operations. It was also used to define exactly how the rock mass around the tunnel would react to the operations.

The model was designed to calculate movements and stress states in the elastic-plastic field taking into account, as consistently as possible, the real conditions of consolidation and excavation as well as the construction stages, without taking viscous phenomena into account. With this type of model, the lining was introduced, within two and a half diameters from the face, as a stiffening element only (Fig. 36).

The effect of the tubes as a function of the ultimate adherence of the tube-concrete surface, was realistically reproduced. They were attributed values of rigidity that would guarantee their functioning until traction levels were reached that were compatible with those obtained in the experimental tests.

Analysis of the stresses on the tubes and of the extrusion levels of the core after each calculation step of the model checked whether the tubes were effective.

The need to examine a situation under full tunnel construction meant that the preliminary model was calculated through eight steps with the aim of recreating, by approximation (improving each time), the real situation.

The model was calculated through a total of eleven steps, beginning with the calculation of the geostatics («time 1»), during which the various stages of ground improvement, excavation and lining were simulated step by step (Fig. 37).

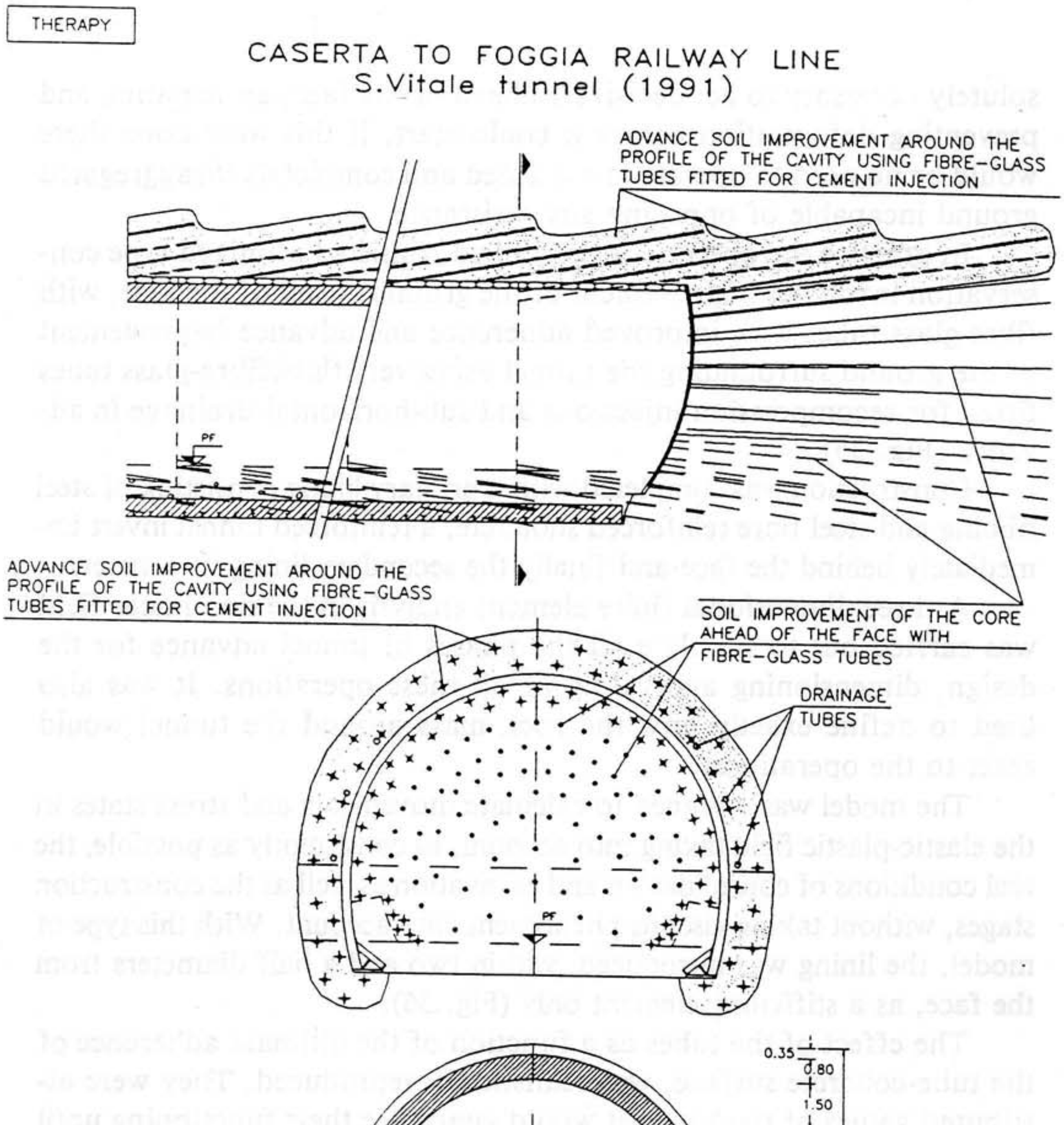


Figure 35 **- San Vitale Tunnel: scheme of the interventions of stabilisation for the full face excavation**

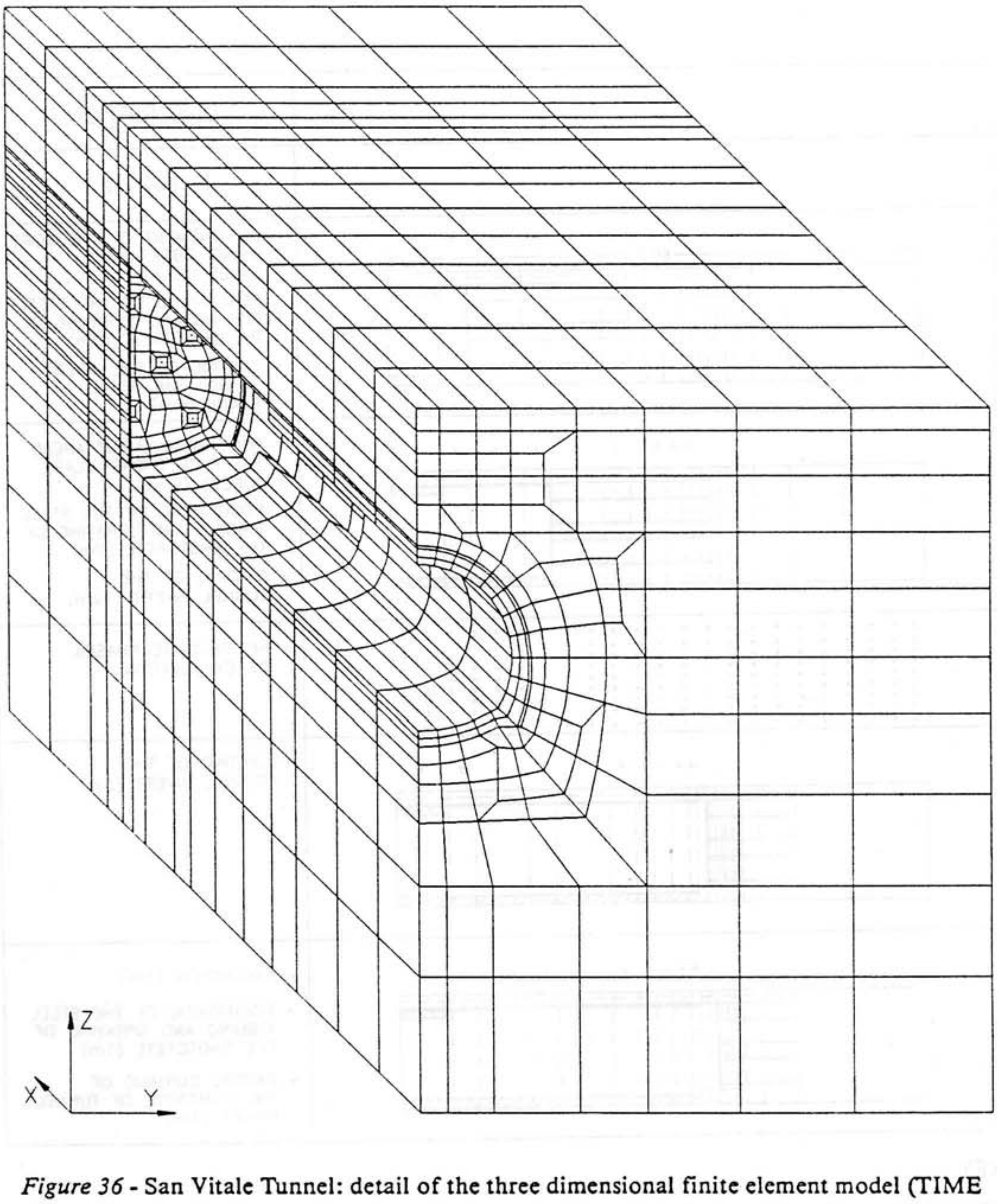


Figure 36 - **San Vitale Tunnel: detail of the three dimensional finite element model (TIME 10)**

The most significant model calculations were the 9th and 10th, representing respectively:

- 9th calculation: laying of the tunnel invert (L = 3 m);
- 10th calculation: excavation of one metre of core + spraying of shotcrete and placing of steel ribbing (L = 1 m) + partial curing of the concrete of the tunnel invert.

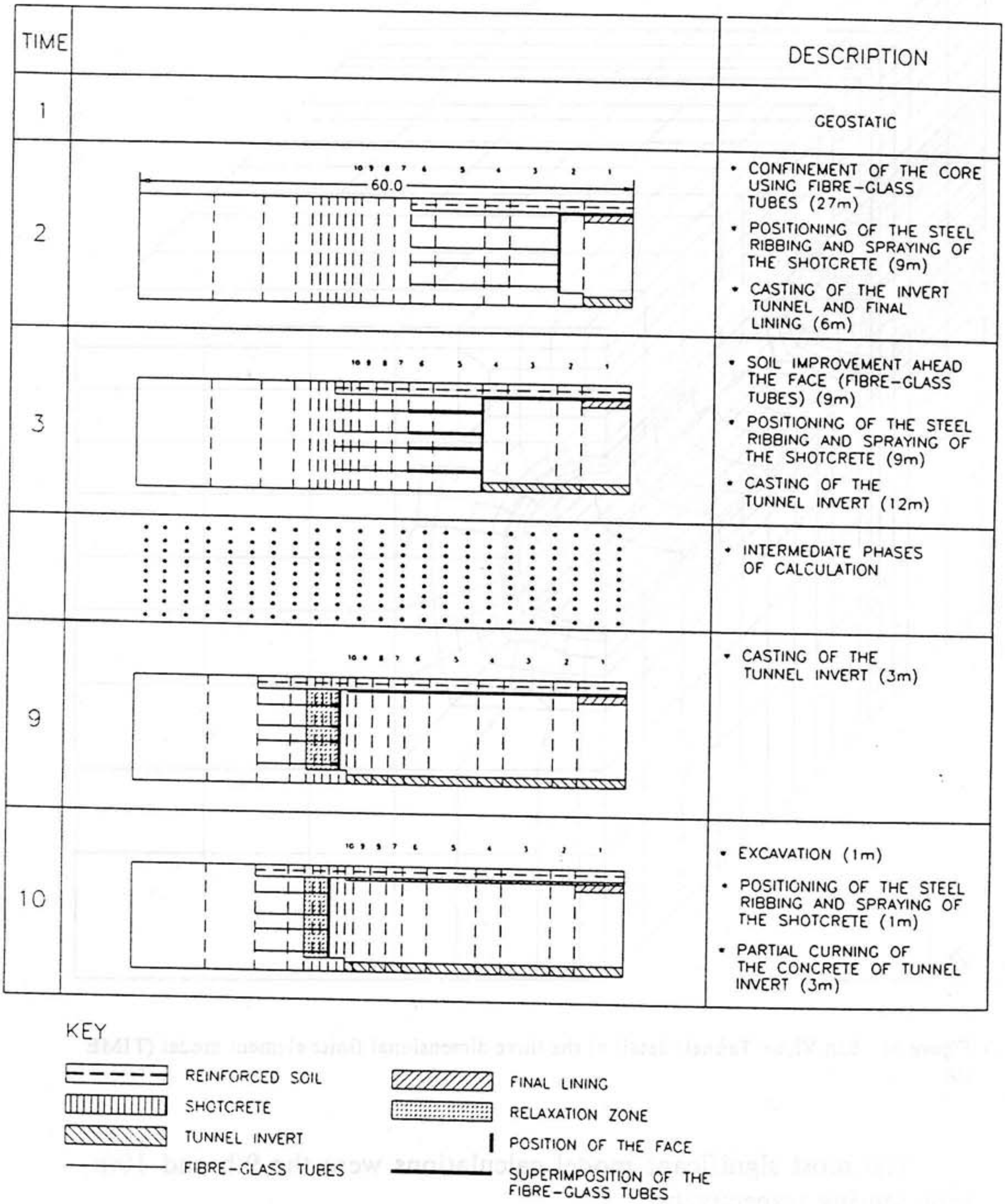


Figure 37 - **San Vitale Tunnel: steps of the FEM calculation**

Settlement at the *apex of the crown* showed a maximum calculated value of little more than 4 cm at a distance of approximately 3 m from the face. The steepness of the curves in the vicinity of the face, especially in the last model calculations, indicated a steep settlement gradient: tunnel advance of one metre produced an increment in settlement equal to 25% of total settlement.

The calculation brought out the importance of containing the face, not only in the upper central area, but around the edges bringing forward, as soon as possible, the activation of the tunnel invert and the primary lining.

The above all went to show that the kinematics activated by the various stages of tunnel advance, in ground with varying mechanical properties, can be controlled with adequate intensity of ground improvement operations. In particular, the simulation suggested that a fall in peak cohesion from 0.7 to 0.5 MPa and of residual cohesion from 0.1 to 0.05 MPa could be dealt with by increasing the density of fibre-glass tubes by approximately 35%.

The most highly loaded tubes were systematically located near the most plasticised zones and these were in the middle of the face at a height near to that of the maximum width of the cross section.

An examination of the results of the computer calculation, that cannot be reproduced here, clearly shows the importance of the function of the primary lining working in parallel with the arch of improved ground around the tunnel, behind it.

The maximum main stresses calculated for the arch of improved ground are around 5.2 MPa as against 2.0 to 2.2 MPa existing in the ground before improvement.

The maximum stress in the shotcrete, calculated for a homogenised cross section, is equal to 20 MPa. The positioning of the main stresses shows that the arch of improved ground around the tunnel is also designed to produce an arch effect.

The tunnel invert in the homogenised cross section presents maximum stresses of around 13 MPa with a marked tendency to concentrate towards the base of the posts.

4.4 Operational phase

Encouraged by the results of the mathematical model, the knowledge gained was then put into practice.

The construction stages are outlined in Figure 38.

After advance ground improvement ahead of the face around the

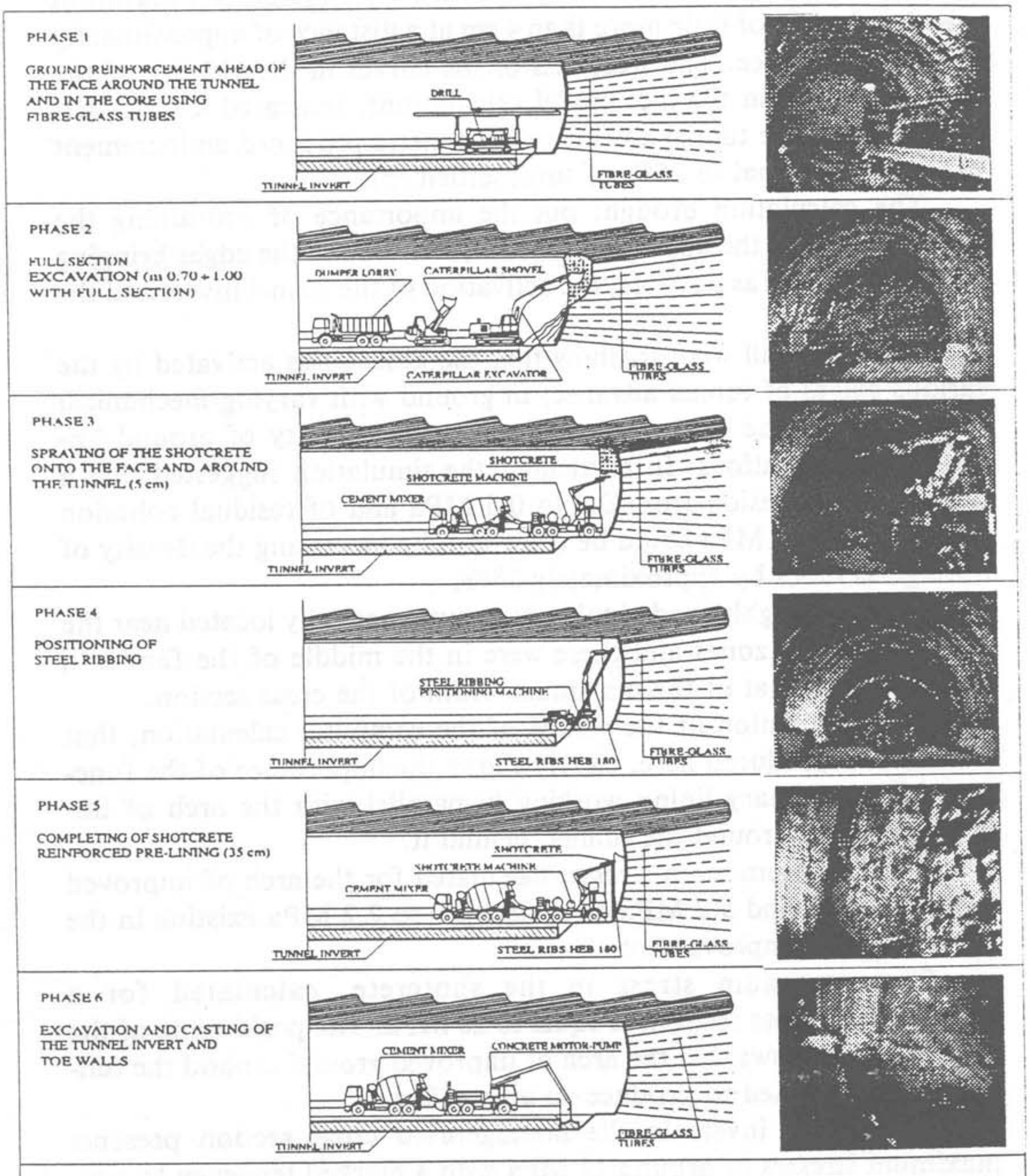


Figure 38 - San Vitale Tunnel: construction stages

tunnel and in the core, excavation was carried out. Positioning of the steel ribbing and spraying of the shotcrete was then repeated four times before closing the section with the tunnel invertand the toe walls to cover the last four ribs.

The advance cycle proceeded for 8 to 10 m of tunnel according to the ground conditions. At this point, having consumed the active part of the ground improvement ahead of the face and around the tunnel, ground improvement operations recommenced again. In order to reduce down times, these operations were carried out at the same time as concrete casting.

By driving from both ends, 471 m of tunnel were completed between October 1992 and April 1993 (191 working days giving an average of approximately 2.2 metres per day with peaks of 3 m). By further reducing down times and improving the already high level of efficiency of the site, it is expected to achieve, during the course of the year, advance rates of around 50 m/month at each face.

4.5 Construction monitoring phase

The behaviour of the tunnel is constantly monitored not just for safety but also for refining operational and design decisions. A total of 1,900 measurements were obtained from 640 m of tunnel. The results of the measurements, compared with those of the theoretical model, made it possible to reliably check the appropriateness of the design and operational decisions.

These measurements included systematic convergence measurements, which showed average convergence of 7 cm for the section of ground analogous to that considered in the finite element model. It is interesting to examine the development of convergence over a complete work cycle. From Figure 39 it can be seen that the cluster of convergence measurements tends to increase for the last few steel ribs of the cycle, when the end of the improved ground is being reached and the quality of the ground is poorer. This clearly shows the influence of the stiffness of the core on the subsequent deformation behaviour of the cavity.

It can be clearly seen that each casting of the tunnel invert completely stabilises the convergence of the steel ribs that it covers (and that is why they must be cast at the face!).

It is interesting to compare, in Figure 40, the course of convergence as measured in October 1990, with half cross section tunnel advance, steel ribbing and shotcrete, and the same convergence measured in

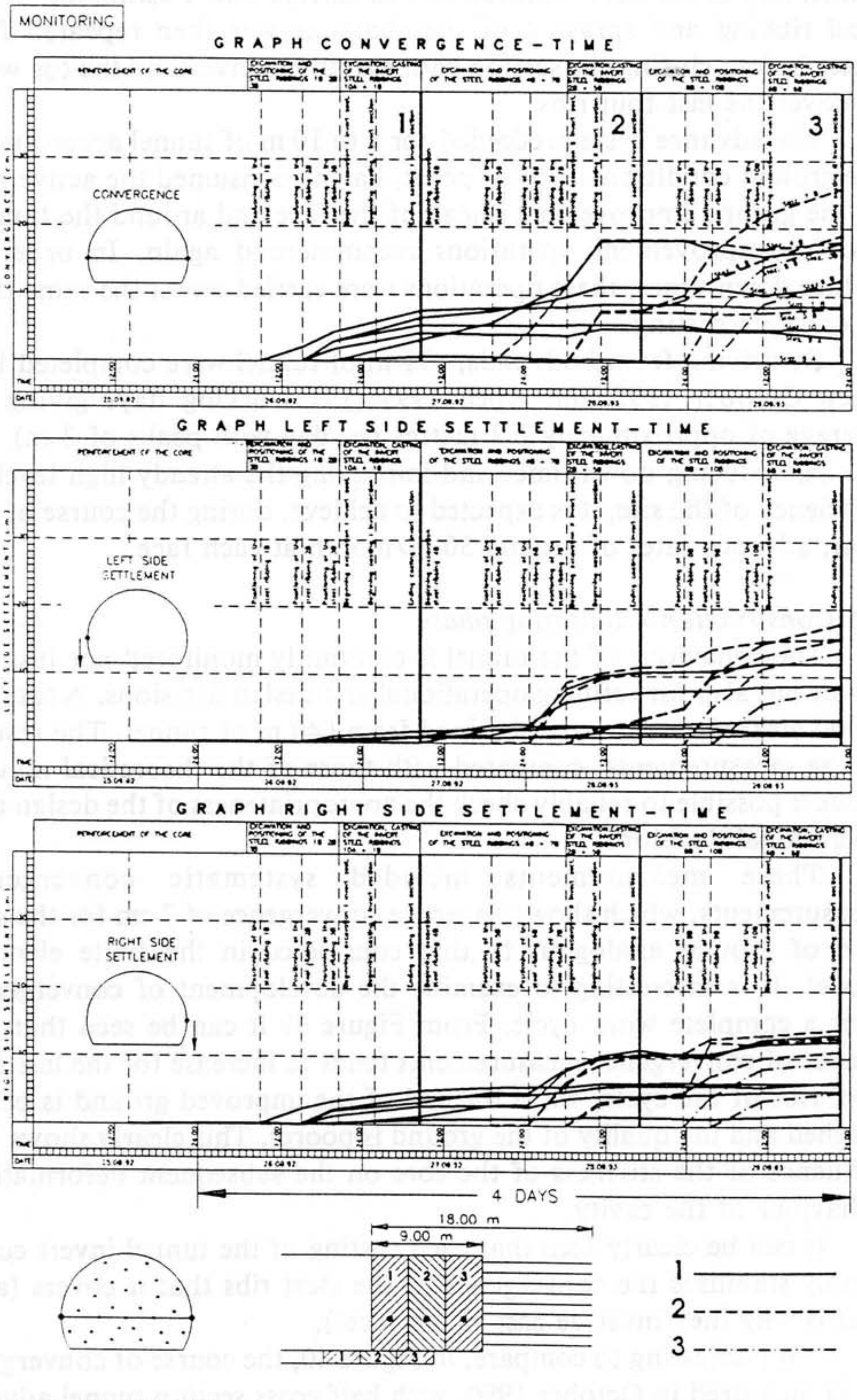


Figure 39 - **San Vitale Tunnel: measures of convergence and settlement**

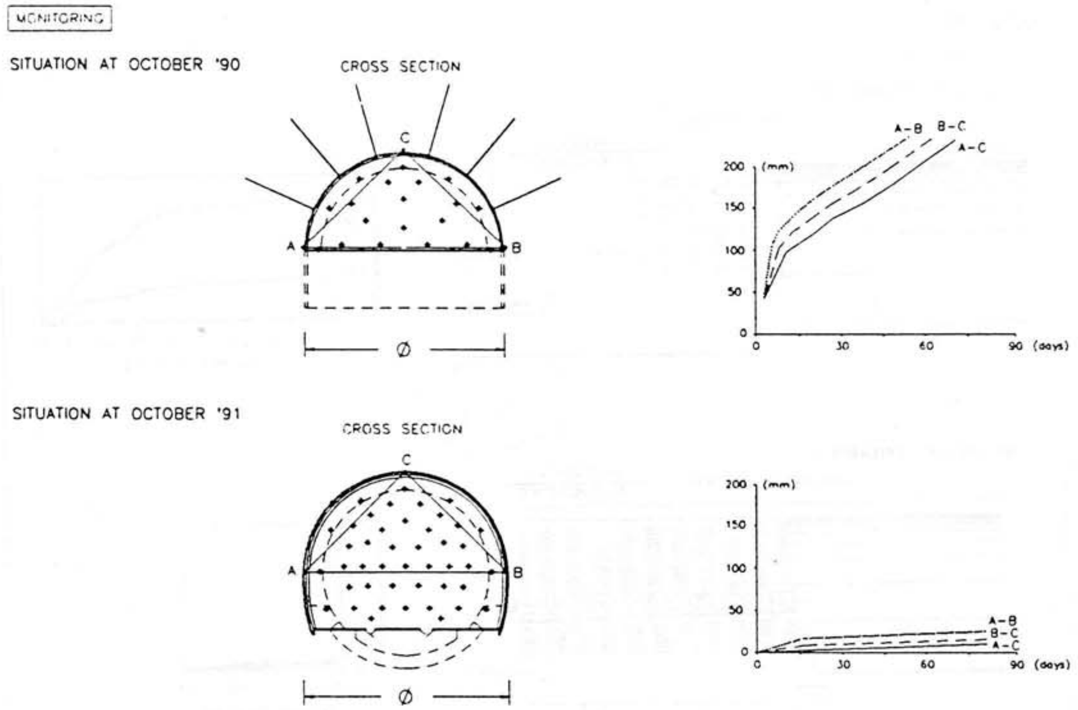


Figure 40 - San Vitale Tunnel: comparison between the course of convergences measured in October 1990 and in October 1991

October 1991 with full face advance using ground conservation methods.

Before, following the triggering of extensive plasticisation of the ground at the face and around the excavation, the convergence practically never settled down with the negative consequences already described. Today, on the other hand, the convergence is considerably reduced and above all it tends towards stability.

In addition to convergence, extrusion measurements are taken by inserting incremental subsidence meters into the core ahead of the face.

Figure 41 shows how in October 1990, the whole parcel of improved ground slid in a rigid lump towards the inside of the tunnel giving rise to average face extrusion of 20 cm. With the new ground treatment this extrusion does not exceed 2 cm.

When comparing these results with those obtained from the mathematical model, it is essential to keep in mind the fact that the model did not take into consideration viscous deformation: it should therefore not be surprising that it provided convergence values equal to approximately one third of those actually measured on site. Such a dif-

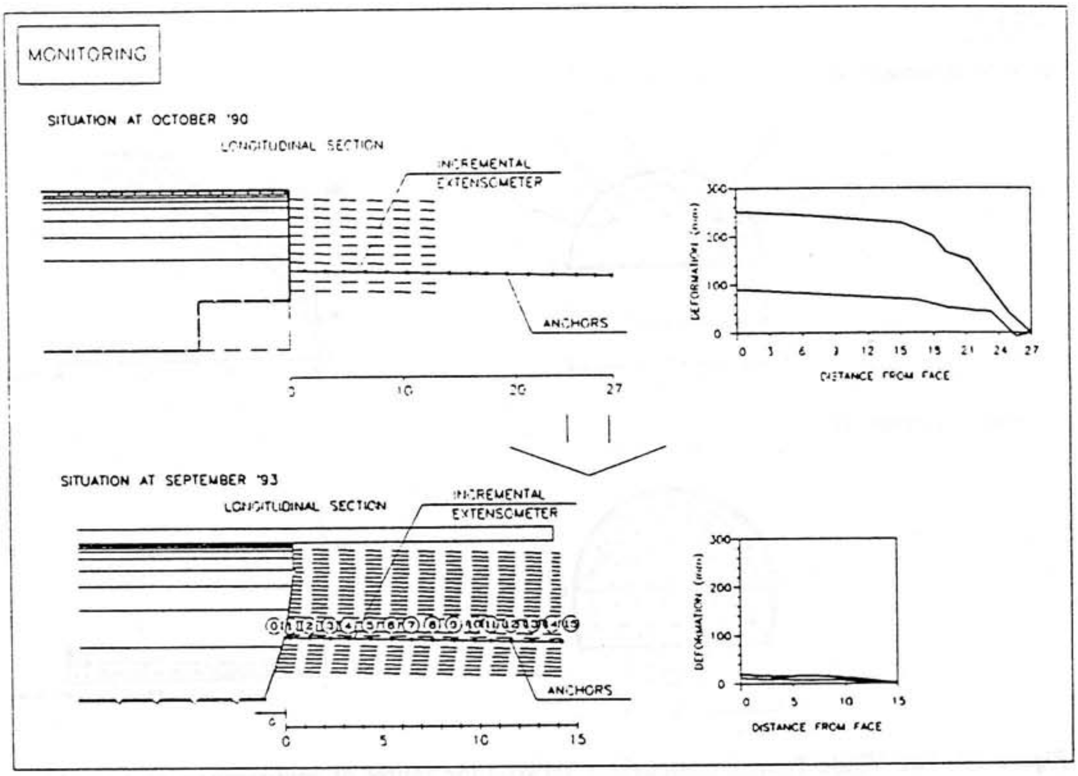


Figure 41 - San Vitale Tunnel: comparison between the course of extrusions measured in October 1990 and in October 1991

ference is not seen with core extrusion: comparison seems to indicate that the fibre-glass tubes, by their nature not subject to considerable viscous phenomena, manage to maintain the face, by their interaction with the ground, within a range of movement that can be classified as elastic-plastic.

It should be noted that the data obtained from the readings relates to better ground than that considered in the mathematical model.

Finally systematic measurements were taken of the contact pressure between the ground and the steel ribs (both radially and beneath the feet of the ribs themselves). The values obtained were equal to 50 bar and 10 to 20 bar under the feet of the ribs and in a radial direction respectively.

Figure 42 shows a comparison of production rates obtained with the two methods of advance employed. It can be seen that with the New Austrian Tunnelling Method average advance, while advance was possible, was 5 m/month as against the current 45 m/month.

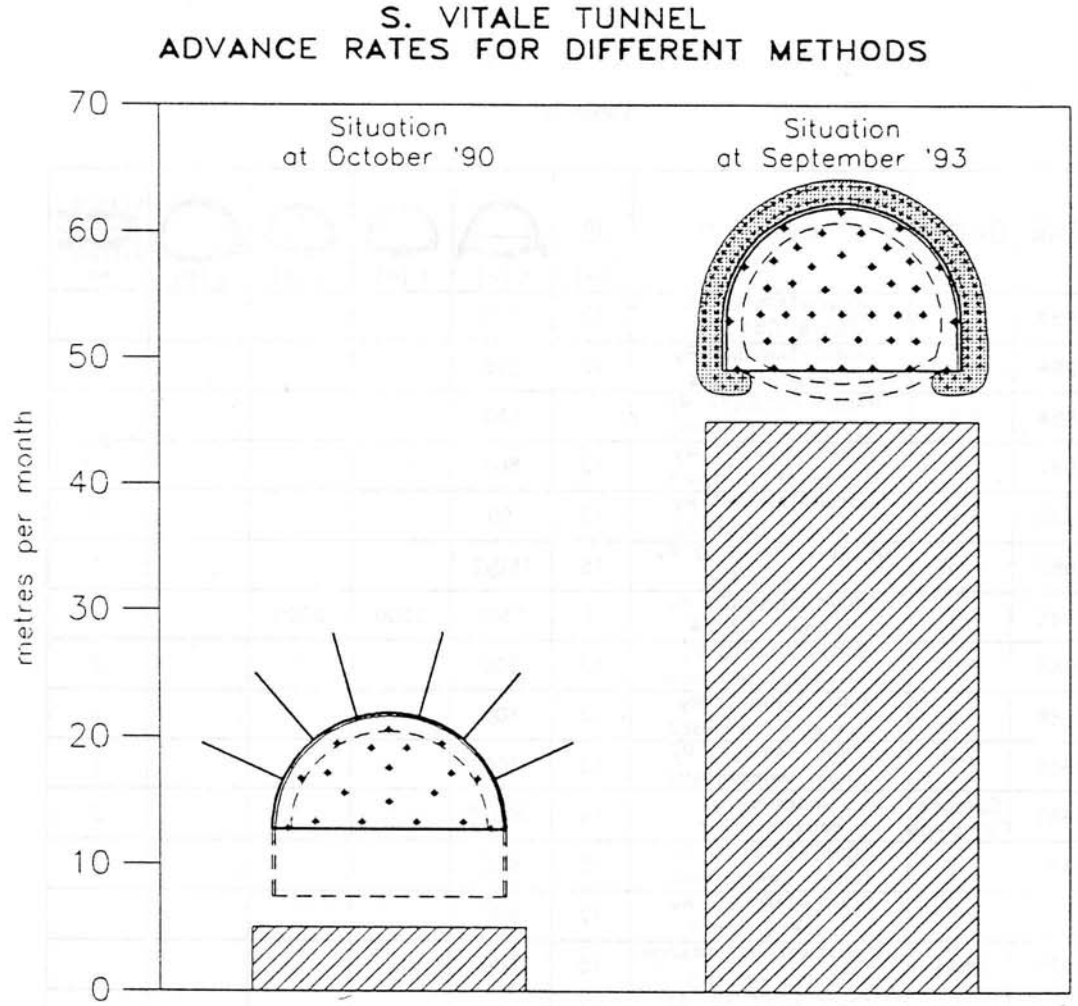


Figure 42 **- San Vitale Tunnel: comparison between the production rates obtained in October 1990 and in September 1993**

5. Conclusions

The results obtained show why conservation methods, and advance improvement of the core using fibre-glass tubes in particular, is finding vast application in the field of underground construction: because for the first time they make it possible to tunnel in the most difficult ground with the same advance rates and the same safety as can be achieved with tunnels in ground with good consistency.

With the introduction of conservation methods, we now possess the construction instruments necessary to operate successfully and effectively in all types of ground and finally it is possible to plan underground construction in same way as has been done for years with surface construction.

Table 1.

YEAR	OWNER	WORK	Ø [m]	L [m]	L [m]	L [m]	L [m]	No
1983	F.S.	UDINE-TARVISIO Ry. Tunnel Campiolo	12	170				
1984	F.S.	UDINE-TARVISIO Ry. Tunnel M. Palis	12	200				
1984	F.S.	PAOLA-COSENZA Ry. Tunnel Santomarco	7	150				
1984	F.S.	COMO-CHIASSO Ry. Tunnel M. Olimpino	12	800				
1985	F.S.	UDINE-TARVISIO Ry. Tunnel S.Leopoldo	12	50				1
1985	ANAS	MESSINA-PALERMO Ry. Tunnel S.Elia	15	150x2				1
1985	F.S.	SIBARI-COSENZA Ry Tunnels 1,2,3,4	10	1300	2300	2300		
1986	F.S.	BARI-TARANTO Ry. Tunn. Mad. del Carm.	12	200				2
1986	F.S.	BARI-TARANTO Ry. Tunnel S.Francesco	12	100				2
1986	F.S.	UDINE-TARVISIO Ry. Tunnel Malborghetto	12	150				1
1987	Comune Campinas	Underway of goods-station	14	300x2				2
1987	F.S.	UDINE-TARVISIO Ry. Tunnel Camporosso	12	650				2
1987	F.S.	UDINE-TARVISIO Ry. Tunnel S.Rocco	12	600				
1988	ANAS	AUTOSTRADA DEI TRAFORI Tunnel Valsesia	15	600				
1988	ANAS	STATE ROAD No 42 Tunnel Lovere	11	200				
1988	MM	MILAN RAILWAY LINK "Venezia" station	30				250	
1988	F.S.	ROME-FI Ry. H.S. Tunn. Talleto e Caprenne	8	2500	2800	2800		4
1988	F.S.	ROME-FI Ry. H.S. Tunnel Tasso	12	150		1650		2
1988	F.S.	ROME-FI Ry. H.S. Tunn. Terranova le Ville	12	200	1800	2200		3
1988	F.S.	ROME-FI Ry. H.S. Tunnel Poggio Orlandi	13	250		600		2
1988	F.S.	ROME-FI Ry. H.S. Tunnel Crepacuore	13	60		120		2
1989	ANAS	CATANZARO RING-ROAD Tunnel S.Giovanni	12		400	400		2
1989	ANAS	STATE ROAD No 42 Tunn. Costa Volpino	11			200		1
1989	Acqued Pugliese	Ofanto aqueduct	5			200		3
1989	F.S.	TARGIA-SIRACUSA Ry. Tunnel di Siracusa	12		1000	1000		
1989	R.A.V.	AOSTA-Mt. BLANC HIGHW. Tunnel Villeneuve	12	275x2				2

TUNNELS DRIVEN USING CONSERVATION METHOD AND DESIGNED BY ROCKSOIL S.P.A., MILAN

Table 2.

YEAR	OWNER	WORK	Ø [m]	L [m]	L [m]	L [m]	L [m]	No
1989	R.A.V.	AOSTA–Mt. BLANC HIGHW. Tunnel Avise	12	290×2				4
1989	F.S.	UDINE–TARVISIO Ry. Tunnel Tarvisio	16	1000				1
1989	SAT	LIVORNO–CIVITAV. HIGHW. Tunnel Malenchini	12	900×2				2
1989	SAT	LIVORNO–CIVITAV. HIGHW. Tunnel Rimazzano	12	900×2		900×2		2
1990	Comune di Roma	Underway Cristoforo Colombo road	12.5	150				
1990	ANAS	STATE ROAD No 1 Tunnel Montenero	11	150×2		2350×2		2
1990	LAURO	STATE ROAD No 38 Tunn. Valmagg. Bolladore	12	150				4
1990	COGEFAR	STATE ROAD No 38 Tunnel Mondadizza	12	60				1
1990	CARIBONI	STATE ROAD No 38 Tunn. Le Prese and Verz.	12	100				2
1990	LODIGIANI	STATE ROAD No 38 Tunnel San Antonio	12	40				2
1990	TECNO_ SVILUPPO	STATE ROAD No 38 Tunnel Tola	12	80				2
1990	SECOL	STATE ROAD No 38 Tunnel Cepina	12	70				1
1990	R.A.V.	AOSTA–Mt. BLANC HIGHW. Tunnel Leverogne	12	270×2				2
1990	R.A.V.	AOSTA–Mt. BLANC HIGHW. Tunnel Villerot	12	240×2				4
1990	R.A.V	AOSTA–Mt. BLANC HIGHW. Tunnel Les Cretes	12	450×2				2
1990	MM	MILAN RING–ROAD EAST Underway	11	70				2
1991	ANAS	STATE ROAD No 510 Lots 5,6,7	11	900		400		15
1991	ANAS	STATE ROAD No 237 Tunnel Sabbio	11	300				3
1991	F.S.	CASERTA–FOGGIA Ry. Tunnel S.Vitale	12		300	1300		
1991	ANAS	STATE ROAD No 62 Tunnel M. Barro	11	100×2				1
1991	PIZZA_ ROTTI	STATE ROAD No 470 Tunnel Lenna	12	30				
1992	ANAS	E 45 Tunnel Quarto	11	100×2		200×2		3
1993	F S	ANCONA–BARI Ry. Tunnel Vasto	10	1270	1850	4970		
1993	F.S.	FIRENZE–EMPOLI Tunnel S Vito	12	130		200		1
1993	F.S.	FIRENZE–EMPOLI Tunnel Bellosguardo	12	125		360		2
1993	ANAS	STATE ROAD No 106 Tunnel Lofiri	11	90×2		90×2		2
1993	INTER_ METRO	ROME UNDERGROUND Baldo degli Ubaldi stat	21		120	120		

TUNNELS DRIVEN USING CONSERVATION METHOD AND DESIGNED BY ROCKSOIL S.P.A., MILAN

The development of conservation methods was the natural consequence of detailed study, both theoretical and on physical and numerical models, that has been extensively illustrated in this paper.

In this paper we have discussed, in detail, use of fibre-glass tubes underground. It should, however, be made clear, before concluding, that they can be employed with success on the surface, for example for the stabilisation of slopes to improve safety.

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