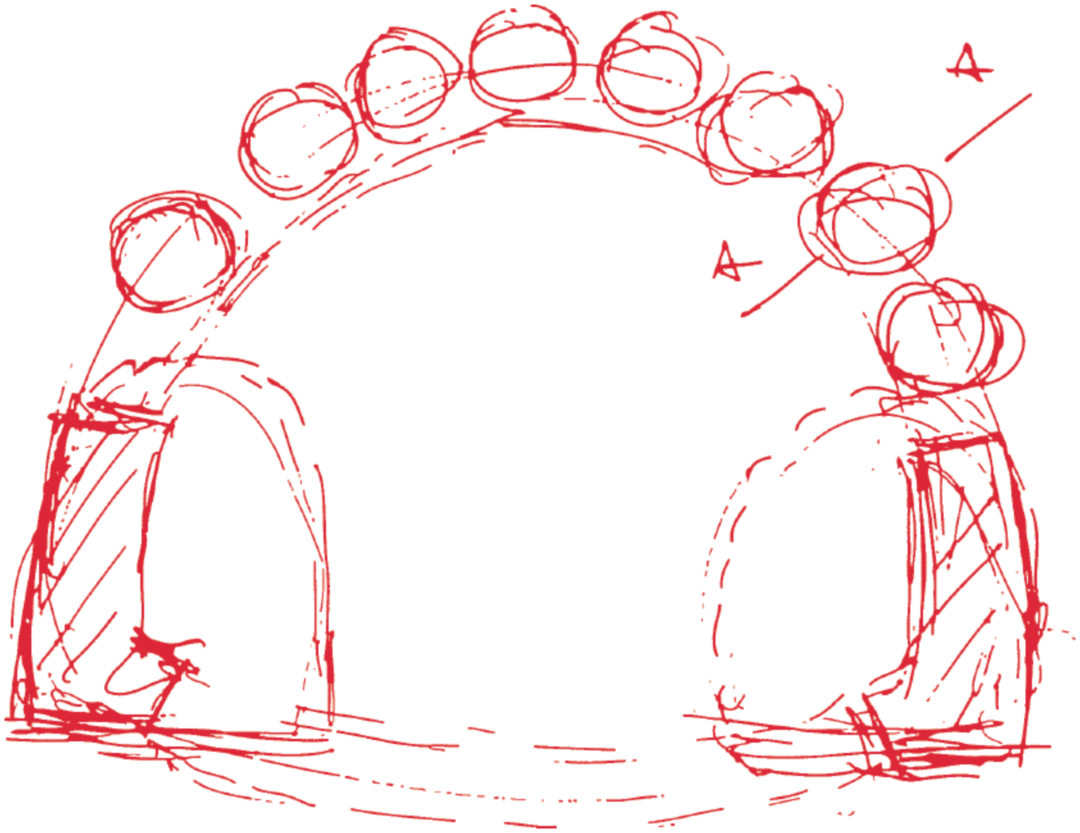
1979 - 2009



ROCKSOIL S.p.A.

S.p.A.

1979 - 2009

Central coordination:
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Conference Photographer:
FIORENZO CANTALUPI

Publication date: xxxxxxxxxx

Editorship:
EDI-CEM Srl
Main Office: Via A. Sciesa 6/A, 20135 Milan (Italy)
Legal Office: Via E. Morosini 22, 20135 Milan (Italy)

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Print:
GRAPHICSCALVE SpA
Loc. Ponte Formello
20024 Vilminore di Scalve (BG)

Printed in Italy

SUMMARY

PART ONE
THE CONFERENCE

PART TWO
THE TECHNOLOGIES

It was to leave a mark, that on the occasion of the thirtieth anniversary of the engineering consulting firm, Rocksoil S.p.A., I wanted to organise a day of study on great developments that have occurred in the "design approach in the field of tunnelling" since the nineteen fifties until today, a mark both of what the engineering consulting firm Rocksoil S.p.A. has been able to produce in its thirty years of work in the sector of major underground works and of the consequent and prestigious results it has achieved in the field of design.
Experts on the subject, the most outstanding in the field nationally and internationally, have wished, with their presence at this meeting and with their fascinating contributions, to pay tribute to the subject addressed which, without doubt, represents a milestone for those who work in the field and for those who face the challenges of underground engineering.
I consider the result of this thirtieth anniversary a deserved tribute to all my valued assistants, business friends, major clients and all those with whom we have shared moments, emotions and successes and finally to those men and women who, when they pass a day in a train or in a car travelling through our tunnels, can hardly imagine, on the one hand what lies behind the design and construction of an underground work, and on the other, how these works, which by their nature are not born in the light of day, are among the most fascinating and special in the field of civil engineering applied to public works.

Prof. Ing. Pietro Lunardi
Founder of Rocksoil S.p.A.

PREFACE

The plasticisation that may be produced within a rock mass during the construction of an underground work often produces substantial disturbances, difficult to stop and costly in terms of time and money.

The conference held on the occasion of the thirtieth anniversary of the foundation of the company Rocksoil on the evolution of design and construction approaches in the field of underground works is of a really exceptional nature, because the contributions have been prepared by academics, professors, engineers and experts who have excavated tunnels all over the world and also because the progress made over the last forty years has been spectacular. The plasticisation that is produced in rock masses both at the face and around the excavation during and even after the final lining has been placed, in what we call the "excavation disturbed zone", must be appropriately controlled to prevent undesired disturbances. To achieve this, reinforcement of the core-face, as suggested by professor Pietro Lunardi more than twenty years ago, has made it possible to drive tunnels full-face, which is always the most economical solution.

Pierre Habib
Chairman of Rocksoil S.p.A.

Formerly:
- *President of Société Internationale de Mécanique des Roches (ISMR),*
- *Director of the École Polytechnique of Paris*
- *Director of the Laboratoire de Mécanique des Solides de l'École Polytechnique*

• presentation
DILETTA PETRONIO

• welcome greetings
PIETRO LUNARDI

• first session
PIERRE HABIB
KALMAN KOVÁRI
FULVIO TONON
MARC PANET
GEORGE ANAGNOSTOU
NORBERT VOGT

• second session
ADOLFO COLOMBO
GIOVANNI BARLA
GIOVANNA CASSANI
HIROMICHI SHIROMA
BRUNO MATTLE
JUAN JACOBO SCHMITTER

PART ONE
THE CONFERENCE DAY

INTRODUCTION
Diletta Petronio

It is with great pleasure and interest that I introduce this day of meetings and discussion on this highly specialist subject of geo-engineering as applied to underground works. We will be listening during the day to protagonists of the highest international standing in a fascinating and extremely important sector, that is developing constantly.

It is a sector that is far removed from the work I have done for many years everyday as a journalist with TG4 (news department of a national TV channel). It is very close to daily reality, because just like everybody else, I happen to travel through tunnels and ask myself "Who made, this? How was it built?". It is the normal amazement of ordinary people like myself, who are not tunnelling experts.

DILETTA PETRONIO

A HAS BEEN A JOURNALIST SINCE 1988 AND IS A NEWSCASTER FOR TG4

A LARGE AND EXPERT AUDIENCE AT THIS DAY OF MEETINGS AND DISCUSSION TO CELEBRATE ROCKSOIL'S 30TH ANNIVERSARY

MILAN 16TH OCTOBER 2009

A HIGHLY SYMBOLIC PHOTO OF THE VENEZIA STATION BEFORE THE FINAL FINISH

MILAN URBAN LINK LINE

∅ = 30 M.
GROUND: SANDS AND GRAVELS
OVERBURDEN: 4 M.

DESIGN BY ROCKSOIL S.P.A.

I have learnt that very often the answer to this question is Rocksoil, a company which, for thirty years, working with others, has designed and provided technical assistance for the construction of works which make transport and life easier for us all.

Here behind me is an emblematic photo of Venezia Station on the Milan urban railway link line. It is Rocksoil's masterpiece. Let us begin here with this photo to tell the story of this company's success and of its great commitment.

Pietro Lunardi asked me to introduce this very important day for Rocksoil consulting engineers, which with thirty years under its belt and 150 employees in Milan and Rome, represents a key niche in the civil and geo-technical engineering sector, a point of home-grown Italian excellence.

I have met and appreciated Pietro Lunardi, its founder, on many occasions. I have interviewed him several times on the main editions of the TG4 news broadcasts. On live television as Minister of Infrastructure and Transport, he always clearly explained to viewers how the Country was to change as a result of the major projects commenced by the Berlusconi government between 2001 and 2006. The Right Honourable Pietro Lunardi also knows how things are to change in future, because the period of change has not come to an end and the current government and parliament are working in this direction. I remember that as a result of the "Objective Law" which he wanted, a single decision-making process means that you now know exactly how long it will take for projects to be approved. It was a real revolution for the construction of infrastructures.

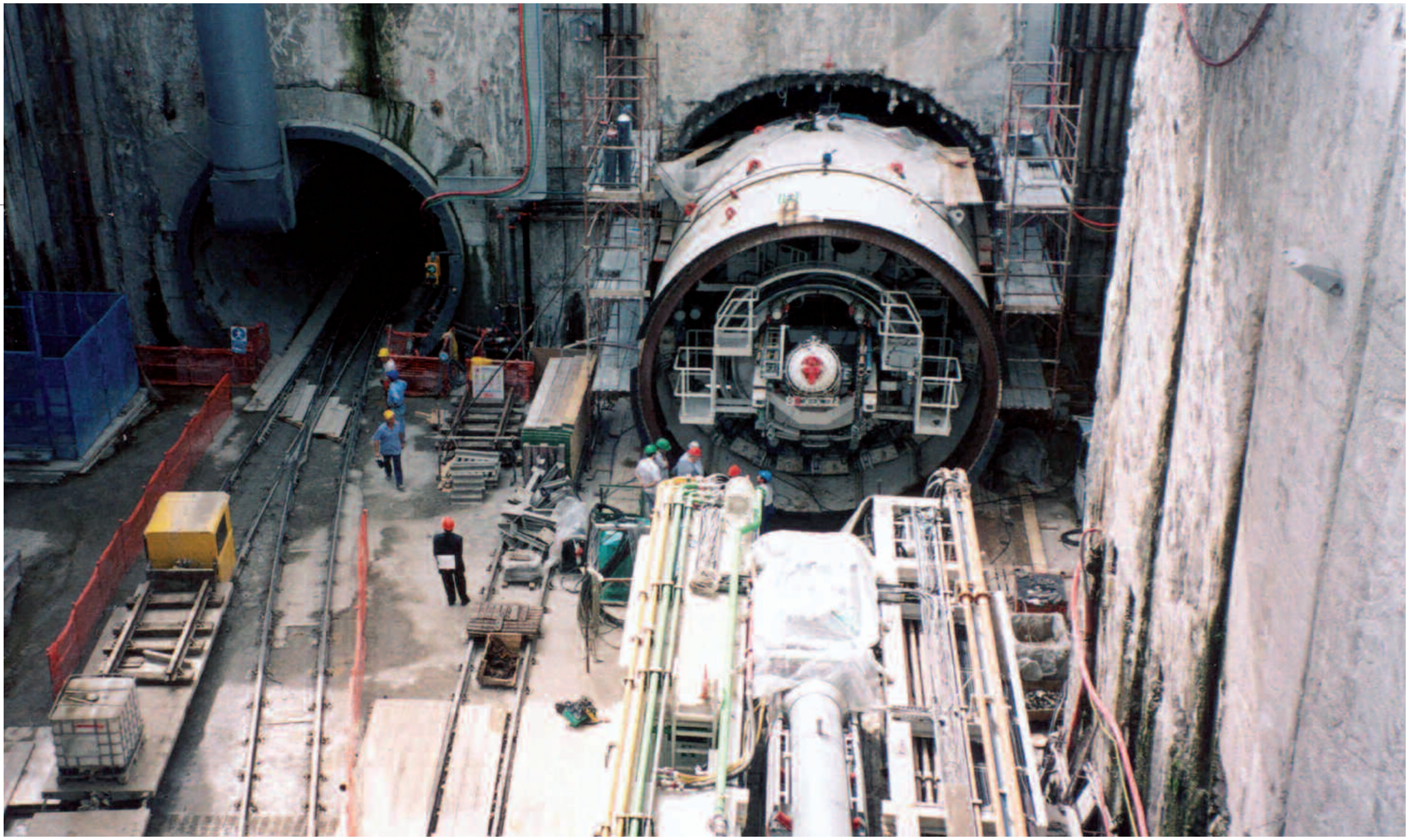
A DAY OF MEETINGS AND DISCUSSION FOR ROCKSOIL'S 30TH ANNIVERSARY

MILAN 16TH OCTOBER 2009

In a collection of articles by Pietro Lunardi entitled "The will to act", published in the newspaper Libero, Vittorio Feltri, the editor of the newspaper Il Giornale, defined him as the "Perfect engineer, who makes you safe". He beat me to it. However, having read, over the last few weeks, the material and articles needed to prepare for this day together, I can add that Pietro Lunardi is a genuine professional, an expert with a constant passion for his work.

So, thirty years have passed since the company Rocksoil S.p.A. was born.
The photos that we can see are of the numerous tunnels constructed over the years. They are works which have charted our future and will continue to do so.
To give a few figures, the company has designed and provided on site technical assistance for over 800 kilometres of tunnels. More than 9,000 tunnel faces have been supervised and monitored for road, motorway and mainline and metro railway tunnels.

NAPLES METRO
LINE 1
∅ = 6.75 M.
GROUND: TUFFS AND POZZOLANS
OVERBURDEN: 30 M.
DESIGN BY ROCKSOIL S.P.A.



TGV MÉDITERRANÉE
LYON-MARSEILLE LINE
"TARTAIGUILLE" TUNNEL
Ø = 15 M.
GROUND: CLAY
OVERBURDEN: 110 M.
DESIGN BY ROCKSOIL S.P.A.

Rocksoil S.p.A. has developed innovative technologies for digging deep into the bowels of the earth. It is a silent, effective and constructive approach, which does not disturb, as Pietro Lunardi explained to me with great simplicity. It was a remark which struck me a lot, if you consider the inconvenience that construction sites usually cause.

CATANZARO EAST RING ROAD
SAN GIOVANNI TUNNEL
∅ = 12 M.
GROUND: MARLY CLAY AND SAND
OVERBURDEN: 10 M.
DESIGN BY ROCKSOIL S.P.A.

Rocksoil's founder is the creator of many inventions and holds many patents in the geo-engineering field. They include:

▸ the Cellular Arch technology for the underground construction of wide span cavities in cohesionless soils with shallow overburdens, for which he received recognition in 1990 by the United States journal Engineering News Record, published by McGraw Hill, which each year nominates a "Man of the Year" in the construction field;

▸ the use of "horizontal jet-grouting" for tunnel advance in cohesionless soils;

▸ "vertical jet-grouting", employed for the construction of tunnel portals in cohesionless soils with minimal overburdens;

▸ the full-face application of a technology termed "mechanical precutting", for tunnel advance through clay;

▸ the stabilisation of a tunnel face, by reinforcing the core of ground ahead of it with fibre glass reinforcement, for tunnel advance with the face stable in the short term or unstable;

▸ the Analysis of COntrolled DEformation in Rocks and Soils (ADECO-RS) design approach, which we will speak of later, which opened a new page in the design and construction of underground works;

▸ a system for analysing the operating parameters of a TBM during tunnel advance to measure the strength of the rock mass, known as the "RS Method";

▸ a construction procedure for driving tunnels underground with no or insufficient overburden.

▸ the "Nazzano Method" which was employed for widening the Nazzano Tunnel of the Rome-Milan motorway from two to four lanes without interrupting traffic. It can be used to work under a great variety of ground conditions to widen road, motorway or rail tunnels to increase their capacity, while they remain in service.

TGV MÉDITERRANÉE
LYON-MARSEILLE LINE

"TARTAIGUILLE" TUNNEL

∅ = 15 M.
GROUND: CLAY
OVERBURDEN: 110 M.

DESIGN BY ROCKSOIL S.P.A.

We are looking at photos of the most significant works designed by Rocksoil consulting engineers. They include the Cellular Arch for the Venezia Station on the Milan urban railway link line.

The Tartaiguille Tunnel in France, constructed on the high speed line between Lyon and Marseilles.

The difficult Apennine section of the new high speed Bologna-Florence railway line, constructed between 1996 and 2005, with more than 100 km of tunnels completed fully on time and to budget. The project was considered one of the most important in the world as far as the underground works were considered. For those of you who did not know, the high speed section between Bologna and Florence will be completed on 21st December, making it possible to travel from Rome to Milan in two hours and fifty minutes.

The company Rocksoil SpA has not just designed tunnels, but it has also provided systematic technical assistance to construction contractors during the whole construction period, continuously refining designs during construction even in cases of projects subject to high risk.

Rocksoil's is therefore a history of successes, research and solutions even under extremely difficult conditions.

One of the solutions to major disasters worth recalling among the accomplishments of the company and its founder was the repair of the bridge on the Milan-Bologna railway line over the river Taro, which partially collapsed following record floods. It was repaired and reopened to traffic in just 34 days.

The Valtellina disaster of the Val Pola landslide. Everyone will remember the seriousness of the situation that kept Italy with its breath held in the summer of 1987. Everything was solved brilliantly with the famous controlled overflow. Since then Pietro Lunardi has received numerous government appointments, including the chairmanship of the commission of inquiry into the tragic fire in March 1999 which resulted in the closure of the Mont Blanc tunnel. It was reopened within three years.

In his book on the Gran Sasso Tunnel, the longest motorway tunnel in Italy and one of the most important in Europe, Pietro Lunardi recalls that "A site worker knows from experience that every tunnel has its own history. If this is true in general, it is even more so for the Gran Sasso Tunnel, a huge project, a technical challenge without precedent, which the combination of natural factors such as the rock, water and gas made it a training ground of experiences and life for those who supervised and built it. The particular conditions under which the work was carried out and in

THE TUNNELS AND CHAMBERS OF THE NUCLEAR PHYSICS LABORATORY UNDER THE GRAN SASSO which the different operational stages took place transformed normal tunnelling work into a marvellous piece of history which this book is intended to prevent those who already know of it from forgetting and to let those who pass through the tunnel in their cars know of things they would find it difficult to imagine."

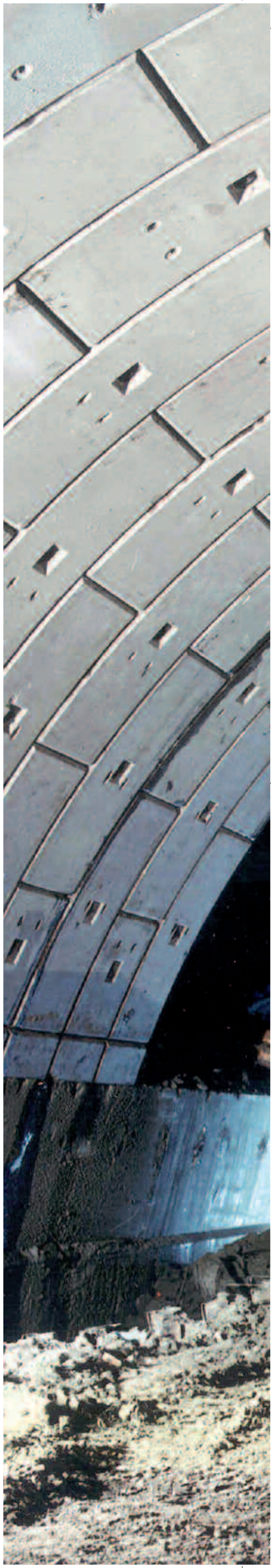
THE ROME METRO LINE "A"
BALDO DEGLI UBALDI STATION
∅ = 21.50 M.
GROUND: CLAY
OVERBURDEN: 18 M.
DESIGN BY ROCKSOIL S.P.A.

The Gran Sasso motorway tunnel constructed between 1968 and 1978 and the Alpine Frejus tunnel, constructed between 1975 and 1978, which Pietro Lunardi was able to experience as the geotechnical and geomechanical engineer, had a profound effect on his career.

It was after acquiring this fundamental experience that in 1979 Pietro Lunardi decided to found the company Rocksoil, with the objective of conducting research in the field of underground construction which might lead to a reassessment of underground works designed to put them on the same level as true and genuine civil engineering works.

The work performed by Rocksoil consulting engineers over the last thirty years has in fact been dedicated to experimental and theoretical research which has resulted in the development of a design approach called the Analysis of COntrolled DEformation in Rocks and Soils (ADECO–RS), an approach which should not be seen as a commercial product, but rather as the result of continuous and systematic theoretical and experimental research conducted in all types of ground and stress-strain conditions.

It is an exciting story with many chapters still to be told.



THE ROME METRO LINE "A"

BALDO DEGLI UBALDI STATION

∅ = 21.50 M.
GROUND: CLAY
OVERBURDEN: 18 M.

DESIGN BY ROCKSOIL S.P.A.

WELCOME SPEECH

Pietro Lunardi

PROF. ING. PIETRO LUNARDI,
FOUNDER OF ROCKSOIL S.P.A.

Dear friends, what better opportunity than the anniversary of a consulting engineers' firm to talk about civil engineering works and underground works in particular. We will be speaking about them with nine friends who have honoured us with their presence, nine speakers who have come from all parts of the world invited to discuss what are currently the most interesting developments in approaches to design. Sincere and special thanks must go to our nine friends:

- Kalman Kovári from Switzerland;
- Fulvio Tonon from the United States;
- Marc Panet from France;
- Norbert Vogt from Germany;
- George Anagnostou from Greece;
- Giovanni Barla from Turin;
- Hiromichi Shiroma from Japan;
- Bruno Mattle from Austria;
- Juan Jacobo Schmitter from Mexico.

Our thanks must also go to the chairmen of the two sessions, our friends Pierre Habib, who has been the Chairman of Rocksoil for over 15 years and Adolfo Colombo, the President of the Italian Tunnelling Society. My thanks must also go to the friends we have invited here today, representatives of design and construction companies and firms which operate in the sector. Last but no less important thanks go to my assistants who, along side my children Martina and Giuseppe, have continued to work and produce, despite my numerous absences over the last ten years. I cannot name them here one by one, but they know that I am sincerely thankful to each of them for what they have done, for what they are doing and for what they will do.
Thanks to them, our firm of consulting engineers, having survived the ups and downs we have experienced in Italy in the last ten years, has distinguished itself and achieved far from insignificant successes in the field of public works and engineering in general.
Ideas are very important in engineering, but it is all in vain if you have no one to help you to translate those ideas into concrete facts.

THE PLAQUE FOR THE PRIZE AWARDED TO PROF. ING. PIETRO LUNARDI BY THE UNITED STATES JOURNAL ENGINEERING NEWS RECORD FOR HIS CELLULAR ARCH DESIGN

PHOTOS OF THE DAY OF MEETINGS AND DISCUSSION FOR ROCKSOIL'S 30TH ANNIVERSARY

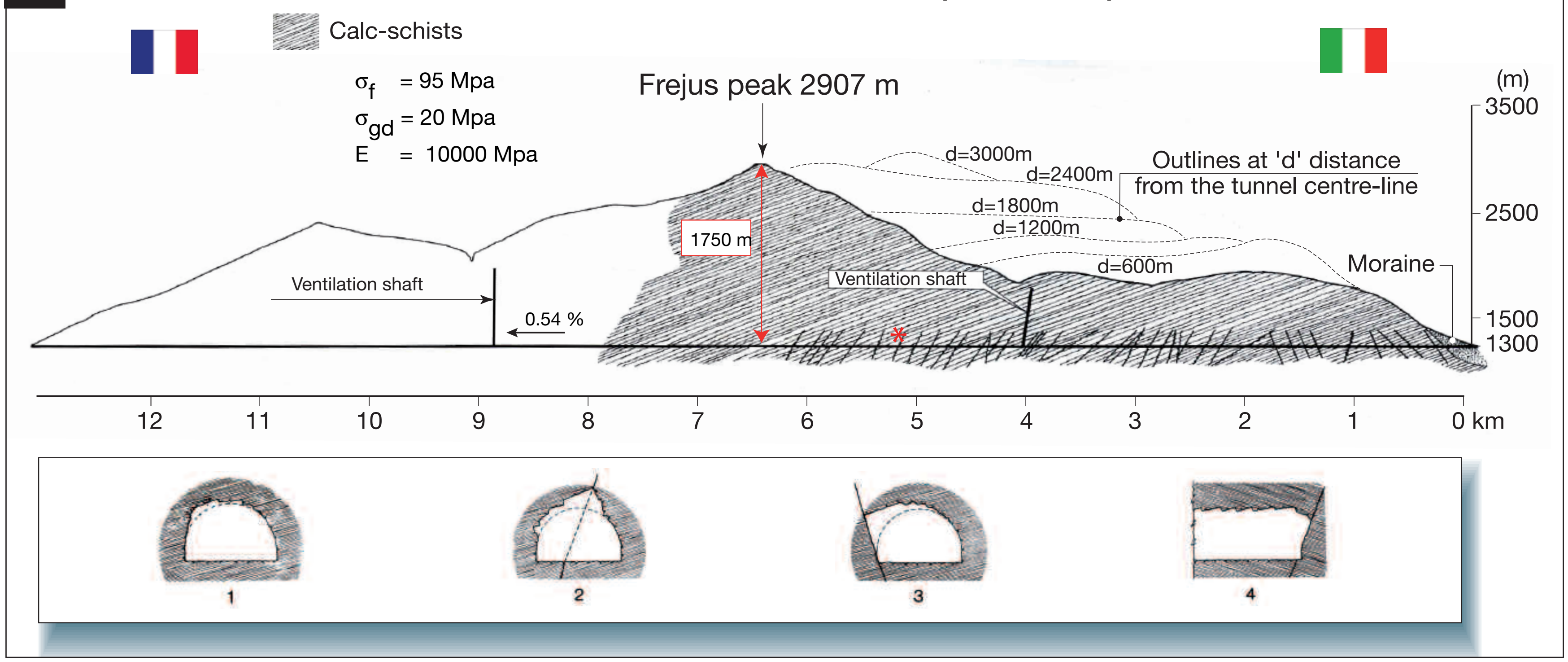
THE FREJUS
MOTORWAY TUNNEL
∅ = 12 M.
GROUND: CALC- SCHIST
OVERBURDEN: 1,700 M.

And that is precisely what my assistants have done!
They have helped me to demonstrate that, by starting in the design of underground works from the assumption that the ground which is subject to a field of stresses, which we will call the medium, is the true construction material of an underground work, the deformation response (DR) of the medium to the action of excavation constitutes the reaction on which a tunnel designer must focus.
They helped me to demonstrate that interpretation of the DR must not stop with consideration of simple cavity convergence alone, because convergence itself is no more than the last stage of a deformation process which begins ahead of the face and which manifests with extrusion at the face which then triggers preconvergence of the cavity.
They helped me to demonstrate that by acting on the core of ground ahead of the face with appropriate technologies, you can transform that core into a tool for controlling the DR by working on its rigidity.

They helped me to demonstrate that with good and indispensible knowledge of the medium and if possible the stress states to which it is subject, an underground work can become and be conceived of as a true and genuine civil engineering work and as such its construction times and costs can be forecast.
They helped me to demonstrate that what happened at a certain point on the alignment and under a certain overburden during the construction of the Frejus Alpine Road Tunnel would provide enlightening answers to our uncertainties and doubts.
The ground which we face with our excavations is something that is alive. It has its own language and its own mood and it can even make sounds! It is up to us to interpret this behaviour which accompanies the deformation response.
"Minima cura si maxima vis" is the motto that Federico Cesi, the founder of the Accademia dei Lincei (Academy of the Lynxes) adopted for the Academy in 1603. "Take care of the little things if you want to achieve the greatest results." In our case the small things are the visual and audible signals that the ground produces during tunnel advance. They are the events that miners tell you of and their suggestions. They are the hours that you pass in their company at the face.
But let us now consider the Frejus Alpine Road Tunnel which I mentioned before. As many of you know, the Frejus Alpine Road Tunnel (approximately 13 km) was constructed between 1975 and 1978 in record time with particularly significant results from a scientific viewpoint.
The road passes exclusively through a calc-schists formation on the Italian side with overburdens varying from a few metres to 1,700 metres, a unique opportunity to assess the

1. THE FREJUS MOTORWAY TUNNEL GEOLOGICAL PROFILE ON THE ITALIAN SIDE, WITH THE GEOMECHANICAL CHARACTERISTICS OF THE ROCK

1

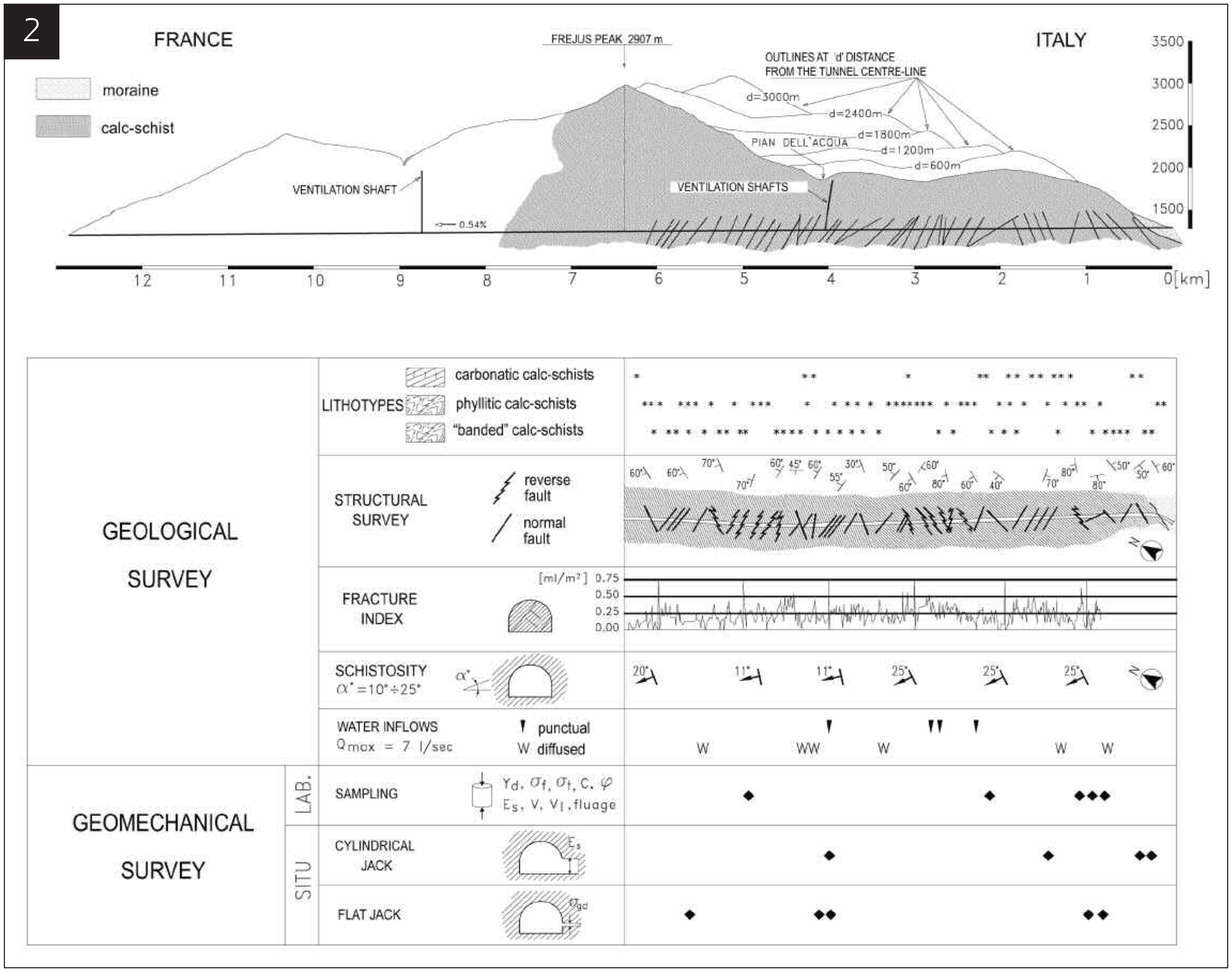


behaviour of the excavation and therefore of the cavity, as a sole function of changes in the stress states resulting from the increasing lithostatic loads. This opportunity, unique of its kind, naturally convinced us to conduct a geomechanical survey campaign appropriate to the situation. The most important data for the tunnel were:

- average excavation cross section of 90 sq. m.;
- cavity dimension of 9x12 metres;
- rock excavation by blasting;
- average tunnel advance of each round, varying from 2 metres to 4.50 metres;
- average daily tunnel advance of 7.50 metres;
- temporary stabilisation of the cavity performed at the face with end anchored rock bolts varying in length from 2.50 metres to 5 metres with a frequency varying between 1.50 to 1 rock bolt per square metre;

2. THE FREJUS MOTORWAY TUNNEL

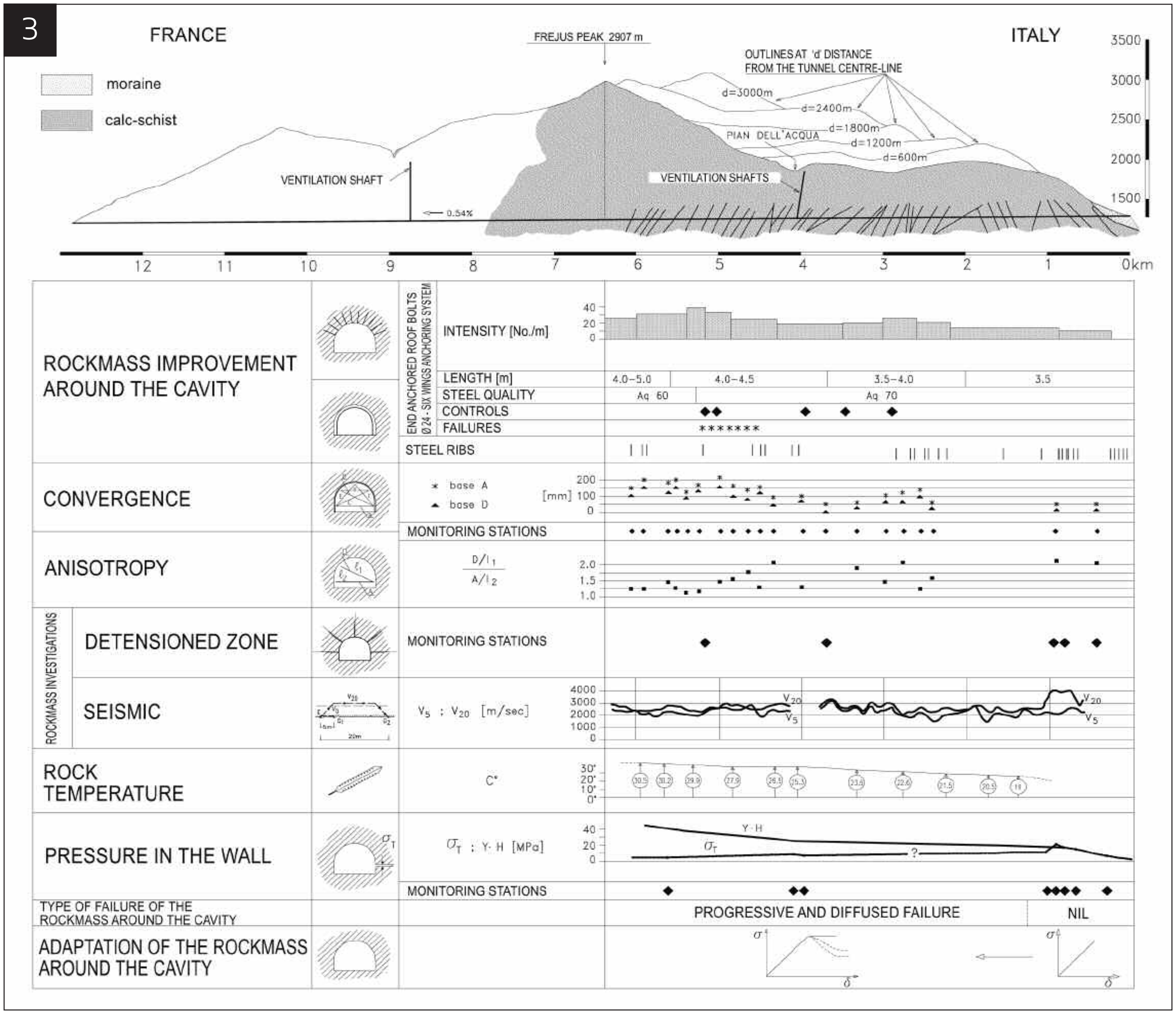
THE RESULTS OF THE LABORATORY AND ON SITE GEOLOGICAL AND GEOMECHANICAL STUDY



- final lining in concrete installed approximately 400 metres from the face.

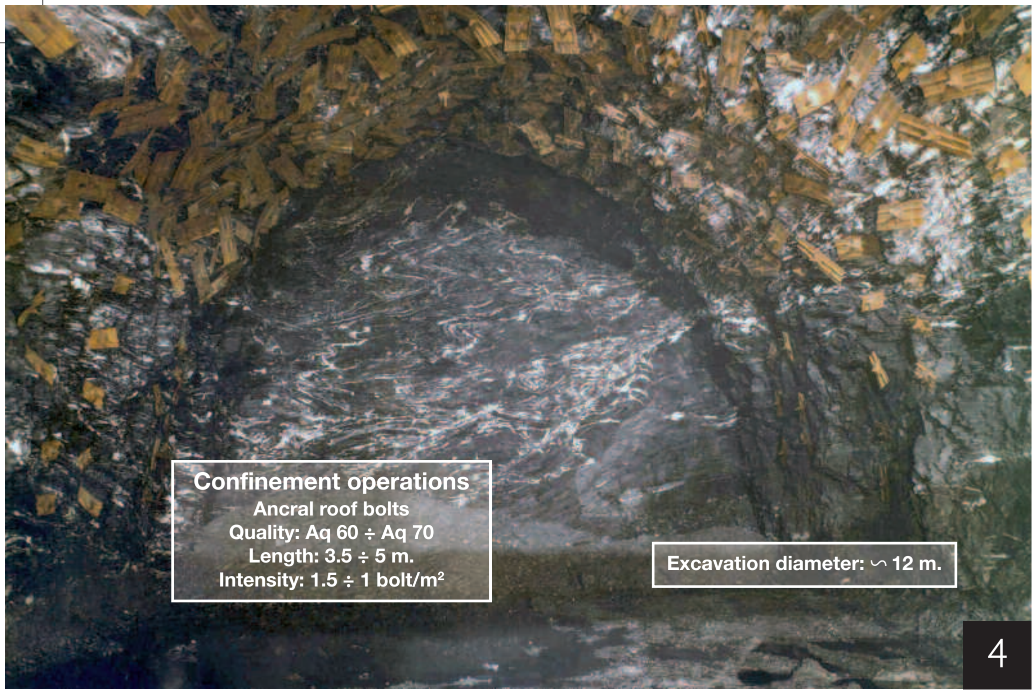
Let us now look at some slides to see what was found during the geomechanical survey: we can see (Figure 1) that we had a maximum overburden of 1,750 m, a matrix strength of 95 Mpa, a rock mass strength of 20 Mpa and an elastic modulus of 10,000 Mpa, so it was extremely good rock.

You can see from Figure 2 that systematic surveys were conducted of the face, the schistosity, the fracturing and water inflow over the 6,500 m. of the underground alignment. Just think that in very tight rock the maximum water inflow recorded along the tunnel was only seven litres per second.



3. *THE FREJUS MOTORWAY TUNNEL*

OPERATIONS TO STABILISE THE CAVITY AND MEASUREMENT OF: CONVERGENCE, ANISOTROPY, TEMPERATURE AND STRESSES IN THE WALLS

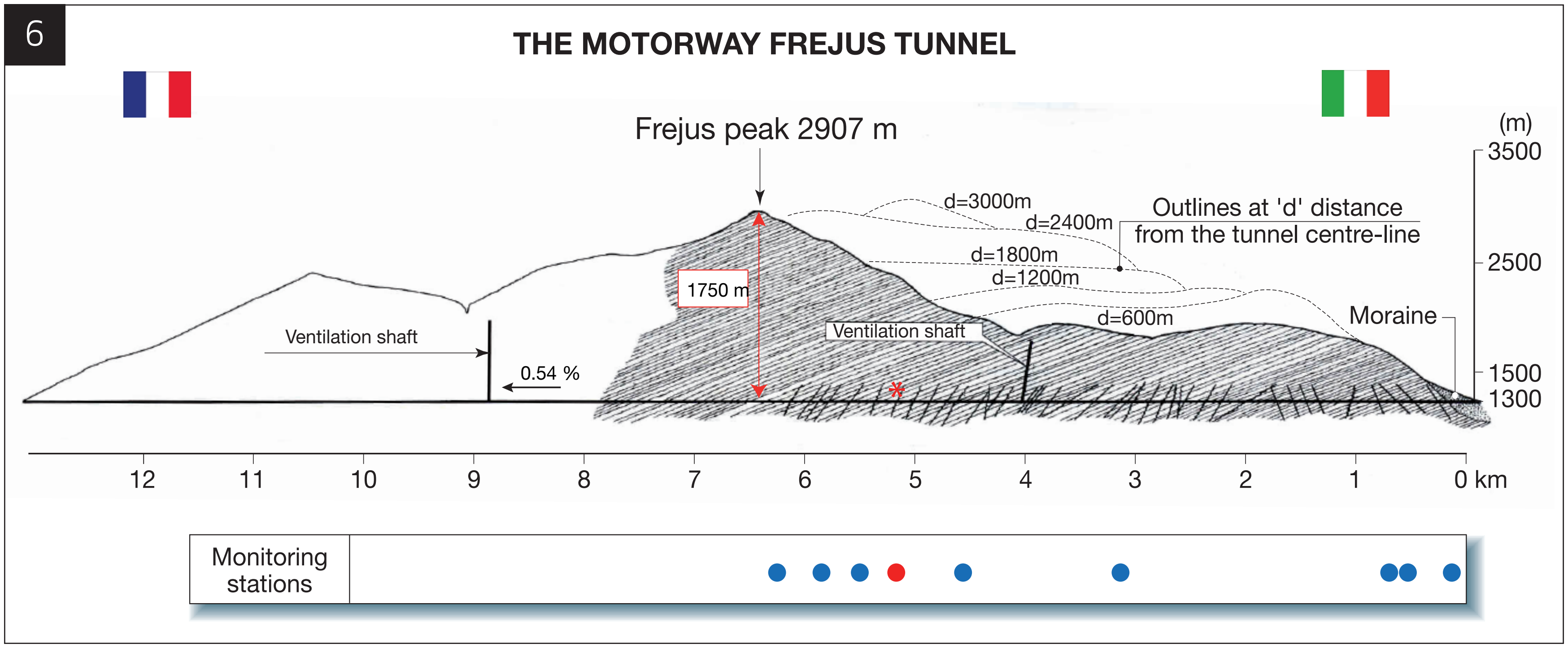


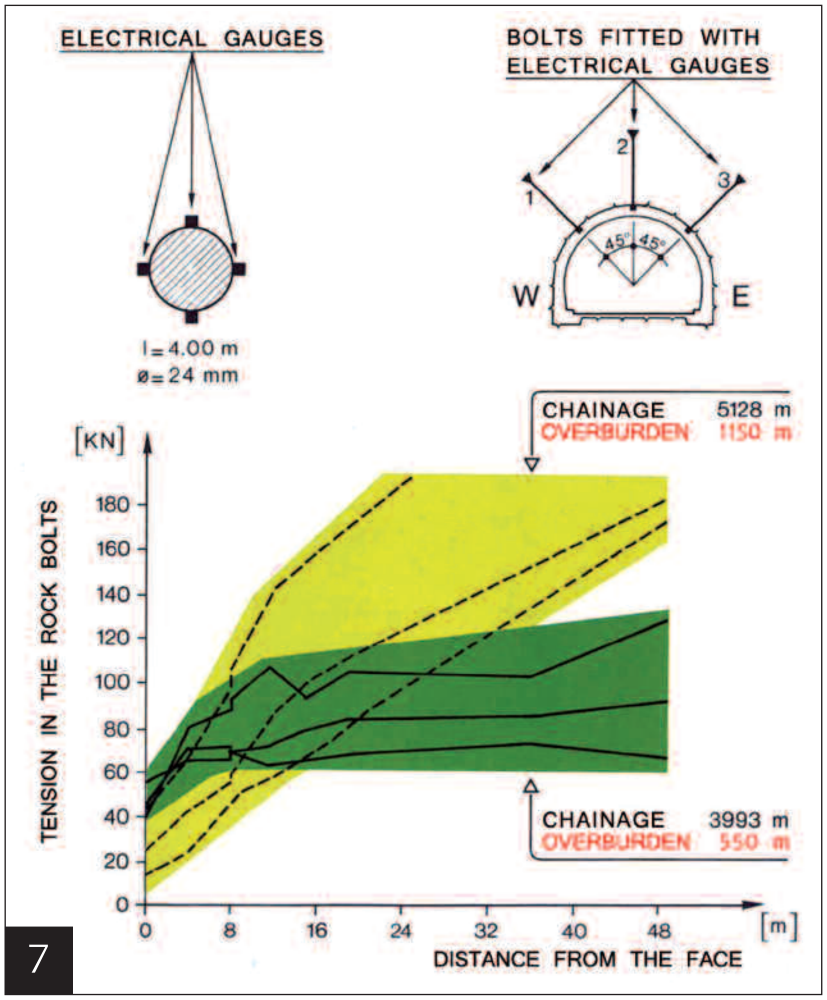
4, 5. *THE FREJUS MOTORWAY TUNNEL: THE FACE AND STABILISATION OPERATIONS*

6. *LONGITUDINAL PROFILE*

WITH THE POSITION OF THE MONITORING STATIONS FOR CONVERGENCE MEASUREMENTS

Figure 3 shows the intense research work that was conducted along the tunnel, while the photographs in figures four and five clearly illustrate the 12 m. full face method of excavation and the stabilisation employed using rock bolts inserted into the walls of the tunnel close to the face. This was followed by a 70 cm. thick final lining.
As we have seen a very in-depth survey campaign was conducted during tunnel excavation. Figure 6 shows the location of the monitoring stations, while Figure 7 gives an example of the stress measurements in the anchors which were made using electrical extensometers. Figure 8 which follows shows changes in convergence and it is particularly important.
In fact it shows the series of convergence curves which we recorded at the chainage points given at the bottom, corresponding to particular overburdens. You can see that for





7. THE FREJUS MOTORWAY TUNNEL: MEASUREMENT OF THE STRESSES IN THE ANCHORS USING ELECTRIC EXTENSOMETERS

WITH OVERBURDENS OF 550 M. AND 1,150 M.

overburdens of approximately 500 m., the curves show a pseudo elastic development, typical of a tunnel which is advancing under stable face-core conditions, whereas beyond that the behaviour is elastic-plastic with convergence of around 10-20 cm. at tunnel advance speeds of approximately 200 m. per month, typical of a tunnel advancing under conditions with the face stable in the short term.

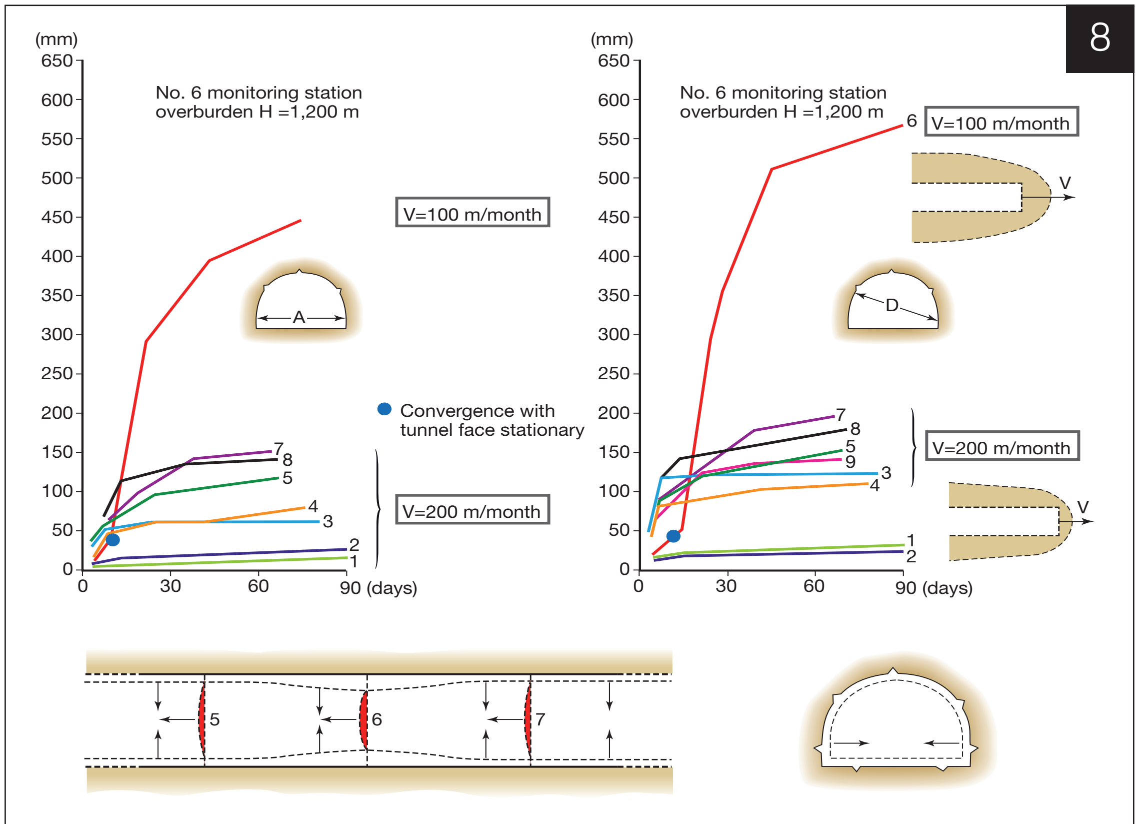
The most interesting thing is what happened at chainage 5172 under an overburden of 1,200 m. (Station No. 6). Here the face was halted for the summer vacations, after reinforcing the ground around the cavity only with more than 30 Ancral rock bolts per linear metre and after installing a convergence station one metre from the face (convergence station No. 6). After 15 days of halted advance, maximum deformation (due to fluage) of approximately 10 cm. was found. When excavation resumed using the same methods as before, convergence rose sharply to reach 60 cm. after three months. After the face had advanced a few tens of metres further, convergence returned to normal values.

Such high convergence, after the face had moved on from the measurement station, can only be explained if it is admitted that the calc-schist subjected to the high stresses produced by the lithostatic load behaved like clay, extruding towards the cavity and predisposing the future profile of the excavation ahead of the face to a form of pre-convergence. The pre-convergence functioned as an antechamber, predisposing the cavity to the strong convergence systematically measured after the face had moved forward from the measurement station.

This simple and perhaps even obvious consideration convinced me of the need to organise serious and in-depth research into the relationships between changes in the stress state of the medium (the ground) induced by the advance of a tunnel face and the consequent deformation response (reaction) (Figure 9).

The research, conducted with my assistants at Rocksoil S.p.A., was carried out in three stages as shown in Figure 10. It was immediately recognised in the first stage of the research that new reference parameters needed to be identified (Figure 11): the advance core and the three components of the deformation response, which were then

8

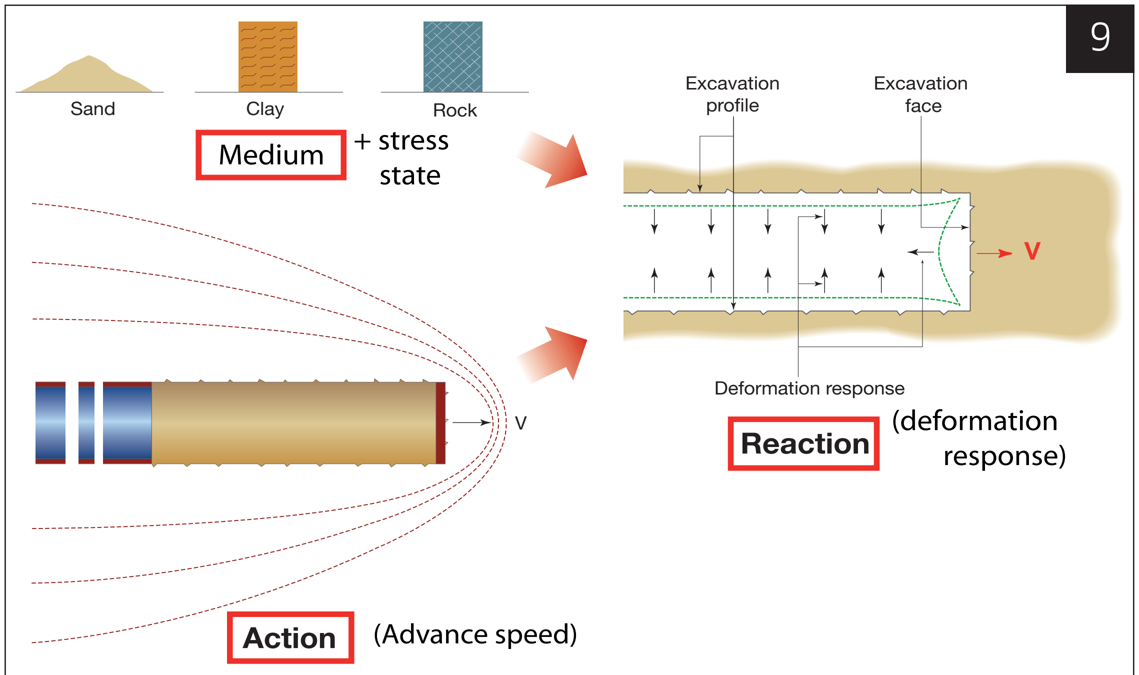


Monitoring stations	1	2	3	4	5	6	7	8	9
Chainage (m)	845	861	2772	3954	4507	5172	5533	5915	6066
Overburden (m)	490	500	580	590	740	1200	1400	1530	1640

8. *THE FREJUS MOTORWAY TUNNEL: MEASUREMENTS OF CONVERGENCE AT DIFFERENT MONITORING STATIONS*

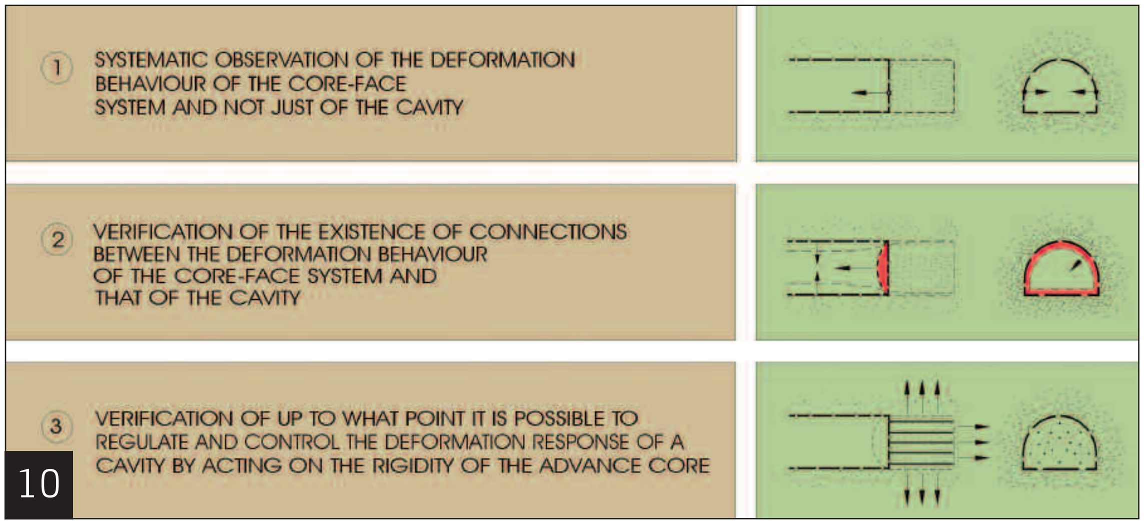
UNUSUALLY LARGE CONVERGENCE WAS OBSERVED AT STATION NO. 6 AFTER THE FACE HAD HALTED FOR 15 DAYS, WHILE OTHER PARAMETERS REMAINED UNCHANGED. THIS RETURNED TO NORMAL VALUES AFTER THE FACE HAD ADVANCED A FEW TENS OF METRES FURTHER

9



9. *MEDIUM, ACTION AND REACTION*

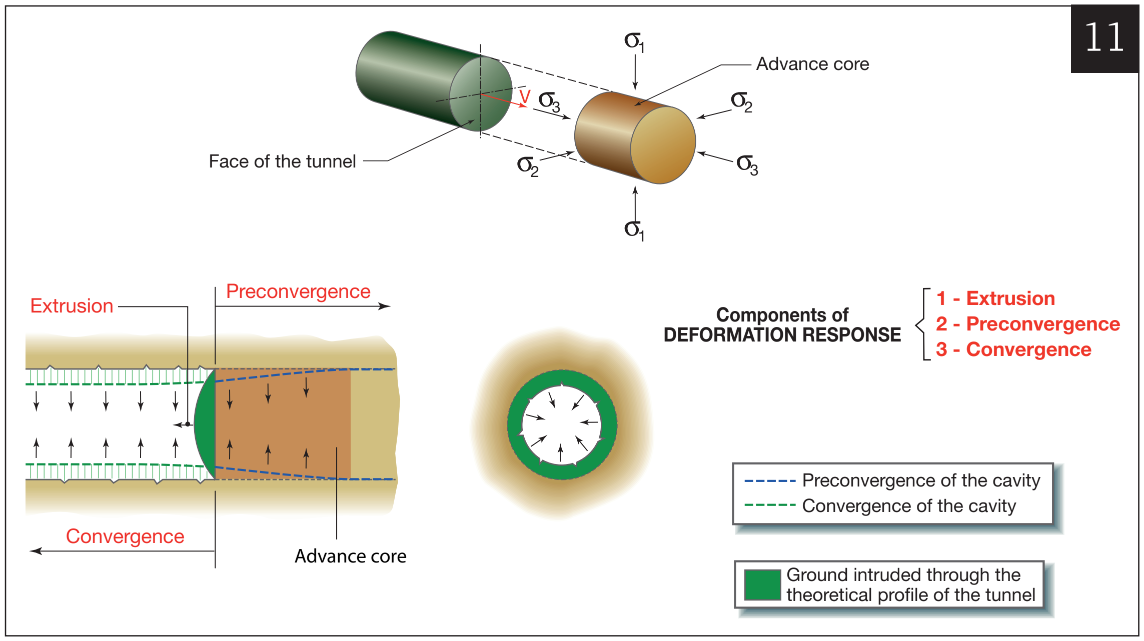
THE FREJUS EXPERIENCE LED TO IN-DEPTH RESEARCH INTO THE RELATIONSHIPS BETWEEN CHANGES IN THE STRESS STATE OF THE MEDIUM INDUCED BY THE ADVANCE OF A TUNNEL FACE AND THE CONSEQUENT DEFORMATION RESPONSE

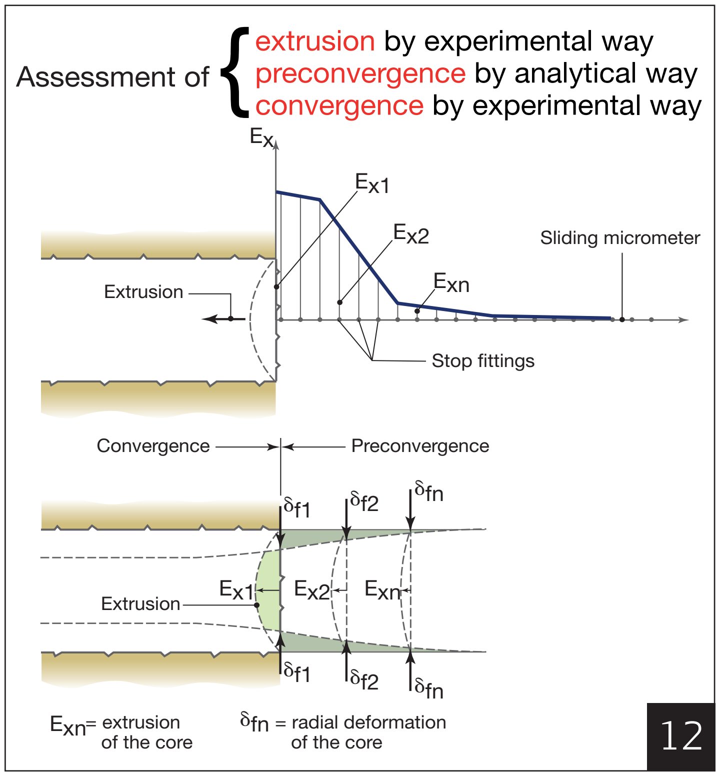


10. *THE THREE STAGES OF THE RESEARCH*

11. *THE NEW RESEARCH PARAMETERS*
• THE ADVANCE CORE
• THE DEFORMATION RESPONSE CONSISTING OF:
- EXTRUSION
- PRE-CONVERGENCE
- CONVERGENCE

systematically and carefully monitored during the excavation of each tunnel, especially under difficult stress-strain conditions (Figure 12).
The results of the research led to the conclusion that by acting on the rigidity of the advance-core with conservation operations and reinforcement techniques, it was possible to control deformation in the core (extrusion and pre-convergence) and as a consequence to also control the deformation response of the cavity (convergence) (Figure 13).
Even tunnels, like all other civil engineering works can be constructed according to industrial criteria, on time and to budget, as forecast at the time of design, provided the correct design approach is adopted (Figure 14).
We are convinced that, priority must be given in a correct design approach, to analysis of the deformation response. This must be predicted theoretically, with the support of





mathematical models, before methods to control it can be defined using pre-confinement and/or confinement of the cavity (tunnel section types). During the construction stage the deformation response, measured by experimental means, must then be compared to the deformation response predicted at the design stage for the final calibration of the action taken during construction.

I believe that it is only by following a correct design approach of this type that we will be able in the future to avoid the mistakes of the past which confused the ideas of designers and operators in the sector:

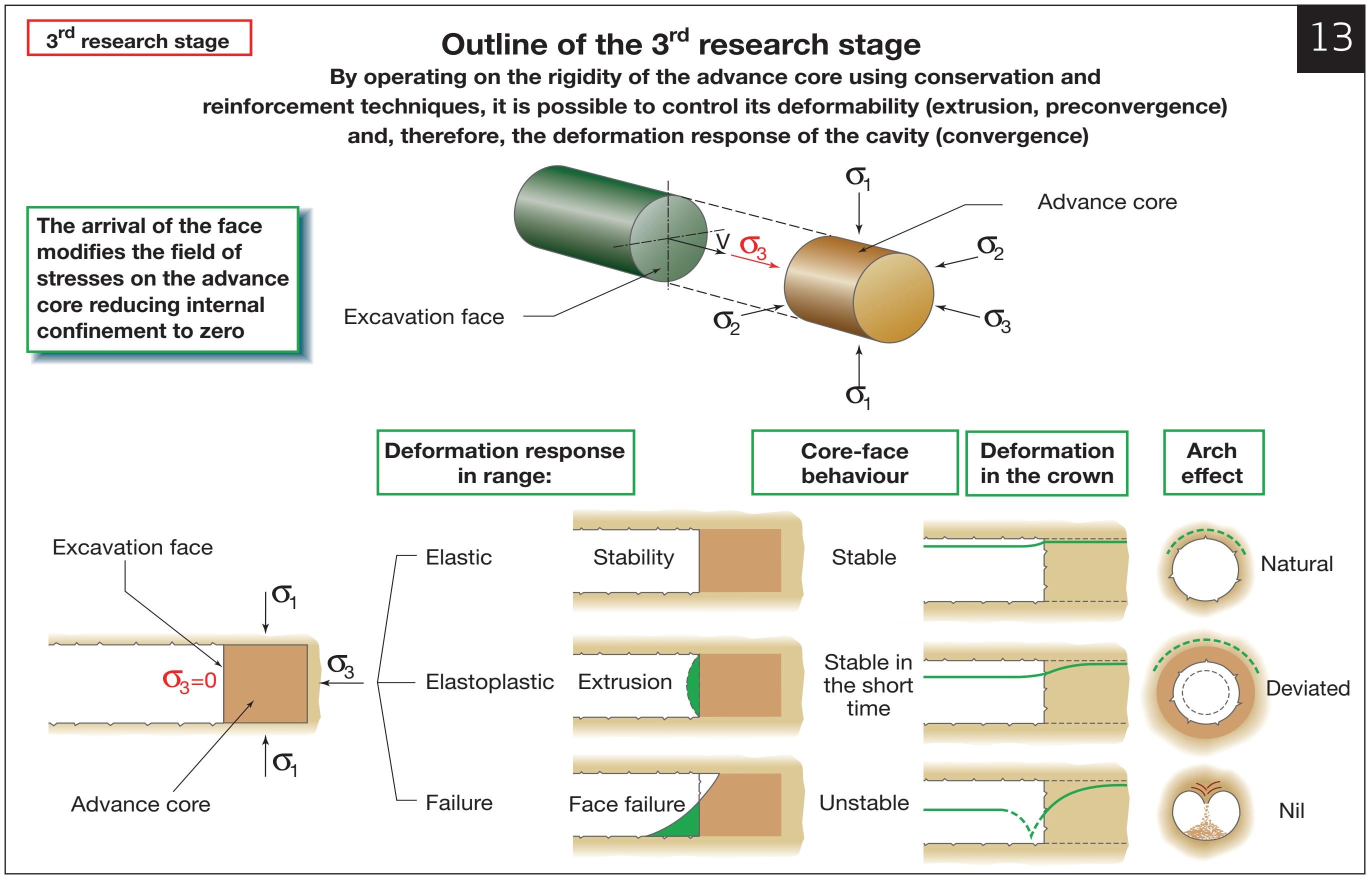
- avoid making non homogeneous comparisons (Figure 14);
- avoid considering tunnels as a plane geometry problem.

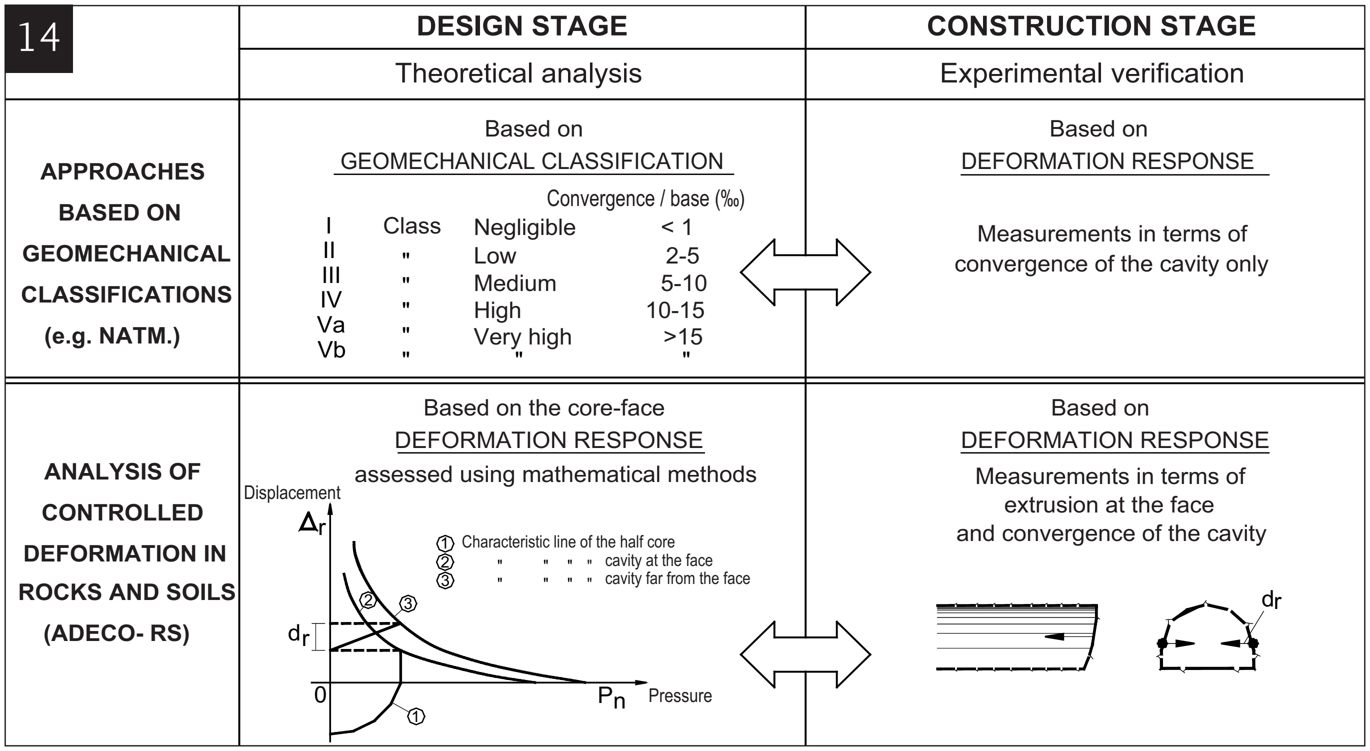
In this manner we will be able to consider tunnels as true and genuine civil engineering works, i.e. as works that can be constructed to budget and on time. We will then be able to say that we have succeeded in industrialising excavation, not just under easy conditions, but also for conventional excavation under more difficult, if not extreme, stress-strain conditions (Figure 15).

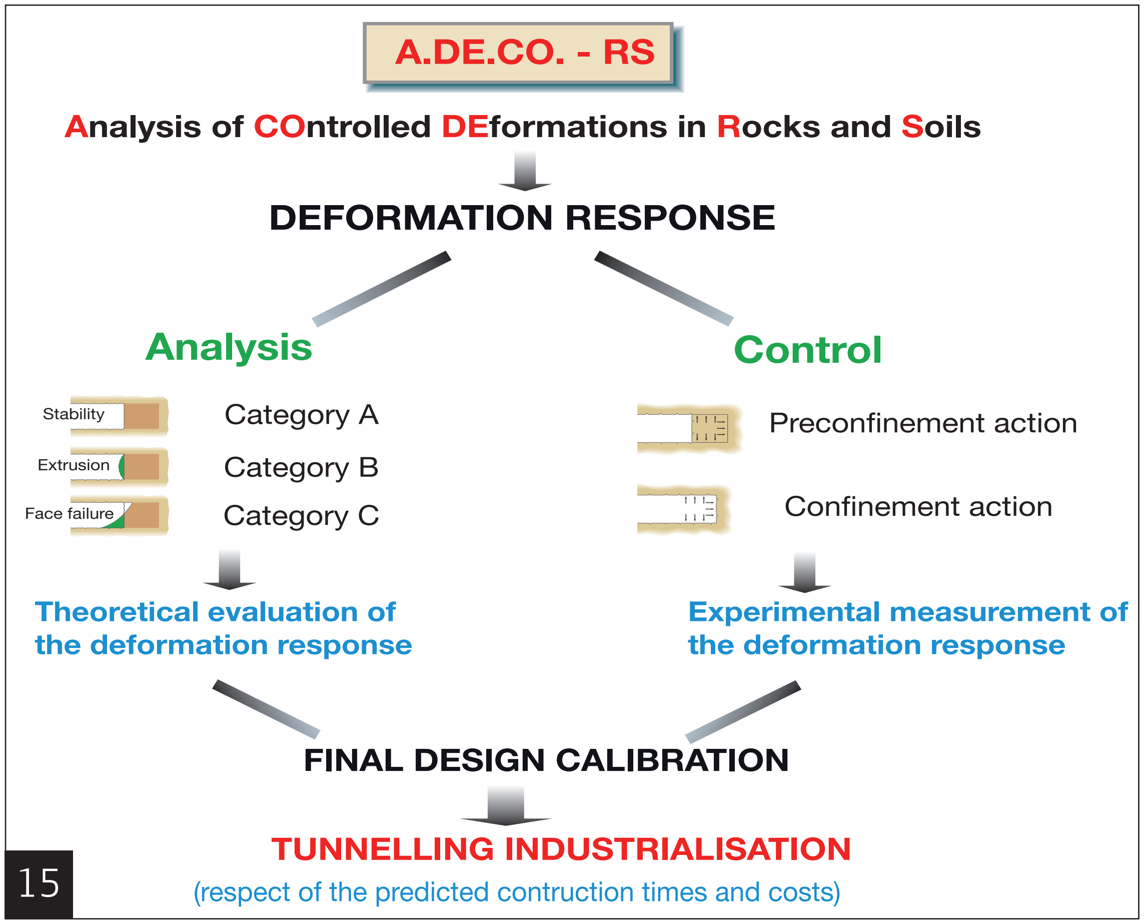
***12.** THE THREE COMPONENTS OF THE DEFORMATION RESPONSE WERE MONITORED DURING THE RESEARCH*

EXTRUSION OF THE CORE-FACE
PRE-CONVERGENCE OF THE CAVITY
CONVERGENCE OF THE CAVITY

***13.** THE RESULTS OF THE RESEARCH*







14. *MONITORING DURING CONSTRUCTION*

THE ADECO-RS APPROACH AVOIDS THE PAST MISTAKE OF COMPARING NON HOMOGENEOUS PARAMETERS AND CONSIDERING TUNNELS AS A SIMPLE PLANE GEOMETRY PROBLEM

15. *THE INDUSTRIALISATION OF TUNNELLING IS PERFORMED THROUGH ACCURATE ANALYSIS AND EFFECTIVE CONTROL OF THE DEFORMATION RESPONSE*

TUNNELS, LIKE ALL OTHER CIVIL ENGINEERING WORKS, CAN BE CONSTRUCTED ACCORDING TO INDUSTRIAL CRITERIA, ON TIME AND TO BUDGET, AS FORECAST AT THE TIME OF DESIGN, PROVIDED THE CORRECT DESIGN APPROACH IS ADOPTED

FIRST SESSION
Pierre Habib

On this occasion of the thirtieth anniversary of the company Rocksoil, I am delighted and honoured to present the first of the two sessions on underground works.
This day is very exceptional because the professionals, experts, professors and scientists which you will hear have driven tunnels throughout the world and have contributed to the significant advances that have been made in this sector over the last forty years.

PROF. ING. PIERRE HABIB,
CHAIRMAN OF ROCKSOIL S.P.A., FORMER PRESIDENT OF THE INTERNATIONAL SOCIETY FOR ROCK MECHANICS AND DIRECTOR OF THE SOLID MECHANICS LABORATORY AT THE ECOLE POLYTECHNIQUE DE PARIS

CHAIRMAN'S TABLE
FIRST SESSION

THE CRACKING OF ROCKS AROUND TUNNELS AND UNDERGROUND WORKS

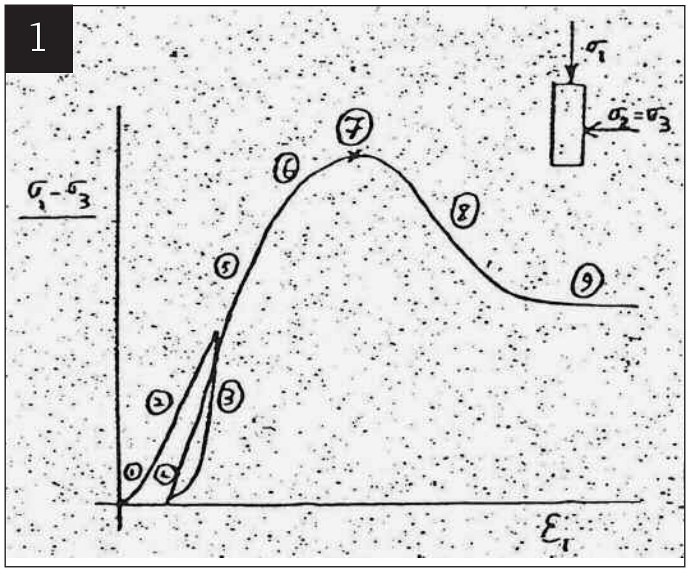
1. THE BEHAVIOUR OF THE ROCKS

Figure 1 shows a typical curve of the behaviour of geomaterials (rocks or concrete) during a triaxial compression test. At point (1), at the beginning of the load, sometimes a slight initial stiffening is observed due to the closing of open cracks in the material and also perhaps to an initial uncertain contact of the sample with the platforms supporting it. At points (2) and (5) the curve of the first load is approximately a straight line and one may speak of an elastic modulus E_1 (as well as a Poisson modulus ν_1). On the other hand, the unloading curve (3) can also be approximated, but with another straight line (also with another elastic modulus E_2 and another Poisson modulus ν_2). Il ricarico (4) rimette le cose in ordine e il tratto (5) si trova, grosso modo, sul prolungamento di (2). The reload (4) tidies things up again and the section (5) is more or less an extension of (2). Generally, the relationships between the velocity of the sound wave V, the elastic modulus E, and the density of the material p of the form:

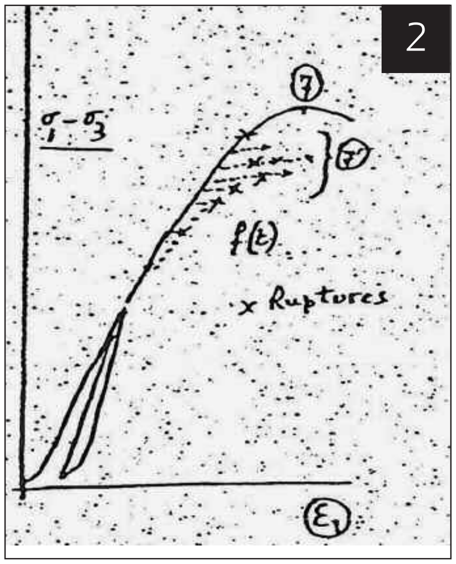
$$V = \sqrt{\frac{E}{\rho}}\, f(\nu)$$

do not give good results with E_1 and ν_1. Obviously, since the stresses due to the propagation of the sound waves are cyclical, it is best to compare V with E_2 rather than with E_1. However, even this method does not give good results and all that can be said is that the greater the velocity of the propagation of the sound, the greater the breaking strength of the material: a monotonic relationship is obtained between E and Rc.

1. INSTANTANEOUS BEHAVIOUR DURING A TRIAXIAL TEST



2. DEFORMATION DEFERRED OVER TIME



As the load increases, small cracks are produced in the rock. They gradually increase in number as the stress-strain curve (6) begins to curve and as the maximum load that the rock can support is reached (7). At this point, if two observers watch, one focusing on the increasing deformation and the other on the appearance of a failure surface, it will be the observer who sees the slope of the curve (6) start to change to a horizontal position who warns the other that a slip surface (8) is about to appear (and not the other way round). In fact the work of J. Desrues (1984) at Grenoble showed that local deformation assumes a diffuse organisation very early during loading and causes a localisation of deformation into a genuine failure surface. As it develops (8) an increase in volume is produced, as occurs in soil mechanics during the development of a slip surface in a dense sand. In the case of dense sand, if the slip surface is prevented from developing, the increase in volume is general throughout the sample. This happens, for example, for a sample that is not very thick placed between two plates with nil friction. Here, the slip surfaces hit the plates and are unable to develop further and the volume continues to grow. The same thing happens for rocks and for cements. In a triaxial test, the slope (8) therefore corresponds to the material softening and being crushed in the slip plane. The test ends with a step of friction (9) in the failure plane.

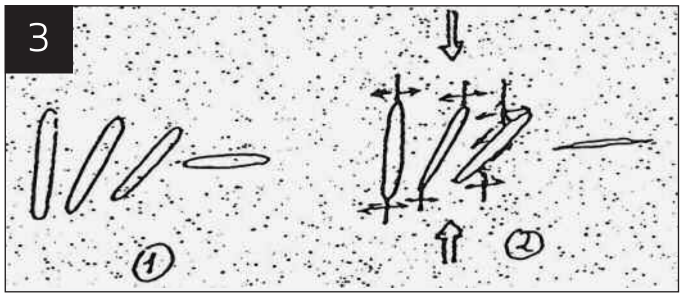
If simple compression is exerted on fragile materials, like glass or very hard rocks or very strong concretes, breaks appear known as column or scale fractures, the orientation of which is almost parallel to the direction of the greatest stress applied. Sound is emitted before failure occurs. They occur before (7) in a triaxial test and correspond to micro-breaks, the sources of which can be identified if various surface sensors are available. The orientation of the micro-breaks is clearly random, because it is a function of the non homogeneous orientations of the existing defects or of the contacts between the constituent minerals of the crystalline rocks or of the different aggregates in the concrete. The number of noises generated in this way increases with the load, but continues to occur over time even under a constant load, without actually being predictable, in such a way that Figure 1 should be completed with Figure 2 if time were included, with (or without) deferred deformation, and that is with stabilisation or even with final failure. Consequently, the peak (7) in Figure 1 must be reduced to the value (7') in the Figure 2 when the construction of a structure is designed to last for a very long period of time.

Measurement of the mechanical properties of geomaterials must therefore be estimated with account taken of the effects of time and of the duration of the loads.

However, the scale effects must also be considered. As is known, these can be characterised by the following observations: the smaller the samples, the stronger they are and the more important the scatter of the results is. One attempt at interpretation is to say that the extreme strength of a rock is conditioned by the presence of a structural defect. The smaller the samples the less likelihood there is that they will contain a serious structural defect. However the more numerous the samples are the higher the probability of encountering different defects with a consequent greater scatter of the results.

The effects of temperature must also be taken into consideration: the higher the temperature, the lower the failure strength. The lower the temperature the greater the strength.

The rheology of geomaterials is without doubt very complex.



3. DIFFERENT ORIENTATIONS OF CRACKS AND THE ASSOCIATED LATERAL DEFORMATIONS

Often jointed or faulted cracks caused by tectonic disturbances are found in geological formations which have been subjected to considerable distortions. Sometimes cracks are "healed" and closed by calcite or by silica. We encounter this phenomenon in situ and also in stones transported by rivers. In these cases it is very difficult to rediscover the orientations of the stresses that generated the initial cracks. However, as Talobre J. (1957) wrote: "It is clear that the veins (of calcite) have been lodged in a crack that has opened."

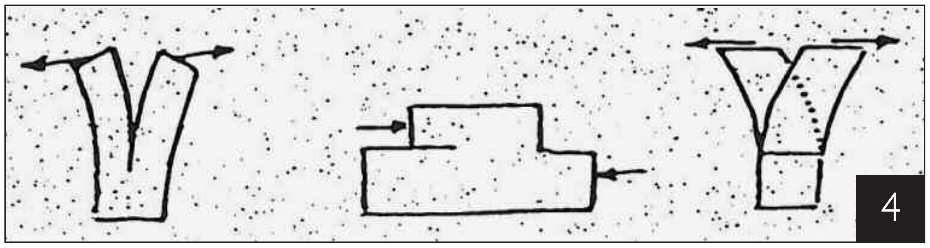
In one of the last conversations that I had with the late Pierre Sirieys on scale networks of cracks, I pointed out that for cracks to open, the rock must have been subjected to tensile stress and he replied that the break may be produced under the effect of excessive transverse stretching connected with the Poisson modulus. He cited triaxial tests on very fragile rocks subjected to relatively weak lateral pressures during which breaks appear in small columns, even if normal compression stresses have been exerted on the cracks. Figure 3 explains the origin of this excessive stretching in a fractured medium in cases other than the initial orientation of the cracks.

Finally it is precisely those breaks that are close to correctly oriented cracks, subjected to axial compressive stresses, which cause a significant percentage of lateral movements and therefore a high Poisson modulus.

However, it is also known that the permeability of rocks is closely connected with cracking and stresses.

Generally, three methods exist by which cracks can develop (Figure 4).

- Method 1 by tensile stress.
- Method 2 by shear stress.
- Method 3 by torsion.



***4.** THREE WAYS IN WHICH CRACKS CAN DEVELOP*

Method 3 is exceptional in rock mechanics and in geology. The particular characteristic of method 1 is that the crack can close completely because the opposite surfaces can knit perfectly. On the other hand method 2 does not allow this and consequently permeability may exist through the crack in the direction of the slip rather than perpendicular to the direction of the cut.

2. THE BEHAVIOUR OF IN SITU ROCKS

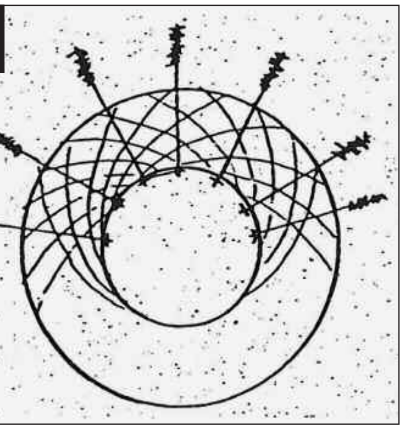
Clearly the behaviour of large masses of in situ rocks is rather different from what happens in a laboratory. Nevertheless, common properties and similar effects are found. For example, the behaviour of rock underground during excavation may be influenced by scale effects, by deferred deformation or anisotropic phenomena. It must also be added that in situ stresses are only really known with sufficient approximation during the first underground operations, something which does not simplify behaviour comparisons.

When rock is very strong or the cavity to be excavated is not very deep, deformation of the medium is limited. It therefore remains within the linear elastic range of behaviour. This is the case, for example, of natural cavities which in most instances are found to be in this condition and the use of supports for them is not necessary! Many rail tunnels have been constructed in this way for many years without causing any significant problems.

However, in a cavity that has been cracked as a result of excavation by blasting, unstable blocks can fall from the roof and these must either be removed or held in place with anchors. At great depths and with weaker and more deformable rocks, the cavity may contract. This can be assessed by measuring convergence between the wall and the roof and between the side walls. These deformations in a horizontal section increase when the face advances and also when deferred deformation occurs. It is not serious as long as the deformation remains small, or small enough that is to remain within the elastic range. In cases to the contrary, when convergence exceeds certain percentage points, support must be installed quickly: shotcrete, steel ribs in greater or smaller numbers and rock bolts. The reason is because this may mean that a plastic zone is beginning to develop and that the rock, after exceeding a peak of maximum strength, is beginning to weaken. This is similar to the phenomena (6), (7) and (8) shown in Figure 1, or (7') in the Figure 2. It is therefore possible to resort to deep anchors using rock bolts in lesser or greater numbers and long enough to be anchored in the elastic zone beyond the plastic zone. This technique has been called suspended reinforcement (Figure 5). However, if the plastic zone is very thick, the zone remaining within the elastic range may not be close to the walls of the tunnel. For example, during the construction of the Frejus tunnel, convergence of approximately one metre was measured in the schistose limestone on the Italian side under very large overburdens. The plastic zone was therefore thicker than the diameter of the tunnel. Consequently, it was not therefore possible to install anchors longer than this diameter to anchor into the elastic range. The rock bolts were therefore grouted along their whole length and not just at the base. The density of rock bolts per square metre and the use of bearing plates enabled the ground to be stabilised before the final lining was installed. In a certain sense the reinforcement of the rock bolts conferred cohesion on the rock mass, thereby limiting plastic deformation around the tunnel.

5. SUSPENDED REINFORCEMENT

5



Generally, when plasticisation around a tunnel is under control, the cracks induced around a tunnel or in situ cracks do not cause concern to a civil or mining engineer. The cracks may even be useful, because they can create drainage around a tunnel and thereby reduce hydraulic pressures behind the lining. This would not be the case if a perfect waterproof seal was desired.

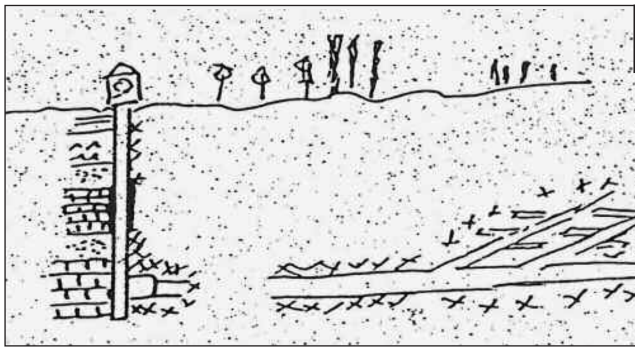
During the construction of a tunnel under a shallow overburden, surface soil deformation – elastic or plastic – may cause damage. This happens, for example, in a city where surface disturbances may occur above underground works in old buildings or in underground utilities.

Generally, with regard to predictions of surface soil movements, two cases can be distinguished: firstly what happens in the short and then in the long term, above the part of the tunnel that has already been excavated and secondly, what happens in the short term ahead of the tunnel face. In the first case, the problem is classic. If we place ourselves in the vertical plane perpendicular to the axis of the tunnel, the deformation calculations are relatively simple, even if the mechanical properties of the materials are less so. However, observations and the results of experiments have made it possible to establish practical rules to predict the magnitude and form of surface settlement as a function of the nature of the ground (Peck. R., 1969).

The problem of the face is obviously more complex, because it is three dimensional to all effects and purposes. Nevertheless, it is naturally limited to the short term only. Excavation with an earth pressure or slurry pressure shield enables surface repercussions to be reduced. Furthermore, the influence of the effects of ground deformation ahead of the face on the surface decreases as the depth of tunnels increases. On the other hand, however, substantial deformation ahead of the face at depth causes great difficulties, because it is difficult to detect and even to measure (because the rock mass that runs into the tunnel is removed as excavation proceeds) and also because it is not possible to install support on the face, except when work is halted temporarily.

3. TOXIC WASTE DEPOSITS

Whether it is chemical, physical, metallurgic, medicinal or nuclear, industry generates toxic waste which must be completely isolated from the living world.
Old mines or underground constructions designed especially as deposits can be used for this purpose. However, one of the great difficulties to be faced is that of water which acts chemically through corrosion of the containers or of the waste itself or which acts physically by transporting toxic substances by means of infiltration.
Waste deposits therefore involve above all the conception of a path which enables us to transport toxic substances to depth, down a shaft or straight or spiral descent tunnel, through more or less permeable geological environments. This path is impermeable at the destination. On the other hand deposits are to a greater or lesser degree surrounded by cracks generated by the works to store the waste. Now between these two, there will be a tunnel to pass between them and it is essential that this tunnel is as impermeable as possible to prevent any connection between the two cracked zones.



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6. SHAFT AND DEPOSIT

4. EXCAVATION DISTURBED ZONE (EDZ)

In order to be able to solve problems caused by the excavation of a tunnel, it is essential to correctly understand what is happening close to the face during tunnel advance. You need to begin with simplest things.
The classic two dimensional load mechanism of a plastic medium with cohesion c and nil internal friction is that of Prandtl. The failure stress qu, is reached when qu = $(2 + \pi)c$ (Fig. 7A). If the ends are loaded and the medium is free between them, Prandtl's model is still valid, but the movements are in the opposite direction to before. On the other hand the limit load is nevertheless the same qu = $(2 + \pi)c$ (Fig. 7B).

In three dimensions, and that is with a circular load on an undefined body, the problem is a little more complicated but it is generally accepted in soil mechanics for a load of qu = 6.4 c. The reverse mechanism (lateral load and free base) is more complicated and it is generally accepted that the pressure under a plate with a circular hole in it is greater than 9,4 c in order to allow the ground to pass through the hole. This situation is in some ways comparable to that which could occur at the face of a tunnel driven through clay.
However, the situation is very different for mediums with cohesionless internal friction because the flow configurations are very different for loading (Figure 8) or for heaving.

The same can be said for mediums with cohesion and internal friction.

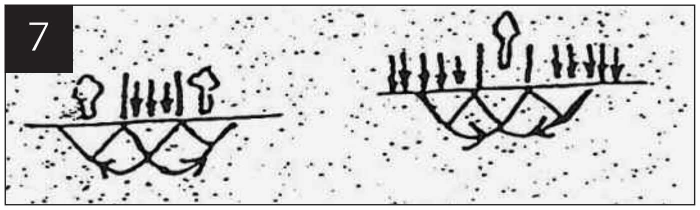
Let us seek to understand how material moves beyond the face when it begins to penetrate towards the inside of a tunnel (Figure 9). When the face approaches the small sample A, this is squeezed to become the small sample B.
The principal stress for this squeezing is perpendicular to the lines of flow. The slip surfaces therefore form an angle of

$$\left(\frac{\pi}{4}-\frac{\varphi}{2}\right)$$

with respect to the normal lines of the general movement.

Everything which lies in the zone of the tunnel advance (dotted line in Figure 9) will be completely destroyed during face advance. Only traces of what lies outside that zone will be found and this is in effect the orientation of the cracks observed in extremely different mediums, such as clay in Belgium or argillites in France.
These indicate a preferred pattern for infiltrations which can circulate in this plastic material. To determine the orientation of these cracks by means of calculation is particularly difficult because only very poor knowledge is possessed of the values for internal friction and cohesion of the rock materials.

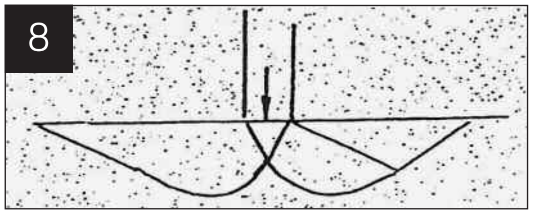
If we now consider Figure 5, we realise that it does not represent the reality at all. Its conception is in fact based on oversimplified assumptions which were chosen to calculate the elastic-plastic equilibrium of tunnels.
We initially assumed that the tunnel was located in a site, the surface of which was rigorously horizontal – a little like a plain. First a tunnel is excavated with a perfectly circular cross section in a medium without gravity. Then, supports are installed. Next, gravity is added to the soil so that the principal stress is very vertical (something which would not occur under the sides of a valley), but equal to the weight of the ground.
It is therefore assumed that the horizontal stresses would all be equal (an assumption that would be almost true for the site of the ANDRA underground laboratory – Gay D. & Al., 2010). And all this is done to analyse how the slip surfaces might be distributed.



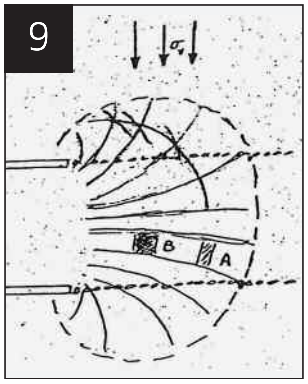
A – PUNCHING INTO THE GROUND

B - UPLIFT OF THE GROUND

7. *PRANDTL'S MODEL*



8. *MEDIUM WITH INTERNAL FRICTION*



9. *BEYOND THE FACE OF A TUNNEL*

Clearly, this does not correspond to the reality at all. And every one knows that because of the gravity gradient, the raising of the ground in a tunnel is less than the lowering of the roof and that there is nothing worse than blocks of stone landing on your head.

It can therefore be clearly seen that the elegant slip surfaces and the relative calculations do not represent the reality at all. In a real tunnel, Figure 5 is subject to the effect of the force of gravity before the supports are installed, always hoping that the elastic limit in the rock mass is not exceeded too much and that the plastic or viscoplastic deformation develops calmly and that it is possible to install the suspended reinforcement.

Furthermore, and in any case, we have forgotten that Figure 5 must be set, before all else, in relation to Figure 9, because generally the excavation of a tunnel always takes place from the face! It is precisely at this stage that the first slip surfaces begin to be produced at the face around the tunnel and in directions that are completely different to those shown in Figure 5.

However, this must not prevent us from studying a working method.

5. HOW TO AVOID THE CREATION OF AN EXCAVATION DISTURBED ZONE (EDZ)

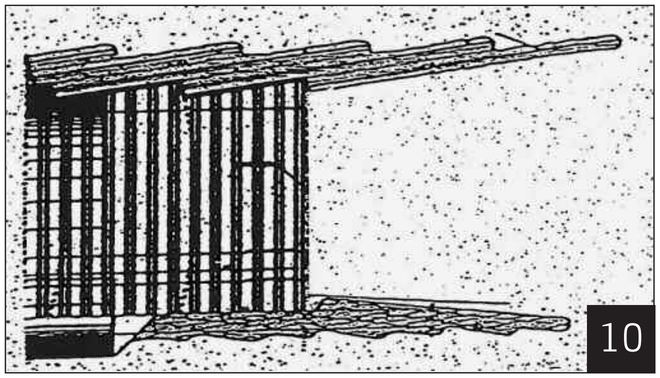
To prevent damage to rock during excavation, deterioration beyond the face caused by movements of the rock mass towards the gaping opening of the tunnel face must be prevented. To achieve this the network of cracks must not be allowed to develop around the zone that is to become the face. Classic methods used to overcome unstable underground zones must therefore be employed. Freezing may be used for example. However, this technique is very slow for large diameter tunnels and requires preliminary thermal studies of the rock and allowance for expansion caused by freezing water.

First of all, support for the tunnel side walls must be extended up to the face. Protection of the face can be provided by means of a support ahead of the face with a truncated cone shape created with overlapping elements excavated with a machine similar to a chain saw pushed into the rock, or with cylindrical elements which intersect each other like co-penetrating columns.

The subsequent excavations are filled with concrete as the various elements advance (Figure 10). The incisions excavated for each of these elements are much smaller than the tunnel. The scale effect is such that this operation will cause almost no cracks in the rock mass.

Widespread use is also now made of face reinforcement using fibre glass elements, with a length of approximately three times the diameter of the tunnel. These can be eliminated as the face advances and replaced with new elements as the work proceeds. The face reinforcement is fixed with anchors positioned far in advance of the face.

However, the best method is obviously that of using both methods at the same time (Figure 12).



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10. *TUNNEL ADVANCE PROTECTED BY A SHELL*

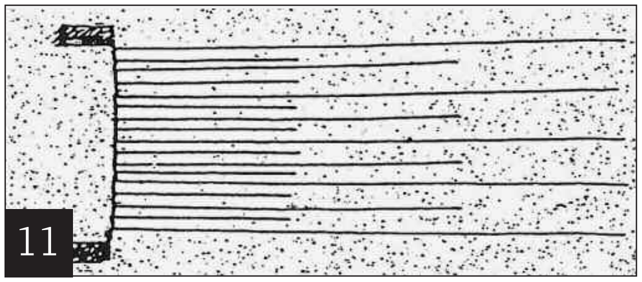
The reality of the rock mass through which a tunnel passes is probably not so simple. More specifically, one principal horizontal stress may be greater than the other two principal stresses, one horizontal and the other vertical. This should translate at the face into specific orientations of rock mass destruction which are easily recognisable. In this case a conical shell must be installed in advance to protect the face which is no longer ring shaped, but which has an elliptical cross section which is appropriately oriented with respect to the stress tensor of the rock mass.
Figure 13 shows the creation of a protected section. Tunnel support is brought close to the face, then the advance shell and the nails in the face are installed simultaneously. Next, conventional excavavation is resumed. In actual fact the method proposed for protected tunnel advance is very probably more costly than conventional tunnel advance methods.

This set of operations for the construction of a protected section can be repeated a certain number of times on different sections of a tunnel to create a waterproof zone. However, it is essential to create one for the connection between a descent tunnel and a deposit (and the same thing naturally also applies to the or the shafts and the deposit).
Either way it is clear that the effectiveness of that method for obtaining a waterproof seal in a section – or of any other method – must be verified by in situ tests performed before work begins on the creation of the deposit itself.

11. *NAILS IN THE FACE*

12. *PROTECTIVE ADVANCE SHELL AND NAILS IN THE FACE*

13. *CREATION OF A PROTECTED SECTION*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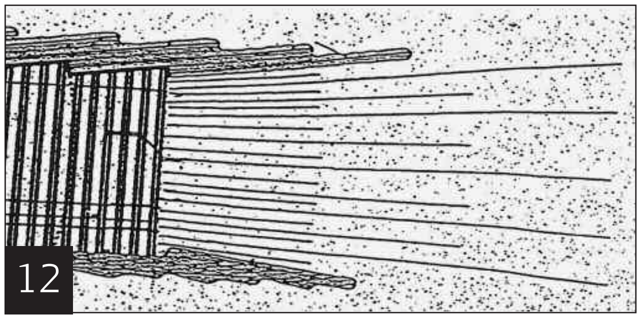
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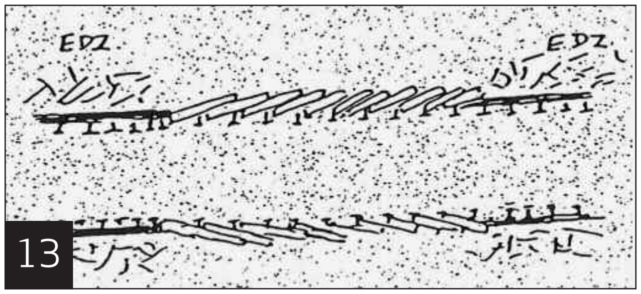

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BIBLIOGRAPHY

ANDRA. Dossier 2005, Argile, tome "Evaluation de sûreté du stockage géologique" (December 2005).

DESRUES J. (1984) "La localisation de la déformation dans les matériaux granulaires". PhD Thesis, 29^{th} July 1984, Grenoble (257 pages).

BÉREST P., GHOREYCHI M., HABIB P., (2000), "Affaissement en surface lors du creusement d'un souterrain", Revue Française de Géotechnique, no. 92, 3^{rd} quarter of 2000, pages 41-47.

GAY O., ALLAGNAT D., MOREL J., ARMAND G. (2010) "Auscultation des alvéoles HA du laboratoire souterrain de recherche de l'ANDRA (CMHM)" - Tunnels et espace souterrain, no. 221, Sept.-Oct. 2010, pages 371-376 and 383-390.

HABIB P. (2008) "Equilibre du front de taille d'un tunnel". Proceedings of the conference on "Le gallerie nelle infrastrutture di trasporto", Verona (Italia), 5^{th} to 6^{th} March 2008).

LUNARDI P. (1989) "Etudes et expérences sur le préconfinement de la cavité et la préconsolidation au noyau du front", Colloque Int. "Tunnels et Microtunnels en terrains meubles", Paris, February 1989, Presses des Ponts et Chaussées.

PANET M. (1995) "Le calcul de tunnels par la méthode convergence-confinement", Presses des Ponts et Chaussées, pag. 93.

PECK R.B. (1969) "Deep excavation and tunnelling in soft ground", 7^{th} Int. Conf. Soils Mech. and Found. Eng., 1969, State of Art, pages 225-290.

SIRIEYS P. (2002) "Géométrie et graphe des déformations rotationnelles". Revue Française de Génie Civil, vol. 6, no. 7-8/2002, pages 1241-1256.

TALOBRE J. (1957) "La mécanique des Roches" , Ed. Dunod, page 36.

WILEVEAU Y., BERNIER F. "Similarities in the hydro-mechanical response of Callovo-Oxfordian clay and Boom clay during gallery excavation". "Clays in natural and Engineered Barriers for Radioactive Waste Confinement", 3^{rd} Int. Meeting, Lille, 17^{th} to 20^{th} September 2007, page 149.


Surface settlements
JLE, St James's Park (Nyren et al. 2002)
20
40
60
mm
Feb 98
eastbound
westbound
5 10 m

Kalman Kovári

YIELDING SUPPORTS IN TUNNELLING

Lining concepts with yielding support for squeezing and swelling rock have deep historical roots in the international tunnelling literature. Recent technological developments permit their application on large scale and in an industrialized manner. Experience shows that it is highly economical to allow controlled deformations of the rock and at the same time to exert a considerable lining resistance. Instabilities and cumbersome cleaning up of destroyed profiles can be avoided.

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1. INTRODUCTION

Although the processes taking place in the ground around a tunnel in squeezing and in swelling rock differ from each other fundamentally, there is one common feature in both cases: with increasing rock deformation the rock pressure decreases (Kovari, 2009). This fact is proved both by experience and theoretical investigations and was clearly recognized as early as at the beginning of the last century. "*With each fraction of (a) millimetre with which the rock mass moves, the amount of pressure acting on the lining decreases*". (Wiesmann, 1914). Based on this observation a number of design methods are nowadays at the disposal of the engineer to control rock pressure even in heavily squeezing and heavily swelling rock. Both the temporary and the final lining can be constructed nowadays in such a way as to exert stabilizing pressure on the rock and at the same time allow the rock mass to deform. In many cases this combined action, i.e. rock support and letting the rock deform, not only presents the most economical solution, in some cases it is the only one that makes tunnel construction feasible. One must bear in mind that in modern traffic infrastructure projects (e.g. high speed rail connections), apart from stabilizing the opening also the limitation of deformations during the long operation life of the tunnel may become a formidable problem to be overcome. This is possible by limiting considerably the maximum pressure on the permanent lining that could develop in the long term and with it the maximum lining deformations or/and lining displacements. It is obvious that the differential displacements with respect to the tunnel lining are crucial rather than the absolute ones. It is well known that the squeezing and swelling potentials along a tunnel are not uniform and also that the development of the corresponding pressures in the long term may be extremely variable.

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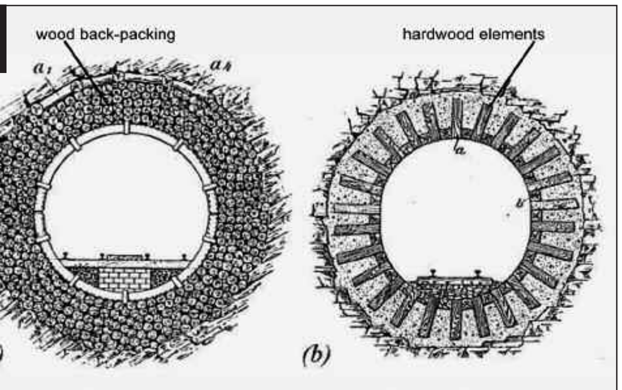
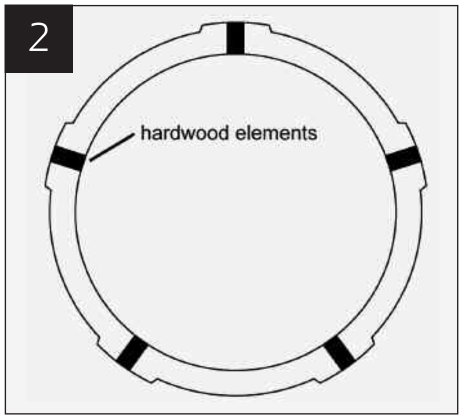


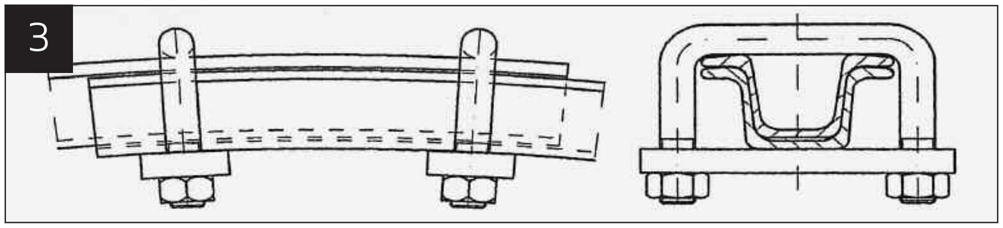
1. *YIELDING SUPPORT CONCEPTS*

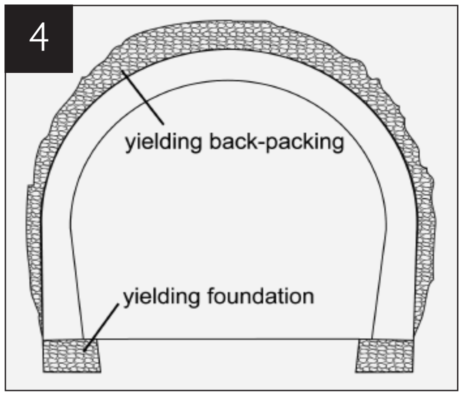
(A) BACK-PACKING WITH WOOD BETWEEN STEEL SUPPORT AND ROCK, (B) INTERLAYER OF WOODEN PANELS IN THE CONCRETE LINING (HEISE AND HERBST, 1913)

The need for yielding types of temporary or final support when tunnelling in squeezing and swelling rock has long been recognized. Recently Anagnostou and Cantieni (2007) have shown two historical examples for yielding support from mining in squeezing ground, which clearly demonstrate two conceptually quite different approaches (Fig. 1). On the one hand, a layer of sufficiently compressible material is inserted between the excavated rock surface and the lining and, on the other hand, the lining itself is made highly deformable (Heise & Herbst, 1913).

In both cases an adequate overexcavation is required to accommodate the expected rock deformations. According to Fig.1 (a) a wood back-packing of sufficient thickness serves as "yielding material " whereas according to Fig.1 (b) compressible wood interlayers serving as "yielding elements" are inserted into the concrete lining allowing it to converge. Later, for both concepts much more practicable solutions were developed. For example, in squeezing ground Mohr (1957) proposed applying highly compressible fuel ash between the lining and the rock of a deep shaft instead of wood. In this context, Mohr provided the first representation of the characteristic line of the rock mass together with that of the yielding support (Kovári, 2003).







2. YIELDING HARDWOOD ELEMENTS BETWEEN PREFABRICATED CONCRETE SEGMENTS (LENK, 1931)

3. SLIDING STEEL RIB CONNECTION (FRÖHLICH, 1948)

4. TUNNEL LINING IN SWELLING ROCK: YIELDING BACK-PACKING IN THE ROOF AND YIELDING FOUNDATION (SCHÄCHTERLE, 1926)

In the 1930s Lenk (1931) reported on a patented method consisting of placing a limited number of yielding wooden elements between prefabricated concrete segments, in this way also providing joints almost free of bending moments (Fig. 2). The deformation characteristics of these wooden elements were determined experimentally.

The first type of yielding support of broad and continuous application, both in mining and in tunnelling, was provided by the so called Toussaint-Heintzmann steel ribs. This involved the design of new steel profiles (top hat cross-section) with friction connecting loops (Fig. 3) permitting the tunnel to withstand larger convergence with more or less constant lining resistance. This marked the beginning of the first industrially-produced supports in squeezing rock, by means of which ground pressure could be reduced with increased convergence (Fröhlich 1948). As the friction resistance in the joints is very limited with respect to the full load-bearing capacity of the ribs, the lining's resistance to rock convergence is also relatively small.

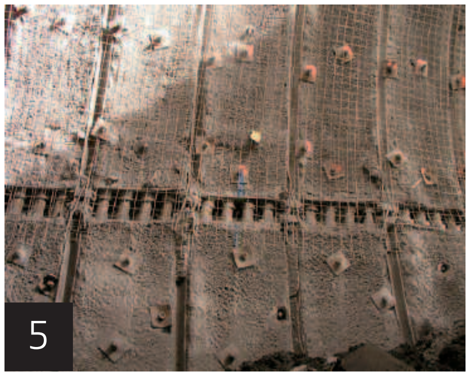
An early attempt to master tunnelling in heavily swelling ground by inserting a yielding medium between lining and rock was reported by Schächterle (1926). As can be seen in Fig. 4, firstly the lining was founded on a compressible layer of rock debris and secondly in its upper part a compressible layer of the same material of 1 m thickness was placed. In fact, in the course of time the roof heaved by approx. 1.3 m, necessitating the reconstruction of the tunnel, which correctly involved the placement of an invert arch.

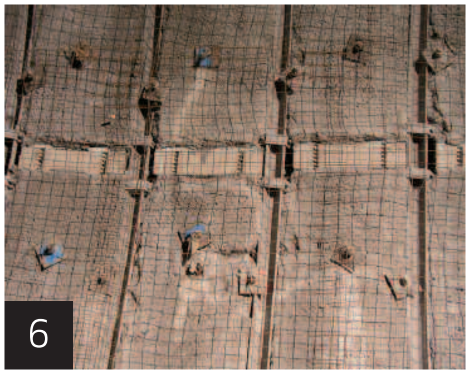
2. YIELDING SPRAYED CONCRETE SUPPORT

An ordinary sprayed concrete lining exhibits a high lining resistance but an extremely low deformation capacity. If it is overloaded, it generally loses its load-bearing capacity due to brittle failure even it is reinforced by the customary steel mesh.

Therefore, a sprayed concrete lining without special measures is not suitable in applications under the conditions of squeezing or swelling rock. However, if the "stiff" concrete lining is provided with a number of yielding elements, as proposed in Fig. 2, allowing the contraction of the profile and exerting at the same time resistance to rock deformation, the sprayed concrete lining becomes a particularly powerful means of controlling rock pressure.

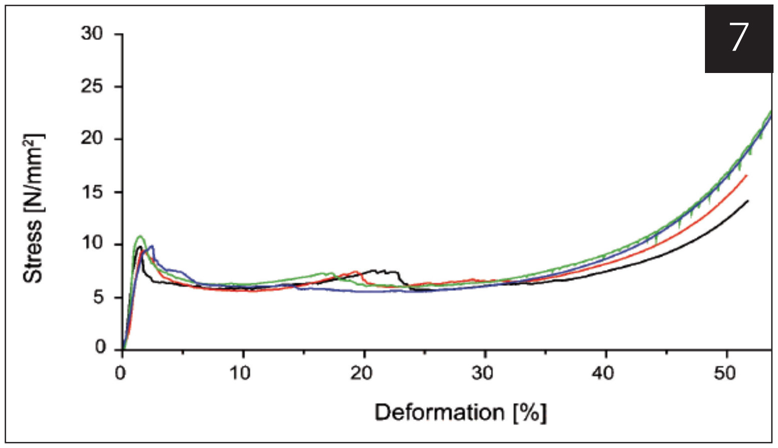
Recently, new types of yielding elements have been proposed and applied successfully in practice. One such element consists of steel cylinders inserted into gaps in the shotcrete lining and loaded axially in the circumferential direction of the profile (Moritz, 1999).

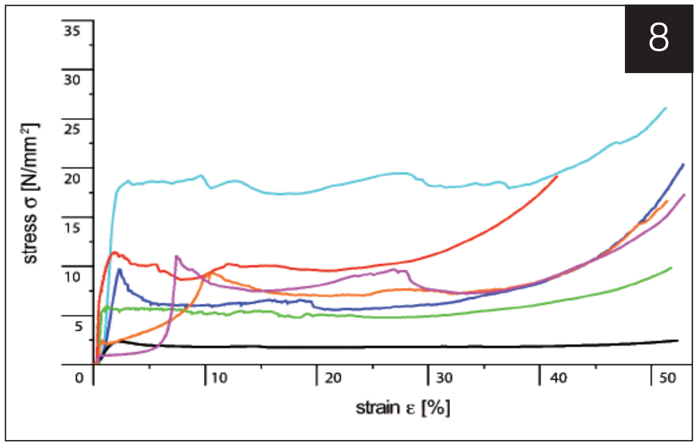




After a given initial critical load the cylinders start to buckle and continue to do so in successive steps, undergoing shortening and thus allowing at the same time a lining resistance to develop. The photo in Fig. 5 illustrates the application in one of the slots in a shotcrete lining.
Further progress in this field was achieved by the development of highly compressible bulk elements on a cement basis. They are composed of a mixture of cement, sand, hollow glass particles, steel fibres and additives and are also provided with suitable steel reinforcement (Thut et al 2006). In Fig. 6 an application is shown in the 37 km long Lötschberg Base Tunnel (Switzerland) driven through highly deformable coal schist under a large overburden (Keller, 2005).

The compressibility of these "concrete" elements amounts to up to 40-50 %, depending on the selected yielding stress (4 ÷ 20 MPa). Fig. 7 shows the results of laboratory tests carried out on such elements, illustrating the high reproducibility of their deformation properties. As can be seen, after reaching a given peak stress of approx. 10 MPa, there is a practically constant yielding state with a stress level of approx. 7.5 MPa, which after roughly 40% compression is followed by strain hardening. This type of element does not exhibit sudden brittle failure - on reaching the full deformation capacity, the strength of the element increases.
In Fig. 8 the various stress strain diagrams illustrate the great possibilities of the highly deformable concrete elements for wide applications are shown. By means of adequate selection of composition, form and reinforcement it is possible to produce elements for very specific applications.
According to the initial strength of the shotcrete and the estimated or measured rate of convergence immediately after an excavation stage the deformability and strength characteristics of the element can be predefined.





***5.** STEEL CYLINDERS INSERTED INTO SLOTS OF THE SHOTCRETE LINING*

***6.** HIGHLY COMPRESSIBLE CONCRETE ELEMENTS INSERTED IN THE SHOTCRETE LINING*

***7.** STRESS-STRAIN BEHAVIOUR OF YIELDING CONCRETE ELEMENTS (TYP SMLP)*

***8.** DEFORMATION CHARACTERISTICS OF VARIOUS ELEMENTS ACCORDING TO SPECIFIC APPLICATIONS*

3. APPLICATIONS IN SQUEEZING ROCK

In underground construction, it is frequently observed that the excavation of an opening leads in some circumstances to major short- or long-term rock deformations, which cause a progressive contraction of the opening (Kovári,1998). If the phenomenon develops completely, the rock penetrates the opening from all sides including the tunnel floor. In such cases the main task is to limit the rock deformations by means of a temporary support. Often this does not succeed because the temporary support is not able to withstand the rock deformations and is either damaged or completely destroyed. Without appropriate countermeasures the rock, so to speak, slowly pushes the destroyed lining in front of it until the movements come to a standstill or lead to a collapse of the opening. One of the countermeasures consists of introducing yielding steel ribs (Fig. 9) together with rock anchors. Another concept consists of a yielding sprayed concrete lining support combined with a light yielding steel support. In the following, examples will be given for both types of application.

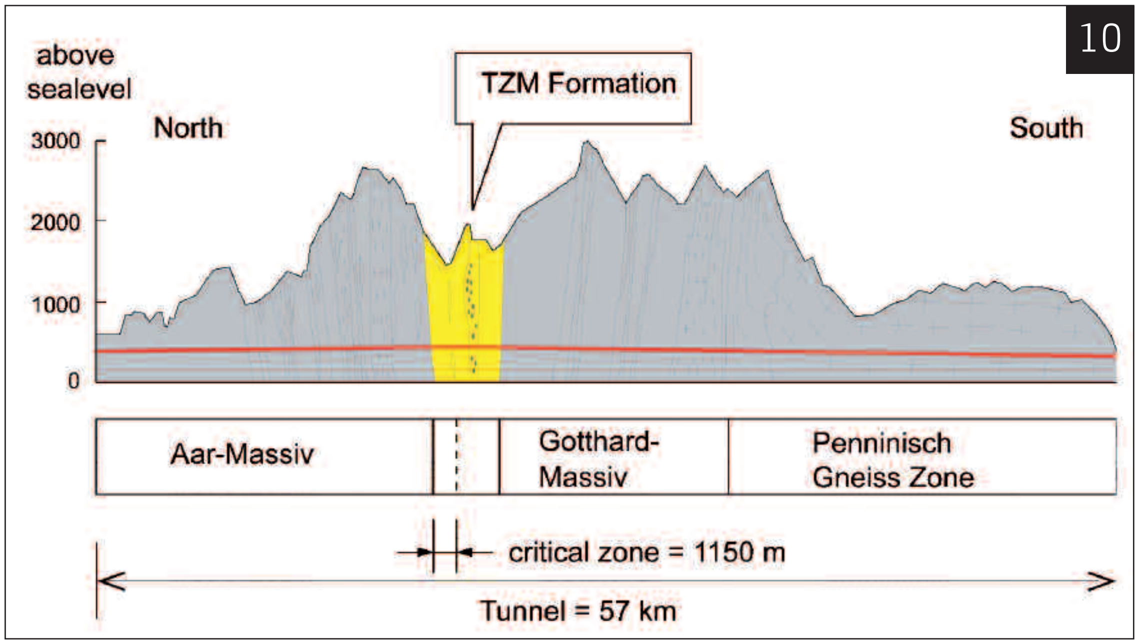
3.1 Gotthard Base Tunnel with yielding steel ribs

9. *YIELDING STEEL SUPPORT*
(A) DETAILS OF THE STEEL RIB CONNECTION
(B) EXAMPLE FOR APPLICATION IN MINING

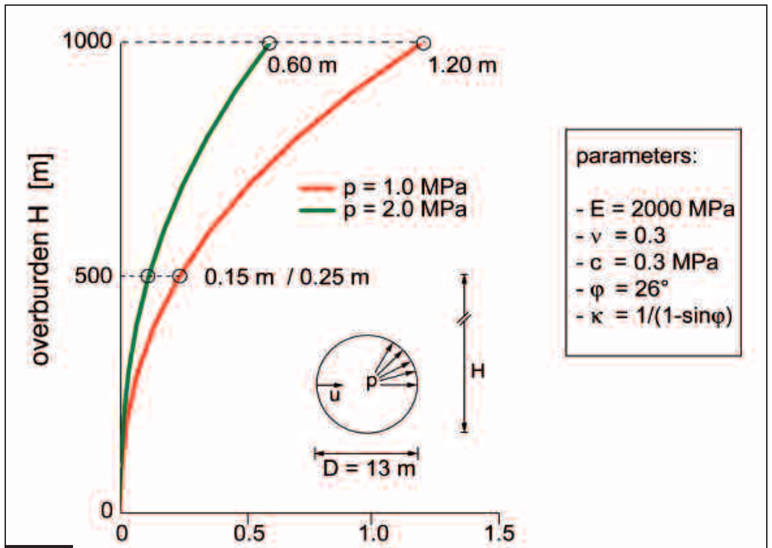
10. *SCHEMATIC REPRESENTATION OF THE LONGITUDINAL GEOLOGICAL SECTION WITH THE SQUEEZING TZM-FORMATION*

In the central part of the 57 km long twin tube Gotthard Base Tunnel driven through the Swiss Alps a stretch of 1150 m of the so called TZM Formation was predicted to be highly squeezing (Fig. 10). In fact, a number of deep exploratory boreholes with lengths up to 1750 m have revealed a rock of very low strength and high deformability, consisting of schists and phyllites.

In this part of the tunnel (excavation diameter Ø=13 m) the overburden was approx. 800 m. From laboratory tests and comprehensive statical calculations it became clear that the



tunnel could only be constructed if radial displacements up to 0.70 m were permitted (Kovári and Ehrbar, 2008). In order to stabilize the opening, the corresponding lining resistance had to be increased to 2 MPa. Fig. 11 illustrates the relation between the overburden H, the radial displacements u and the lining resistance p for the representative rock mass parameters listed in the figure.



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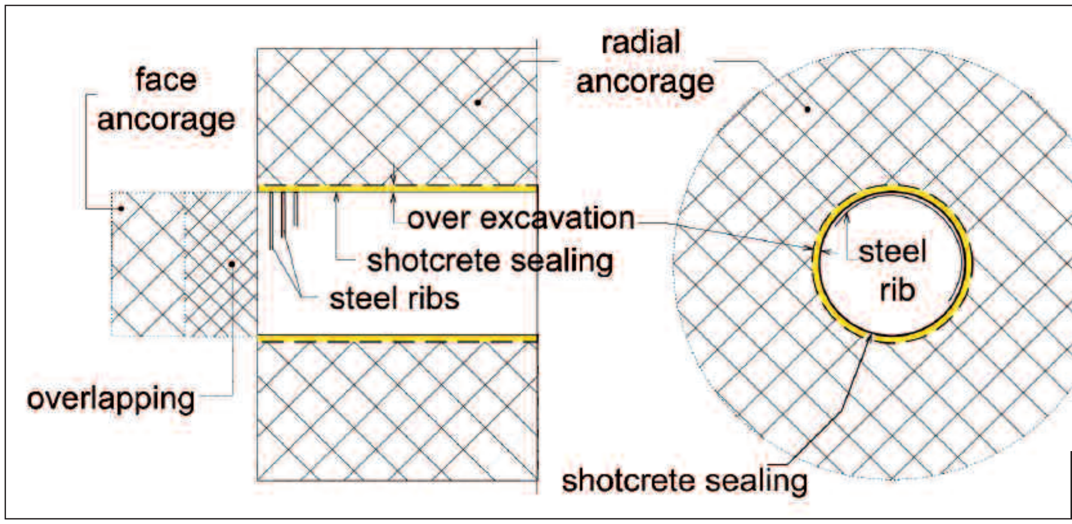
It can be seen that for an overburden of 500 m the radial displacements amount to 0.25 m (p = 1.0 MPa) and 0.15 m (p = 2.0 MPa), respectively. Doubling the height of the overburden to 1000 m the radial displacements at p = 1.0 MPa increase their value five-fold, i.e. 1.2 m. For a lining resistance of 2 MPa the displacements decrease to 0.6 m. The excavation-support system (Fig. 12) involved a nearly circular profile (Ø=13 m) excavated in full face (Lunardi, 1998) and systematically supporting the tunnel face by means of long fully-grouted steel anchors.
As to the support in the cross section, the emphasis was placed on yielding steel ribs of the heaviest type (TH44/58) with a spacing of 0.33-1.25 m. Additionally fully-grouted radial rock anchors with a total length of up to 300 m were placed. A thin shotcrete lining applied immediately after an excavation step had the sole function of sealing the rock surface. This temporary lining concept permitted radial displacements up to 0.70 m in a regular manner. To accommodate this rock convergence, it was necessary to provide space by means of overexcavation. After the convergence capacity of the steel sets has fully been exhausted a sprayed concrete lining of a thickness of 0.40 m was placed in order to substantially increase the lining resistance. This most critical 1.1 km long stretch of the Gotthard base tunnel was completed without the necessity of re-profiling.

__11.__ RADIAL DISPLACEMENT U VERSUS HEIGHT OF OVERBURDEN H FOR TWO VALUES OF LINING RESISTANCE P
(KOVÁRI AND EHRBAR, 2008)

__12.__ SCHEMATIC REPRESENTATION OF THE EXCAVATION-SUPPORT CONCEPT
(KOVÁRI AND EHRBAR, 2008)



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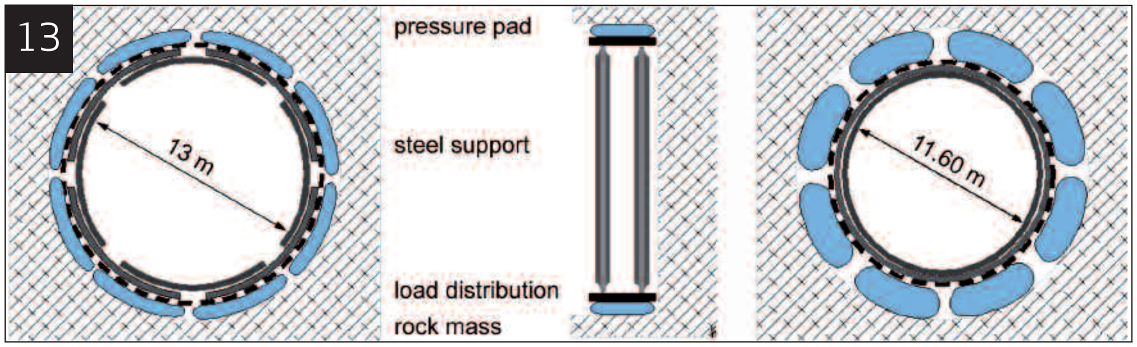
Yielding steelsupport

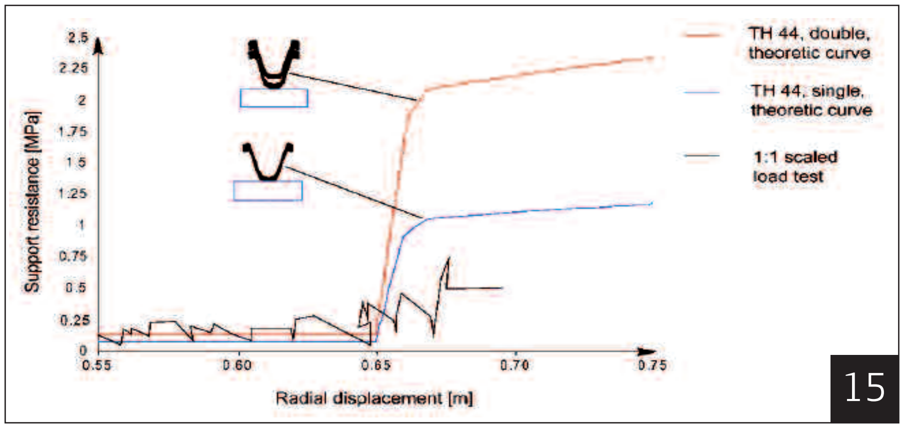
It was planned to place two complete sets of TH 44/70 - type ribs within each other to allow the large movements, involving in total 8 frictional sliding connections. Theoretical considerations indicated a maximum lining resistance of 2 MPa for the case of complete ring closure. The question was whether the system would really behave as predicted considering the extreme high loads combined with the unusually large displacements (sliding) of the individual ribs with respect to each other. To perform the test a niche of adequate dimensions was excavated in hard massive gneiss and two sets of ribs were erected with a spacing of 0.5 m and stabilized to prevent movement out of the plane of the rings.
Liner plates were placed on the ribs to accommodate large inflatable rubber cushions using water as the pressurizing medium.

Fig. 13 shows the test set up schematically, while Fig. 14 shows details of the ribs loaded up to their bearing capacity. The most important results of this test can be summarized as follows: During the sliding process in the joints there is only a very modest frictional resistance resulting in a lining resistance of less than 0.25 MPa as shown in Fig. 15. At a radial displacement (corresponding to convergence) of approx. 0.65 m, the lining resistance increases but does not exceed 30% of its theoretically expected value. Due to local buckling of the ribs, the bearing capacity of the double-rib only corresponds to about 50% of that of a single rib. However, the ability of the double-rib ring, with its connections, to allow a radial convergence of 0.7 m was well confirmed.



13. FIELD TEST OF A LARGE DIAMETER STEEL SUPPORT EXECUTED IN A HARD ROCK NICHE WITH TWO COMPLETE STEEL SETS
(A) THE SETS PRIOR TO LOADING
(B) THE SETS AFTER LOADING BY WATER-INFLATABLE CUSHIONS AND CONVERGENCE UP TO 0.7 M
(KOVÁRI ET AL, 2005)

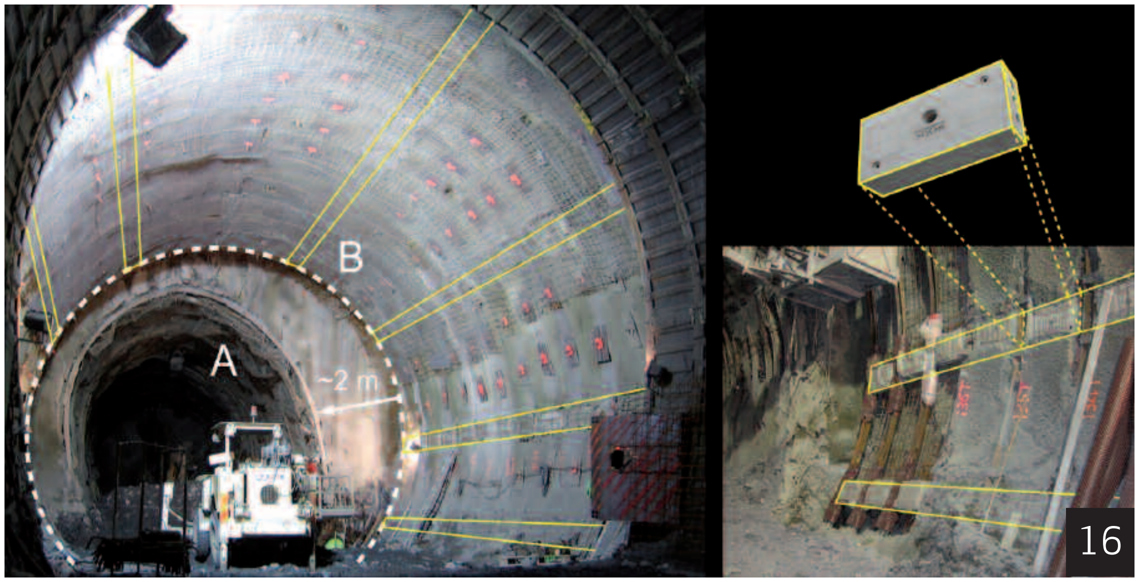
14. STEEL ARCHES LOADED UP TO LOCAL BUCKLING OF THE RIBS (TH44)



15. TEST RESULTS AND THEORETICALLY EXPECTED DIAGRAMS FOR SINGLE AND DOUBLE RIBS

3.2 Saint Martin la Porte access adit with yielding concrete elements

This adit - currently being excavated - will provide construction access to the 53 km long twin tube base tunnel of the new Lyon -Turin high speed rail link (Mathieu 2008).
Exceptionally severe convergences have occurred in carboniferous formations with black schists, sandstones, clay-like shales interspersed with layers of coal, with an overburden of 250 to 350 m. The excavation profile is 77 m^2 to 125 m^2 for a final internal profile of 54 m^2 to 63 m^2.
The temporary support consisted initially of dense radial bolting around the profile including the invert together with yielding steel ribs (TH44/58) and a 200mm thick shotcrete lining interrupted by 4 or 5 longitudinal slots. Due to these slots the shotcrete lining could not develop any support effect for the rock. The greatest convergence occurred after 145 days at a distance of 60 m from the working face and exceeded 2 m.
Convergence rates varied from 30-50 mm/day at the face with 50% of total deformation taking place in the first 20 m (Mathieu 2008). In order to better control the rock deformations, i.e. to avoid the necessity of cumbersome, costly and time-consuming re-profiling a novel support system was implemented.
This involved a near circular cross section with the insertion of the yielding concrete elements described above into 9 longitudinal slots in the sprayed concrete lining.



16. *LYON–TURIN HIGH SPEED RAIL LINK, ACCESS ADIT:*
CONVERGENCE UP TO 2 M (PROFILE A)
RE-PROFILING AND APPLICATION OF YIELDING CONCRETE ELEMENTS (PROFILE B)

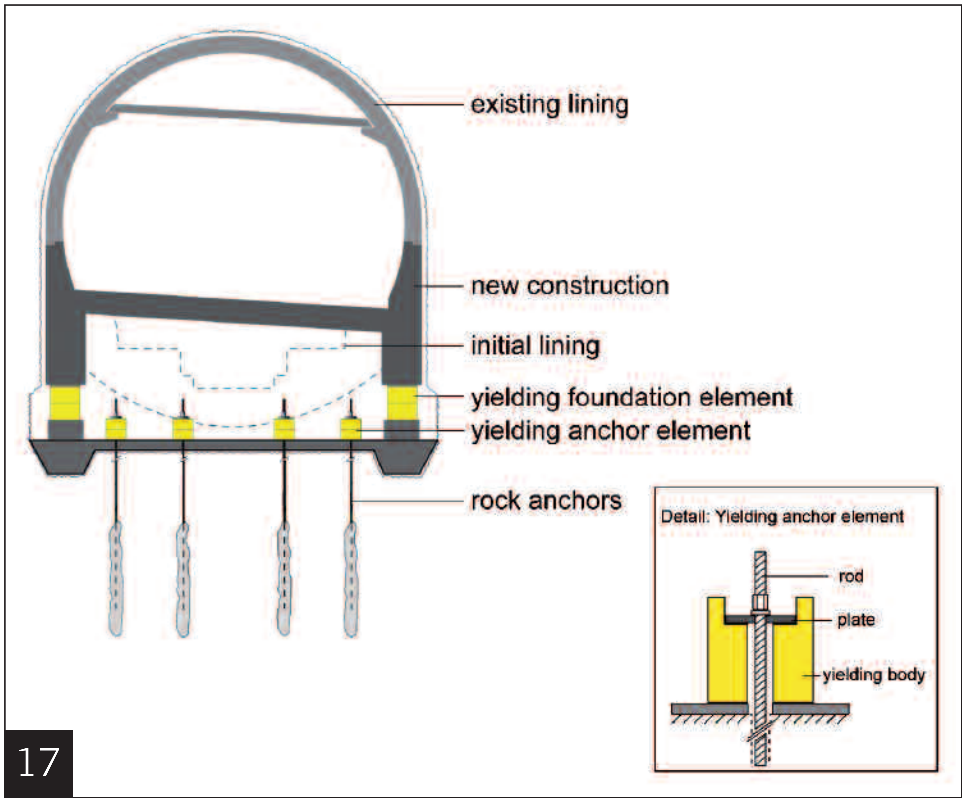
The choice of this countermeasure was based on earlier experience made in the 37 km long deep Lötschberg base tunnel (Keller, 2005).
The beam-shaped elements (height 400 mm, length 800 mm and thickness 200 mm) were designed to yield at approx. 40% compression (Barla et al, 2008). It was verified, by means of an extensive field monitoring programme, that the elements incorporated into the lining were capable of shortening under a nearly constant tangential stress of 8.5 MPa. The system adopted in the Saint Martin La Porte access adit proved to be

very successful. In tunnels through squeezing rock, realized in full face excavation, steel arches with yielding joints and yielding elements in the sprayed concrete lining present a consistence concept for rock support.
A great advantage stems from the fact that the overall deformation of the cross section is less than with the traditional rock support with yielding steel sets alone (Cantieni, Anagnostou 2009).

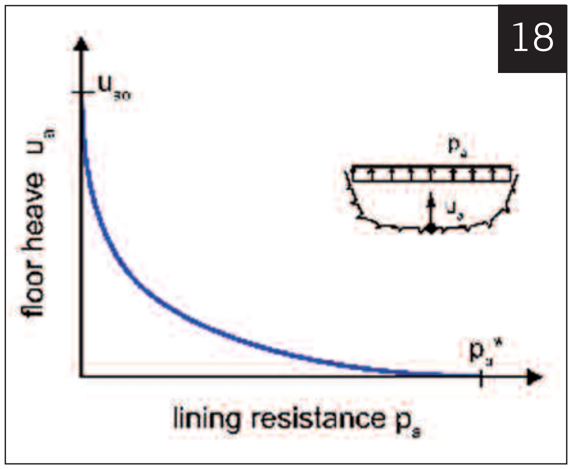
4. APPLICATIONS IN SWELLING ROCK

Rocks containing clay minerals or anhydrite increase in volume when they come into contact with water. This phenomenon is referred to as rock swelling. Tunnelling in swelling rock normally causes two different types of damage. The first type results in the failure of the invert arch due to the pressure from the surrounding swelling rock. The second type occurs under low overburden conditions, in which the tunnel lining results in heave of the entire tunnel and initially may remain only slightly damaged. The tunnel crown and floor experience an upward displacement which leads to limitations or even loss of serviceability (Kovári et al, 1998).
The 1.5 km long Chienberg Road Tunnel in Switzerland, penetrating a heavily swelling anhydrite formation (Gipskeuper), well illustrates this situation. It was designed with a circular cross section and a 1.0 m thick concrete final lining to resist high swelling pressure and excavated with the heading and bench method. As the heading was nearing completion in two individual stretches swelling caused the entire tunnel profile to heave by up to 100 mm. The overburden was modest and the rock located over the roof was very soft. Damage first affected a 60 m long tunnel section and another 370 m section of tunnel (Hofer et al, 2007).
These two tunnel sections were redesigned according to the concept of "Modular Yielding Support" (Kovári and Chiaverio 2007). This involved the application of yielding concrete foundation elements placed under the lining pillars (Fig.17). Other yielding concrete elements were used for the heads of tie-back anchors mounted on the tunnel floor.

To implement this plan the concrete floor of the tunnel lining in the two affected sections had to be removed in stages.
A 6 m deep trench had to be excavated below the original floor to build the new floor for the modified system.



17. CHIENBERG ROAD TUNNEL IN HEAVILY SWELLING ROCK: "MODULAR YIELDING SUPPORT"

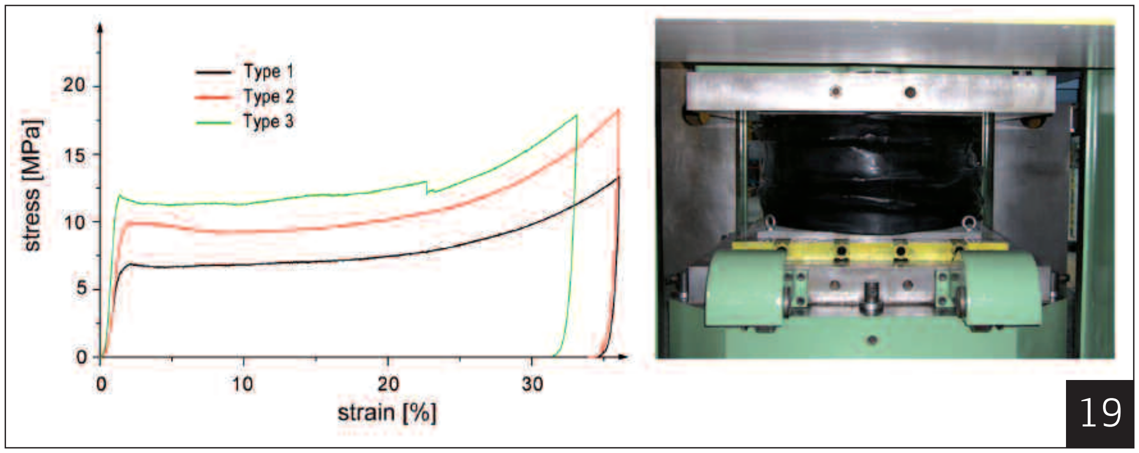


18. THE QUALITATIVE RELATIONSHIP BETWEEN FLOOR HEAVE U_A AND THE LINING RESISTANCE P_A OF SWELLING ROCK P_A (KOVÁRI ET AL, 1998)

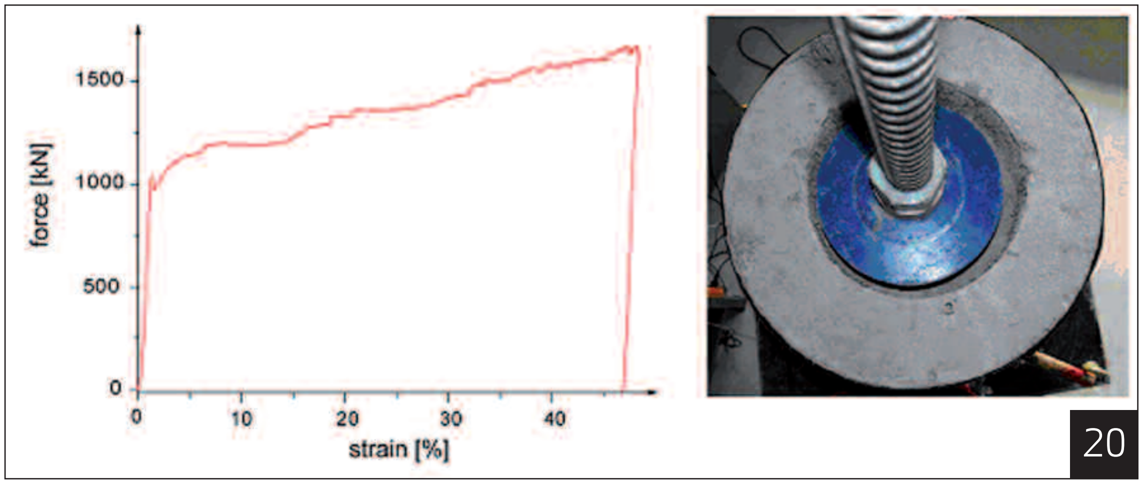
The new carriageway slab is 4 m above the new floor and has bending-resistant connections to the remaining tunnel structure (In figure 17 the dashed line shows the original profile).
The concept of "Modular Yielding Support" is based on the diminution of swelling pressure due to permitted floor heave. Figure 18 shows the qualitative relationship between floor heave u_a and the lining resistance p_a of swelling rock (Kovári et al, 1998). Permitting long term floor heave u_a by using deformable elements results in less vertical stress p_a.

The foundation elements applied at Chienberg Road Tunnel having a height of 1000 mm and diameter of 900 mm were designed in 3 different load classes for the variable overburden along the two heavily swelling stretches. Each type has defined minimum and maximum levels of load resistance. The minimum level prevents tunnel settlements; the maximum level protects the tunnel against overstress and heave.
Within the specified limits, a deformation range of 30-40 % of the original height of the elements can develop (depending on the selected yielding stress). The load capacity of each element type was customized by varying the constituents and the reinforcement within the elements. To configure the different types of element for their design parameters, several tests had been conducted on a 20 MN load testing equipment. Figure 19 shows the results of uniaxial compression tests in the laboratory carried out at cylindrical foundation elements.

The yielding anchor elements in the floor were installed in order to reduce the rate of the floor heave. The elements for the anchor heads are based on the principle of penetrating the anchor plate with a smaller diameter than that of the yielding element



19. LABORATORY TESTS OF CYLINDRICAL FOUNDATION ELEMENTS
(HEIGHT: 1000 MM / DIAMETER 900 MM)
(LEFT) ELEMENTS WITH DIFFERENT STRENGTH LEVELS
(RIGHT) AN ELEMENT COMPRESSED AT 30 % (WITH RUBBER SKIN)



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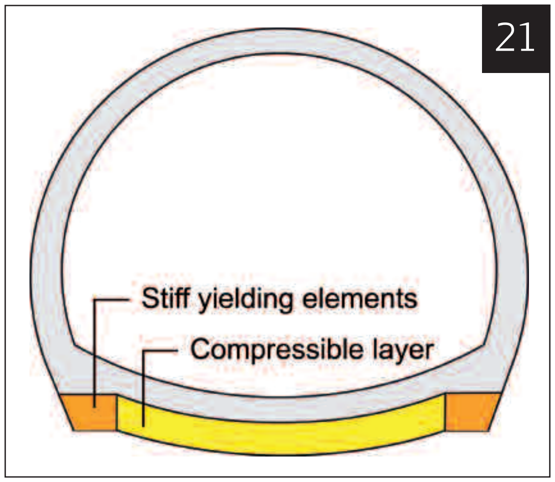
20. *LABORATORY TEST OF A CYLINDRICAL ANCHOR ELEMENT*

(HEIGHT: 600 MM / DIAMETER 600 MM DIAMETER LOAD PLATE 350 MM)

(LEFT) FORCE-STRAIN DIAGRAM OF AN ANCHOR ELEMENT
(RIGHT) AN ELEMENT COMPRESSED AT 40%



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21. *COMBINATION SUPERFICIAL AND SINGULAR YIELDING SUPPORT*

(see also the detail in fig. 17). This system also functions perfectly well if there is some eccentricity of the force transmission (anchor force). Figure 20 illustrates a test result carried out with such an anchor element.

The highly deformable concrete elements at the Chienberg Road Tunnel are designed for a deformation endurance of about 25 years. The advantage of the "Modular Yielding Support" system is that it enables observing and replacing the elements at any time without affecting the traffic in the tunnel. The elements can individually be replaced after reaching their deformation capacity.

A further potential application is offered by the Modular Yielding Support as a combination of superficial yielding support (under the invert) and individual (modular) yielding support under the foots of the vault (Fig. 21).

5. CLOSING REMARKS

Although the physical and chemical processes taking place in the ground around a tunnel in squeezing and in swelling rock differ from each other, there is one fundamental aspect in these two cases: with increasing rock deformation the rock pressure decreases. This is proved both by experience and theoretical investigations. Based on this fact, nowadays a number of design methods are at the disposal of the engineer to control rock pressure even in heavily squeezing and heavily swelling rock.
The need for the construction of long deep tunnels - as is the case under the Alps in Austria, France, Italy and Switzerland - has made the problem highly relevant.

In fact, the heavily squeezing rock zones under high overburden in the 34 km long Lötschberg Base Tunnel and the 56 km long Gotthard Base Tunnel in Switzerland could recently be successfully overcome by introducing new design and constructional methods. The key element of the design of the temporary rock support was the fulfilment of the requirement to allow controlled radial displacements up to 0.7 m. The steel support is provided with sliding joints and yielding beam elements are inserted in the shotcrete lining. In this way the lining is capable of providing considerable rock support (so called lining resistance) and at the same time also permitting convergence leading to a reduction of rock pressure for the final lining.

22. *CHIENBERG ROAD TUNNEL AFTER COMPLETION: "MODULAR YIELDING SUPPORT"*

In the case of swelling rock containing clay and/or anhydrite the problem stems from the ca-pacity of these rock types to increase their volume by absorbing water and thus lead to heave of the base of the tunnel. The solution to the problem is provided again by designing a lining system that allows a given amount of base heave without violating operational requirements. Inserting highly compressible materials of a specified high resistance between rock and invert provides a satisfactory solution.

BIBLIOGRAPHY

ANAGNOSTOU, G. AND CANTIENI L. (2007), "Design and analysis of yielding support in squeezing ground" Proceedings of 11th ISRM congress, The second Half-Century of rock Mechanics, July 9 - 13, 2007, Lisbon, Portugal, pp. XX

AMSTAD, CH. AND KOVÁRI, K. (2001), "Untertagbau in quellfähigem Fels", Bundesamt für Strassen (ASTRA), Schweiz, Forschungsbericht 52/94, pp. 46

BARLA, G., RETTIGHIERI, M., FOURNIER, L., FAVA, A., TRICLOT, J. (2008), Saint Martin Squeeze, Tunnels & Tunnelling, Focus on Europe (May), pp. 15-19

CANTIENI, L. AND ANAGNOSTOU, G.(2009), "The interaction between yielding supports and squeezing ground" Tunnelling and Underground Space Technology, Vol. 24, No. 3. (May 2009), pp. 309-322

FRÖHLICH, K. (1948), "Die Verbindung stählerner Streckenbögen. Glückauf" pp. 543-555

KELLER, M. (2005), "Die Beherrschung der Hohlraumverformungen in der Karbonzone (Lötschberg Basistunnel)", Swiss Tunnel Congress Luzern, Dokumentation D 0229, Vol. 4, Swiss Tunnelling Society

KOVÁRI, K., (1998), "Tunnelling in Squeezing Rock", Tunnel 5/98, pp. 12-31

KOVÁRI, K., AMSTAD, CH., ANAGNOSTOU, G. (1988) "Design/construction methods - Tunnelling in swelling rocks": Key Questions in Rock Mechanics, Proc. of the 29th U.S. Symposium, Minneapolis, Cundall et al. Balkema, Rotterdam, pp. 17-32

KOVÁRI, K. (2003), "History of the sprayed concrete lining method - part I", Tunnelling and Underground Space Technology 18, Elsevier Science Ltd, pp. 57-69

KOVÁRI, K., EHRBAR, H., THEILER, A. (2005), "Druckhafte Strecken im TZM Nord: Projekt und bisherige Erfahrungen, Proceedings of Geologie und Geotechnik der Basistunnel am Gotthard und am Lötschberg", vdf Verlag ETHZ, pp. 239-252

KOVÁRI, K., CHIAVERIO, F. (2007), "Modulares Knautschsystem für Tunnel in stark quellfähigem Gebirge", STUVA TAGUNG'07, Forschung + Praxis 42, bau verlag, Gütersloh, pp. 195-200

KOVÁRI, K. AND EHRBAR, H. (2008), "Gotthard Basistunnel, Teilabschnitt Sedrun - Die druckhaften Strecken im TZM Nord: Projektierung und Realisierung, Swiss Tunnel Congress Luzern, Dokumentation D 0229, Vol. 7 Swiss Tunnelling Society

KOVÁRI, K. (2009), "Design methods with yielding support in squeezing and swelling rocks" World Tunnel Con-gress, May 23 - 28, Budapest, Hungary, Proceedings O-12-04

LENK, K. (1931), "Der Ausgleich des Gebirgsdruckes in grossen Teufen beim Berg- und Tunnelbau", Verlag Julius Springer, Berlin, pp. 21

LUNARDI, P. (1998), "Storia del collegamento ferroviario tra Bologna e Firenze" Gallerie e grandi opere sotterranee, No. 54, pp. 16-39

MATHIEU, E. (2008), "At the mercy of the mountain", Tunnels & Tunnelling, Focus on Europe (October), pp. 21-24

MOHR, F. (1957), "Kraft und Verformung in der Gebirgsmechanik untertage", Tagungsberichte, Deutsche Baugrundtagung Köln, W. Ernst Verlag, pp. 52-65

MORITZ, B. (1999), "Energy absorbing elements for tunnels in squeezing rock - design and experience", Anwenderbeiträge des 9. Österreichischen Abaqus Anwendertreffen, Graz, pp. 1-10

SCHAECHTERLE, K. (1926), "Tunnelbau in quellendem Gebirge", Die Bautechnik, Heft 30, pp. 28-65

THUT, A., STEINER, P., STOLZ, M. (2006), "Tunnelling in Squeezing Rock - Yielding Elements and Face Control", 8th International Con. on Tunnel Constructions and Underground Structures, Ljubljana, Slovenia, pp. 150-157

WIESMANN, E. (1914), "Über die Stabilität von Tunnelmauerwerk unter Berücksichtigung der Erfahrungen beim Bau des Hauenstein-Basistunnels", Schweiz, Bauzeitung 64, pp. 23-47

Fulvio Tonon

A HISTORICAL EXCURSUS ON SEQUENTIAL EXCAVATION, NATM AND ADECO

Rabcewicz maintained that "tunnels should be driven full face whenever possible". ADECO, which stands for "Analysis of Controlled Deformations in tunnels", now allows us to fulfill Rabcewicz's dream in any stress-strain condition. In order to achieve that dream and its consequent control over cost and schedule, however, NATM must be abandoned for the ADECO. The paper traces the history of the sequential excavation, NATM (as first conceived) and ADECO (Analysis of Controlled DEformations) with the aim of shedding light on the unavoidable use of sequential excavation in "soft ground", and of highlighting advances in tunnel design and construction that have occurred in Europe after and as alternates to the NATM. The paper presents the basic concepts in the ADECO approach to design, construction and monitoring of tunnels together with some case histories, including: full face excavation for Cassia tunnel (face area> 230 m^2) in sands and silts under 5 m cover

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below an archeological area in Rome, Italy; Tartaguille tunnel (face area> 140 m²) advanced full face in highly swelling and squeezing ground under 100 m cover where NATM led to catastrophic failure, France; and 80 km of tunnels (face area> 140 m²) advanced full face in highly squeezing/swelling ground under 500 m cover for the high-speed railway line between Bologne and Florence, Italy (turnkey contract).

INTRODUCTION

Several generations of NATM (New Austrian Tunneling Method) consultants have us believe that NATM necessarily uses sequential excavation. Was this the original Rabcewicz's intent? On the other hand, in many countries, such as the United States, sequential excavation is currently used to indicate soft ground tunneling without a tunnel boring machine (Romero, 2002). Many points of view on and definitions of the NATM have been proposed (Kovari, 1994) and reviewed by Karaku and Fowell (2004). Brown (1990) and Romero (2002) suggest to differentiate NATM philosophy:

- The strength of the ground around a tunnel is deliberately mobilized to the maximum extent possible.
- Mobilization of ground strength is achieved by allowing controlled deformation of the ground.
- Initial primary support is installed having load-deformation characteristics appropriate to the ground conditions, and installation is timed with respect to ground deformations.
- Instrumentation is installed to monitor deformations in the initial support system, as well as to form the basis of varying the initial support design and the sequence of excavation.

from NATM construction method:

- The tunnel is sequentially excavated and supported, and the excavation sequences can be varied.
- The initial ground support is provided by shotcrete in combination with fiber or welded-wire fabric reinforcement, steel arches (usually lattice girders), and sometimes ground reinforcement (e.g., soil nails, spiling).
- The permanent support is usually (but not always) a cast in place lining.

This paper traces the history of the sequential excavation, NATM (as first conceived) and ADECO (Analysis of Controlled DEformations) with the aim of shedding light on the *unavoidable* use of sequential excavation in "soft ground", and of highlighting advances in tunnel design and construction that have occurred in Europe after and as alternates to the NATM.

SEQUENTIAL EXCAVATION: A 200 YEAR OLD APPROACH

In his 1963 book entitled "The History of Tunneling", G.E. Sandström talks about the tunneling methods devised when the canal era and the railroad era developed in the first half of the 1800s: yes, this is 200 years ago!. Since the book was published in 1963 and Rabcewicz's papers on NATM were published in late 1964 and early 1965, there is little doubt that what Sandström describes are methods that preceded the NATM. Let's here from Sandström (pages 113 and ff):

1. *BELGIAN SYSTEM USED IN THE 1800s*
FROM SANDSTRÖM (1963)

2. *BRITISH SYSTEM USED IN THE 1800s*
FROM SANDSTRÖM (1963)

3. *CRISTINA (ITALIAN) SYSTEM USED IN THE 1800s*
FROM SANDSTRÖM (1963)

4. *GERMAN SYSTEM USED IN THE 1800s*
FROM SANDSTRÖM (1963)

1

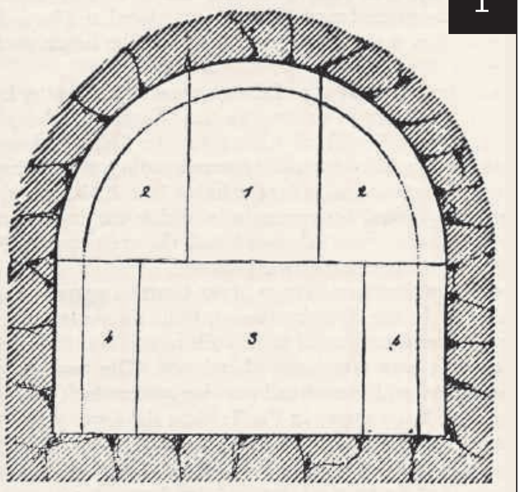


Fig 48. An early sketch indicating the sequence of excavation according to the Belgian System.

2

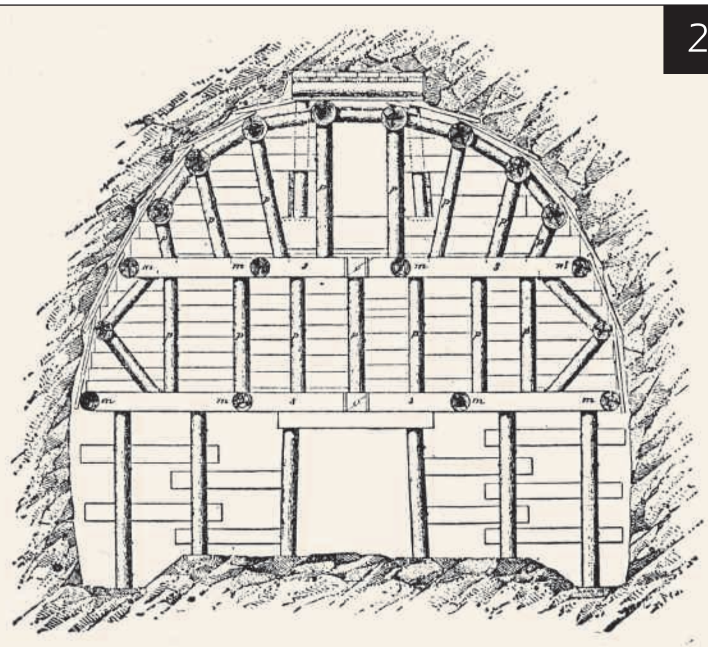


Fig 33. The first Swiss tunnel was the 8,198-ft.-long Hauenstein Tunnel on the central railway between Bâle and Olten. This 25·6×19·7 ft. tunnel was built 1853–8 by the famous British railway contractor Thomas Brassey.

3

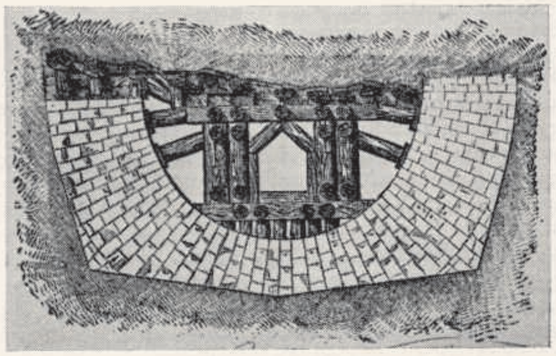


Fig 59. The fully developed Cristina System of tunnelling. The excavated section was filled with ashlar, beginning with the invert, as soon as the clay was removed.

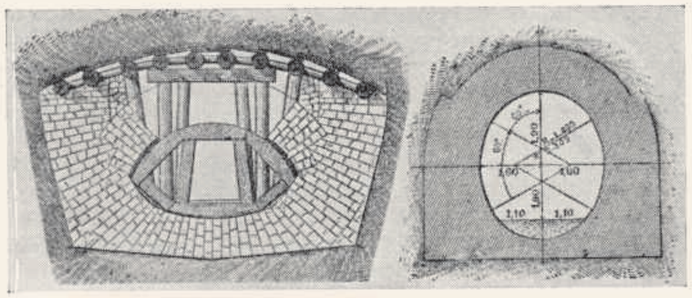


Fig 60. By filling in the excavated space with stone, leaving only a small pilot tunnel open in the centre, the Italian tunnellers finally succeeded in stabilizing the ground (left). The finished Cristina Tunnel is merely a small opening enclosed by a tremendous stone structure. But even this structure did not prove wholly stable and it became necessary to trim the sides to enable a train to squeeze through the tunnel.

4

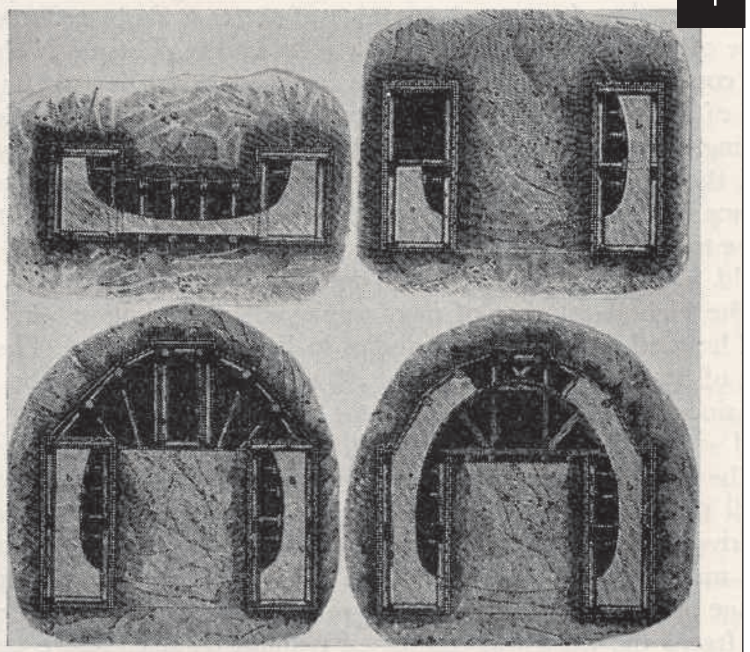


Fig 31. The Köningsdorf Tunnel on the Dresden–Leipzig line was driven in 1837. In this tunnel the foundation for the lining was placed first, after which the masonry lining was put in. With the lining in place the central core was removed.

"An old-time mining tunnel, or drift, seldom exceeded an area of 10 x 10 ft, whereas a single-track railway tunnel used to be given an area of 16 x 22 ft., and a double track 28 x 22 ft. (modern tunnels are larger). The conventional practice used to be to advance a small pilot heading first in the forepoling manner described – if in heavy ground – and subsequently expand it to full size in some other way.
The method of breaking out from a safe, wholly enclosed pilot tunnel is one of the central problems in tunneling and was endlessly debated throughout the last century. As a matter of fact it is still an issue that has to be argued as a preliminary to any tunneling scheme, because if it is not correctly settled beforehand men will lose their lives and the contractor his capital.
During the last century, a number of different tunneling systems were evolved which derived their names from their national origin. These were the English system, the Belgian System, the Austrian system, German (actually French) system, and the Italian so-called Cristina system. The Americans also laid claim to an independent system".
And on page 130: "*…, the interesting feature of these early American railway tunnels is that most of them were driven full face, i.e. the entire tunnel area was excavated, although in poor ground the top half was taken out to the full width and the roof secured with rafter timbering and lagged*".

The methods are illustrated in Figure 1 through Figure 4, and the reader is referred to Sandström's book for excellent details.

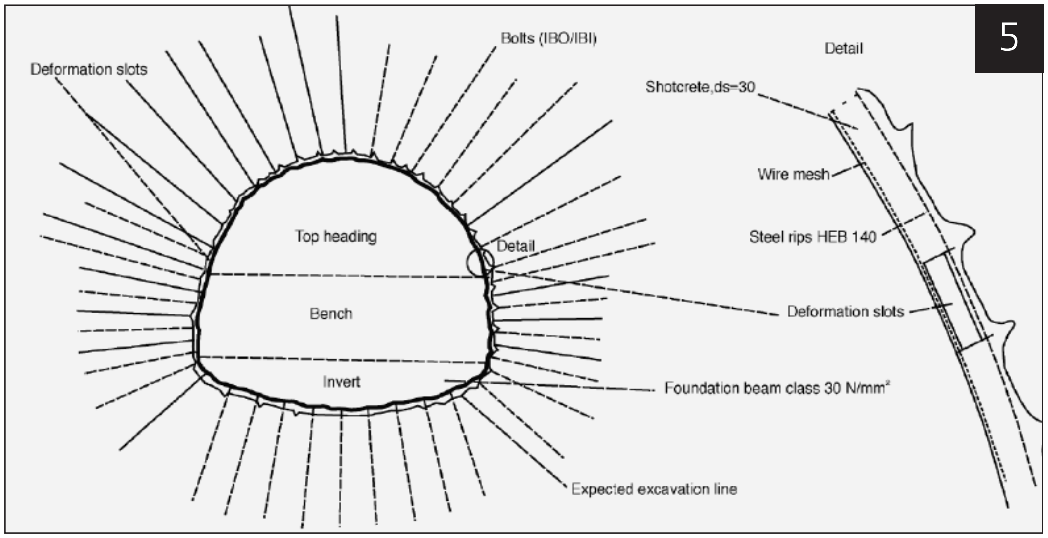
Take home:
The "sequential excavation method" is 200 years old and was well known when the NATM was coined in 1964.
- The "sequential excavation method" was developed 200 years ago by miners that had to adapt their mining techniques to the needs of civil engineering works.
- Power is defined as work/time, i.e. (ability to do work)/time.
- When the "sequential excavation method" was devised, tunnels were driven without electricity and compressed air, i.e. the available power was very small, mainly manpower.
- Breaking out from the pilot tunnel is one of the central problems in tunneling; if it is not correctly settled beforehand men will lose their lives and the contractor his capital.
- Early American tunneling was full face.

… AND RABCEWICZ SAID "*TUNNELS SHOULD BE DRIVEN FULL FACE WHENEVER POSSIBLE*"

In his abstract to the first 1964 paper on NATM, Rabcewicz refers to the NATM as: "*a new method consisting of a thin sprayed concrete lining, closed at the earliest possible moment by an invert to a complete ring - called "an auxiliary arch" - the deformation of which is measured as a function of time until equilibrium is obtained*". In the same paper, on page 454, Rabcewicz states that "*One of the most important advantages of steel supports is that they allow tunnels to be driven full face to very large cross sections. The resulting unrestricted working area enables powerful drilling and mucking equipment to be used, increasing the rate of advance and reducing costs. Nowadays, dividing* *the face into headings which are subsequently widened is used only under unfavourable geological conditions*." On page 457, Rabcewicz continues on this topic: "*There are still some difficulties to be overcome in normal methods of construction, as inverts are still usually built last of all, leaving the roof and sidewalls of the lining to deform at will. In the meantime, experience has taught us that it is by far more advantageous from all points of view, and frequently even imperative, to close a lining to a complete ring at a short distance behind the face as soon as possible. To comply with this requirement, tunnels should be driven full face whenever possible, although this cannot always be done, particularly in bad ground, where it often becomes necessary to resort to heading and benching. In the most difficult cases it may even be necessary to drive a pilot heading before opening it out to full section. An auxiliary arch executed in the upper heading (Belgian roof arch) though fairly effectively preventing roof loosening, represents an intermediate construction stage, which is still subject to lateral deformation. Such instability has to be removed as soon as possible by excavating the bench and closing the lining by an invert*."

Take home:
- NATM has nothing to do with sequential excavation.
- Rabcewicz realized that tunnels should be driven full face.
- Rabcewicz realized that full face allows for the use of large equipment i.e. deployment of large power at the face, which translates into fast tunnel advance and reduced costs.
- Rabcewicz never cared about nor mentioned the ground ahead of the tunnel face or ground support/reinforcement ahead of the tunnel face.
- Rabcewicz wanted but could not find a way to advance full face in difficult stress-strain conditions. His inability to proceed full face in all stress-strain conditions in 1964 was caused by a technological limitation in the normal methods of construction of those days

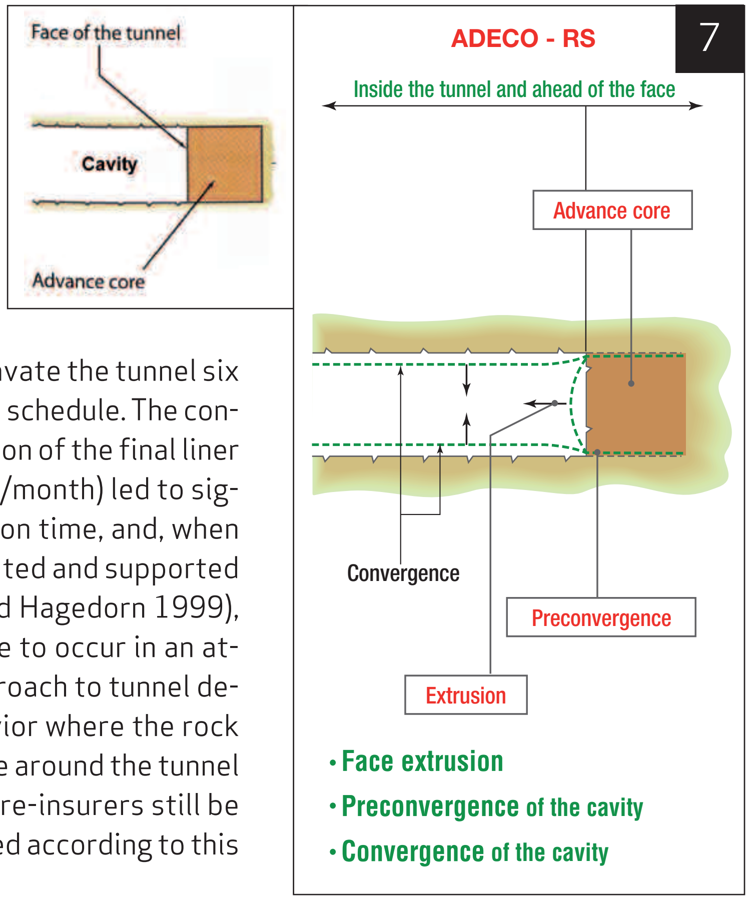
Some papers in this book propose examples where flexible solutions have been successfully used. However, the very well documented example of the Bolu tunnel (Brox and



5. *MINIMUM SUPPORT PATTERN FOR FLEXIBLE APPROACH IN THE BOLU TUNNEL*
ORIGINAL DESIGN BY GEOCONSULT (1996)
AFTER DALGIÇ (2002)

6. *BOLU TUNNEL, HEAVE IN THE ELMALIK RIGHT TUBE AT KM 54 + 135*
CONSTRUCTED PER THE ORIGINAL DESIGN BY GEOCONSULT (1996)
AFTER DALGIÇ (2002)

Hagedorn 1999, Dalgiç 2002) should serve as an example in which ignoring the ground ahead of the tunnel face according to the NATM has led to the use of sequential excavation. Re-start of the excavation for the bench uncontrollably increased the displacements that far exceeded the design tolerances. Flexible linings/supports, overexcavation, longitudinal gaps in the shotcrete lining and yielding rock bolting according to the NATM (Figure 5) forced the contractor to increase the excavation cross-section from 140 m^2 to 220 m^2, and to re-excavate the tunnel six times with dramatic effects on tunnel construction cost and schedule. The concept of monitoring the displacements to delay the installation of the final liner when convergences stop (or reach a small value, e.g., 2 mm/month) led to significant deformations (Figure 6), unpredictable construction time, and, when an earthquake stroke, to failure of 400 m of already excavated and supported tunnel. As stated in the report to the re-insurers (Brox and Hagedorn 1999), the concept of allowing large (50 cm or more) convergence to occur in an attempt to reduce the "rock load" represents a high-risk approach to tunnel design because it leads to unpredictable stress-strain behavior where the rock mass is disturbed by large displacements and the yield zone around the tunnel may reach the surface (60-80 m in this specific case). Will re-insurers still be willing to issue insurance policies to tunnel projects designed according to this type of high-risk approach?.

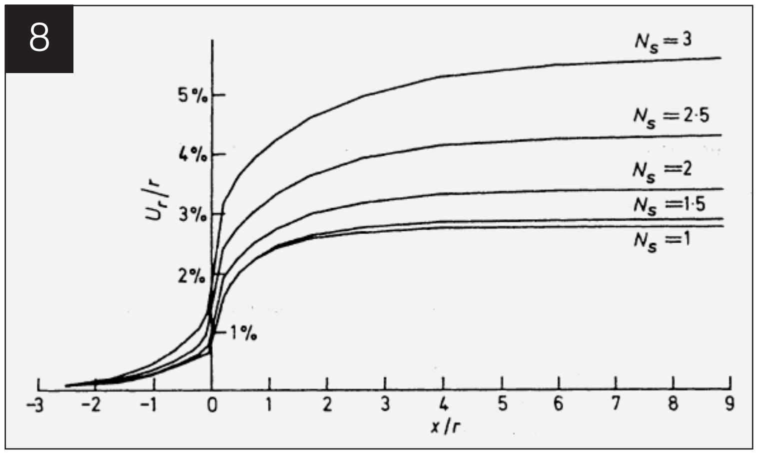


7. NOMENCLATURE
AFTER LUNARDI (2008)

QUANTIFICATION OF PRE-CONVERGENCE

Let us establish the nomenclature illustrated in Figure 7, where cavity is the opening already excavated, and advance core is the ground ahead of the tunnel face and comprised within the future tunnel profile.

In 1982, Panet and Guenot (1982) quantified the radial displacement of the ground at the future tunnel profile that occurs ahead of the tunnel face (preconvergence) in an unlined tunnel (Figure 8) excavated in an elastic or elasto-plastic ground (no time-dependent behavior was considered). At the face, about 30% of the final convergence has already occurred. Other researchers have quantified the preconvergence and convergence with and without the effect of the installed lining (e.g., Corbetta et al. (1991); Bernaud e Rousset (1992), (1996); Nguyen-Minh (1994); Nguyen-Minh et al. (1995); Nguyen-Minh and Guo (1993.a; 1993.b; 1996) and Guo (1995)). In particular, these studies show that a stiff lining may significantly reduce the convergence at the face, and thus preconvergence.



8. PRECONVERGENCE AND CONVERGENCE VS. DISTANCE TO THE TUNNEL FACE FOR TUNNELS IN CLAYS, UNDRAINED CONDITIONS
$N_S = P_0/S_U$; P_0 = IN SITU HYDROSTATIC STRESS, S_U = UNDRAINED SHEAR STRENGTH
AFTER PANET AND GUENOT (1982)

ITALIAN ADVANCES IN PRE-SUPPORT

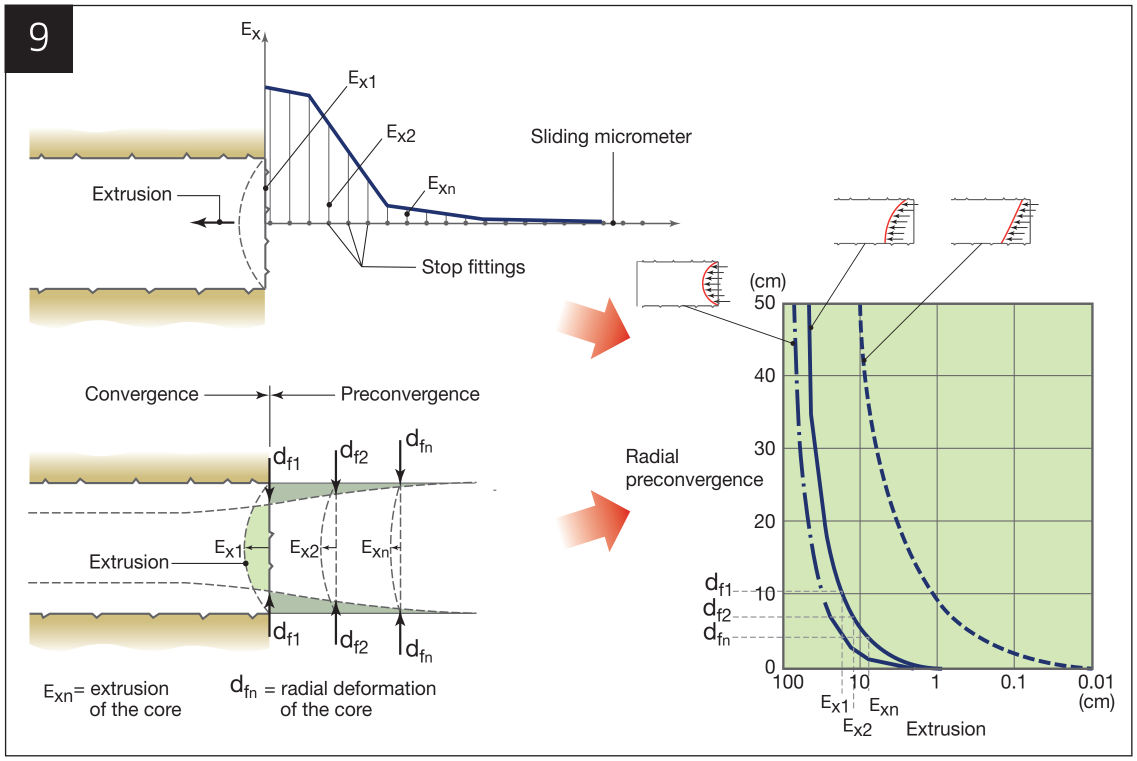
The micropile umbrella-arch (also known as pipe-arch umbrella) consists of sub-horizontal micropiles made up of steel pipes grouted in place at high pressure to improve the ground all around the perimeter of the excavation. In 1975, micropiles at different angles were used to tunnel through a collapsed zone (Carrieri et al. 2002), and in 1976 the first umbrella was designed as integral part of the support system for the S. Bernardino tunnel along the Genova-Ventimiglia railway line (Piepoli, 1976). By 1982, 15 tunnels in Italy had been driven by using a micropile umbrella (Barisone et al., 1982). Unfortunately, in many countries a pipe-arch umbrella is erroneously thought of being part of the NATM. In Italy, other major technological advances were made in the 1980's as a consequence of Lunardi's basic observations on and improved understanding of tunneling. Let's see what they were.

LUNARDI'S BASIC OBSERVATIONS ON TUNNEL BEHAVIOR

The same way as Rabcewicz conceived of the NATM in the 1960's by observing tunnel behavior, in the 1970-80's Lunardi made the following basic observations in the tunnels that he designed and/or built:

1. Convergence (radial displacement of cavity wall, Figure 7) is only the last manifestation of ground deformation. The convergence is always preceded by and is the effect of the deformation of the advance core: preconvergence = radial displacement of ground at the future tunnel perimeter, and extrusion = horizontal displacement of the core.

2. Extrusion can be measured in situ and is related one-to-one with the preconvergence (Figure 9)



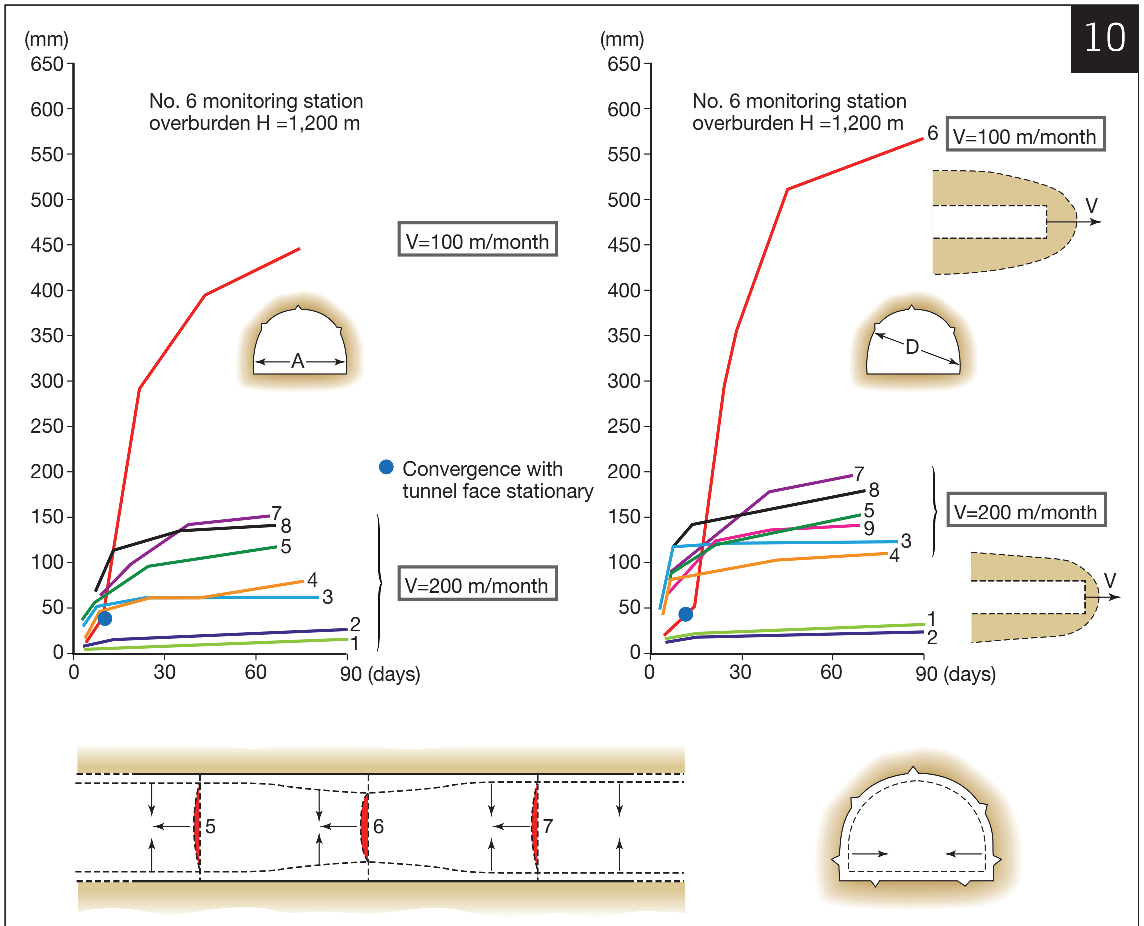
9. MEASUREMENT OF EXTRUSION WITH SLIDING MICROMETER AND RELATIONSHIP BETWEEN EXTRUSION AND PRECONVERGENCE

AFTER LUNARDI (2008)

3. In squeezing ground, everything else being the same, the deformation (convergence) of the cavity increases as the speed of tunnel advance decreases. This is illustrated in Figure 10, which gives the convergence measured in the calcshists of the Frejus tunnel. When the tunnel advanced 100 m/month (Section 6), the convergence in the cavity was three times as large as the convergence measured when the tunnel advanced 200 m/month. When advancing 100 m/month, it was observed that the ground in the tunnel core deformed much more then when advancing 200 m/month.
4. The collapse of the cavity is always preceded by the collapse of the face-core system (Figure 11).
5. In top-heading and benching, the tunnel face starts at the crown of the top heading and ends at the invert of the bench (Figure 12).
6. The arrival of the tunnel face reduces the confinement in the core and increases the major principal stress, giving rise to three basic face-core behaviors: A = stable; B = stable in the short term; C = unstable (Figure 13).

10. *CONVERGENCE MEASUREMENTS IN THE FREJUS HIGHWAY TUNNEL, 1970S*
AFTER LUNARDI (2008)

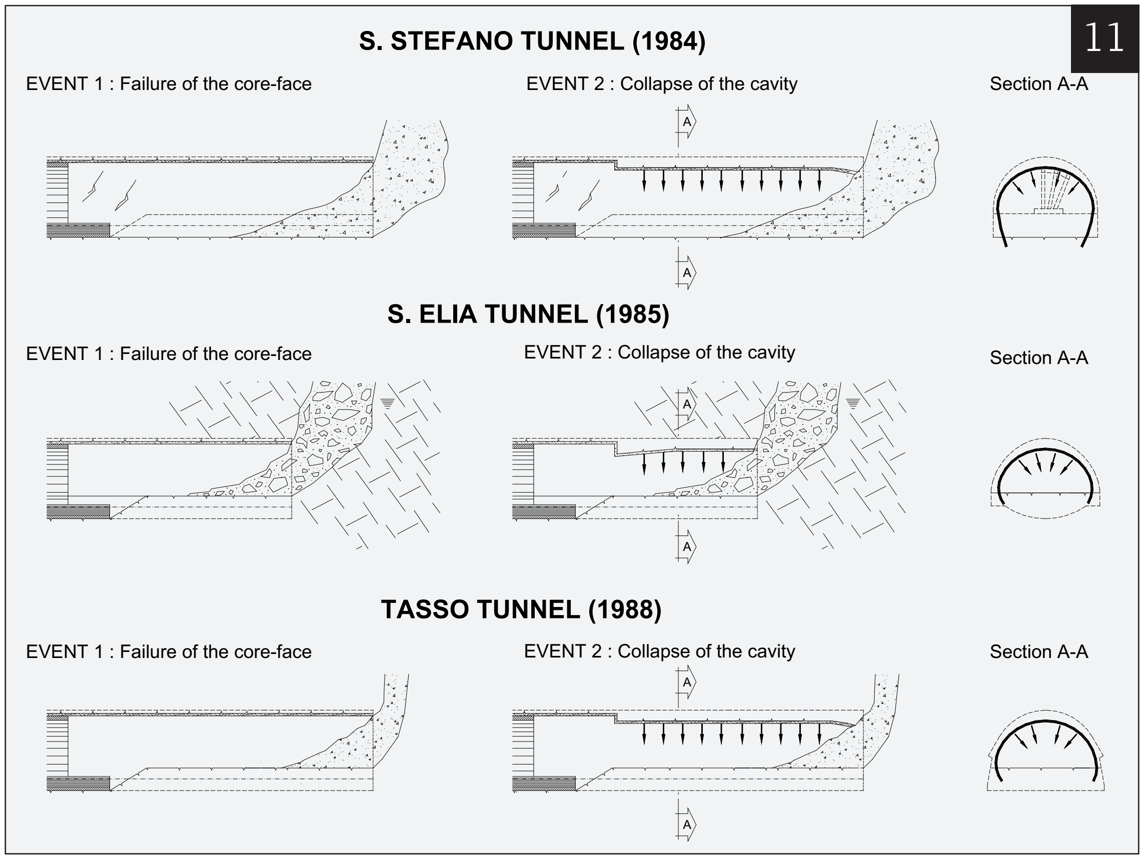
10

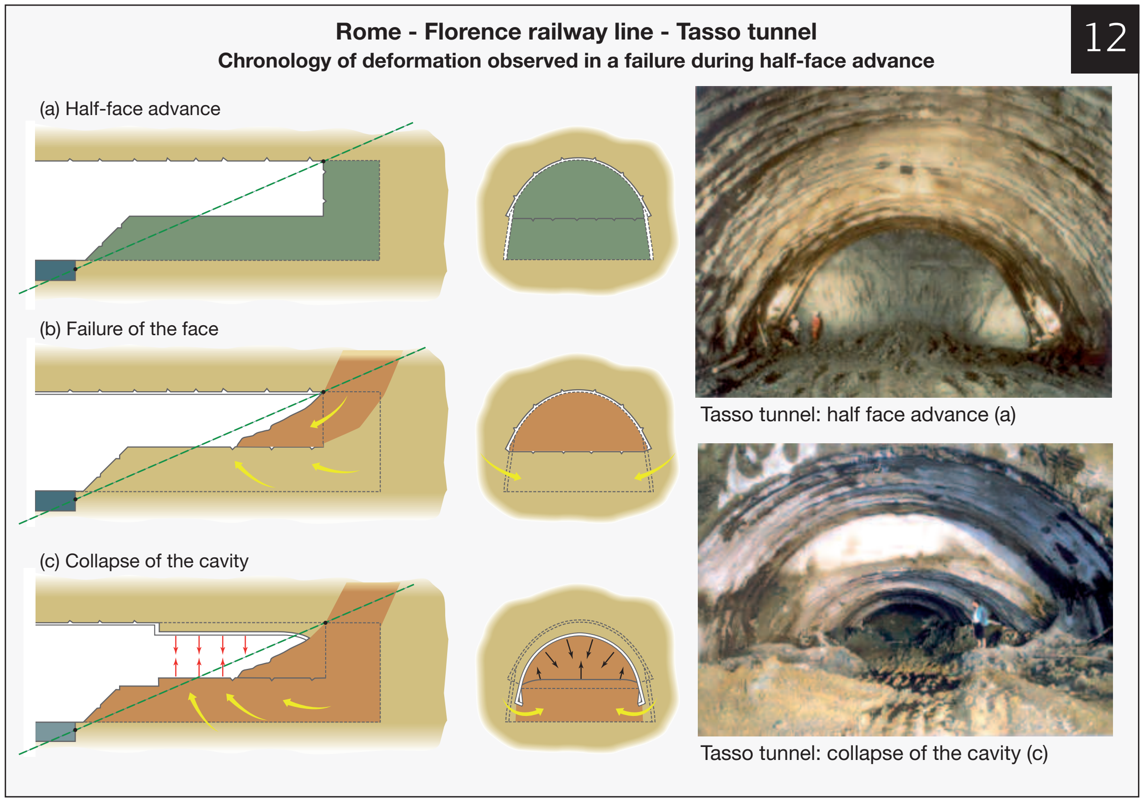


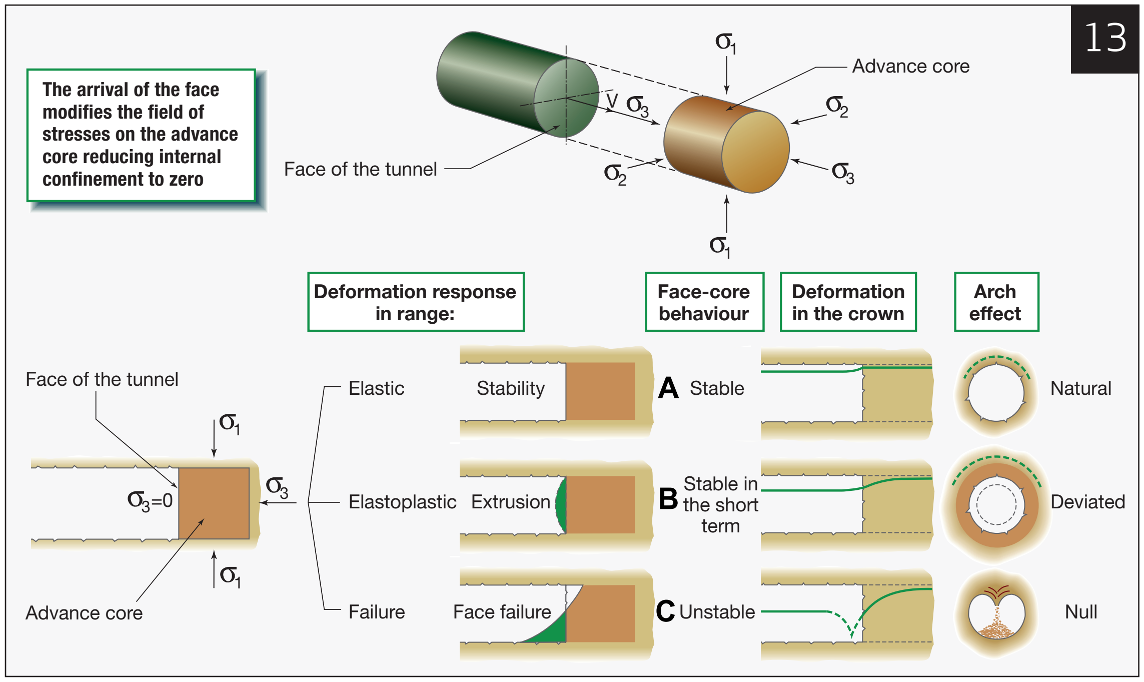
Monitoring stations	1	2	3	4	5	6	7	8	9
Chainage (m)	845	861	2772	3954	4507	5172	5533	5915	6066
Overburden (m)	490	500	580	590	740	1200	1400	1530	1640

11. *CASE HISTORIES OF TUNNEL COLLAPSES*
AFTER LUNARDI (2008)

12. *FAILURE AT TASSO TUNNEL EXCAVATED TOP HEADING AND BENCHING, 1988. NOTICE 2 M CONVERGENCE IN TOP HEADING*
AFTER LUNARDI (2008)







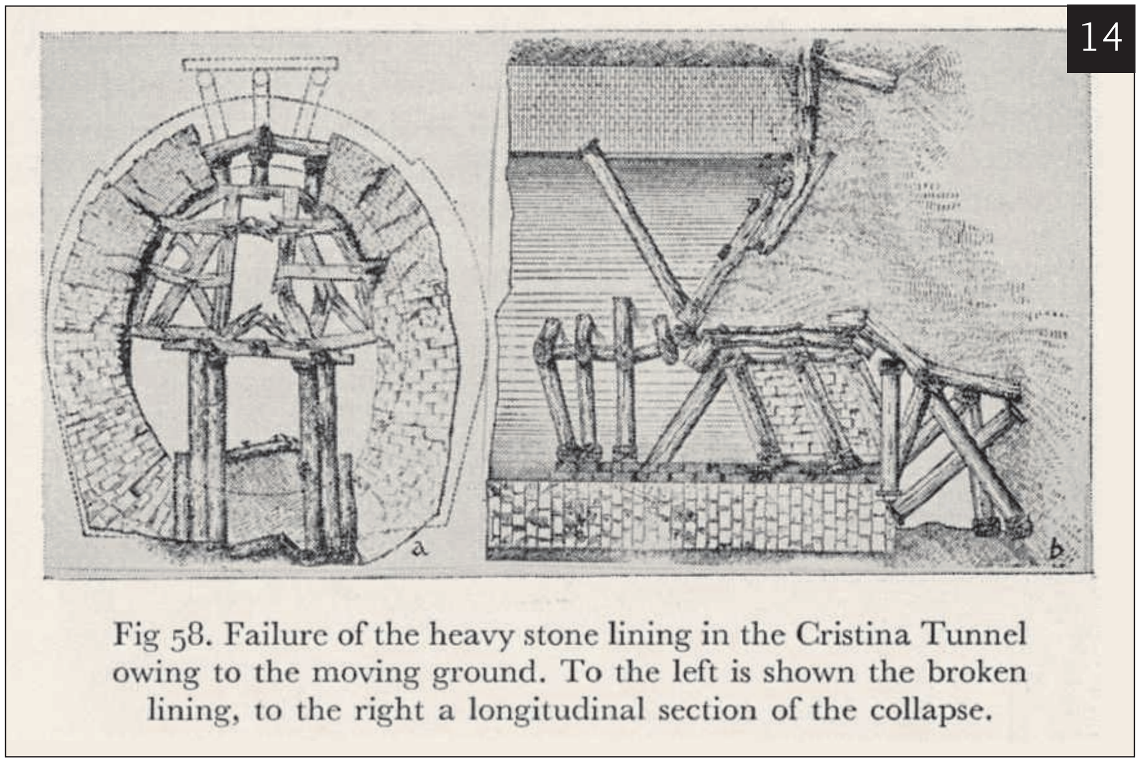
13. *TUNNEL BEHAVIOR CATEGORIES BASED ON FACE-CORE BEHAVIOR*

AFTER LUNARDI (2008)

14. *CRISTINA TUNNEL, 1871: THE FAILURE OF THE CAVITY IS PRECEDED BY THE FAILURE OF THE CORE-FACE*

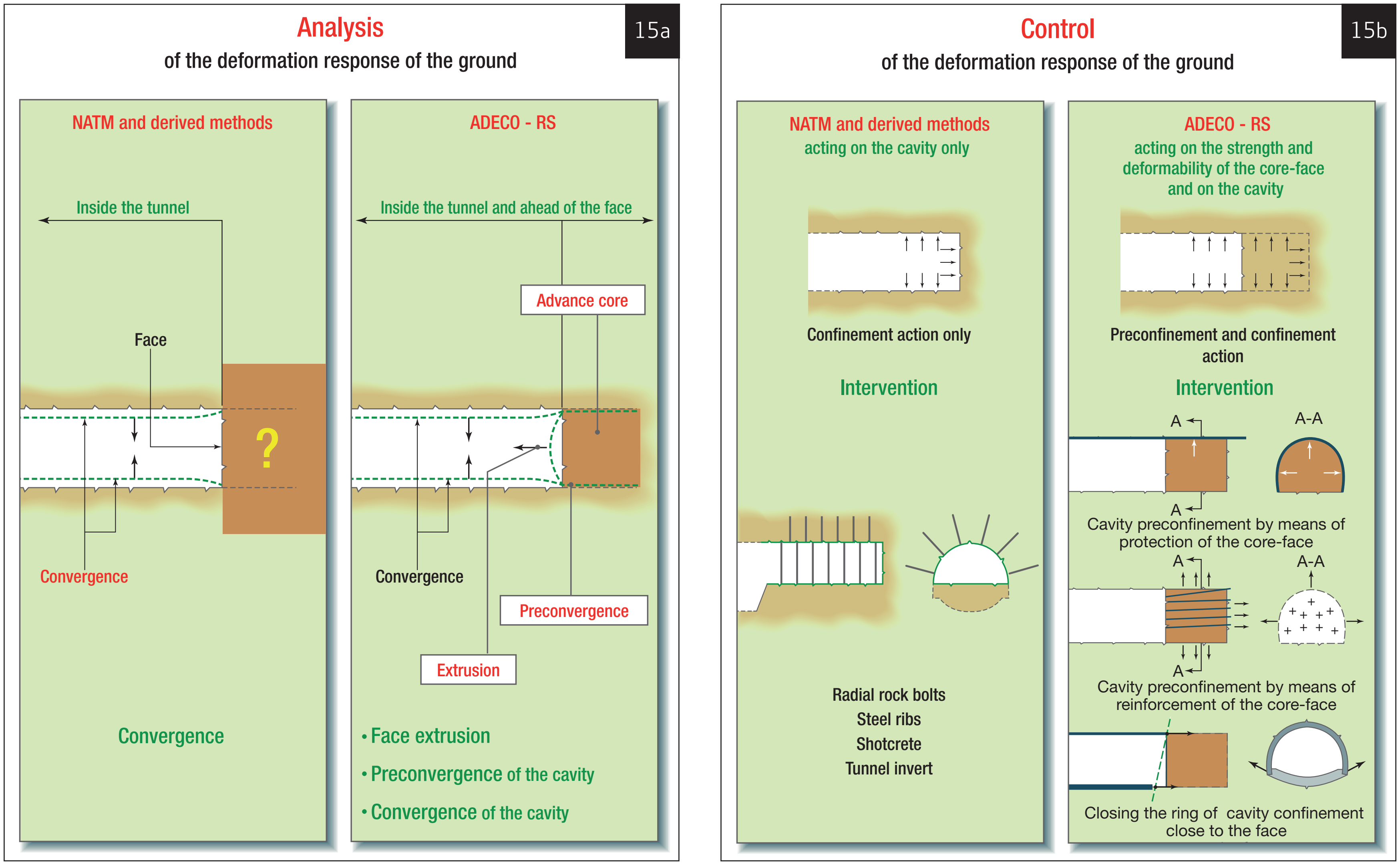
FROM SANDSTRÖM (1963)

The fact that the collapse of the cavity is always preceded by the collapse of the face-core system is not totally new. As shown in Figure 14, it was known back in 1871 in the collapse of the Cristina Tunnel then re-excavated as shown in Figure 3. However, that lesson was completely forgotten until Lunardi re-discovered it in the 1970-80's.



Take home (Figure 15):

- The ground behavior around the cavity and the convergence in the cavity at a given tunnel chainage X are controlled by the deformation and the behavior of the ground in the tunnel core when excavating the tunnel at chainage X (what Rabcewicz did not understand and could not do in 1960s).
- In difficult stress-strain conditions, counteracting convergence is not feasible. One needs to control preconvergence and extrusion, i.e. the deformations in the core ahead of the tunnel face (what Rabcewicz did not understand and could not do in 1960s).
- Sequential excavation extends the tunnel face even if the top heading is lined (same as Rabcewicz "*An auxiliary arch executed in the upper heading ... represents an intermediate construction stage, which is still subject to lateral deformation*") and increases the volume of ground in the core that, by deforming, controls the behavior of the cavity (what Rabcewicz did not understand).
- If the extent of the face and of the core must be minimized, one has to proceed full face (same as Rabcewicz "*tunnels should be driven full face whenever possible*").



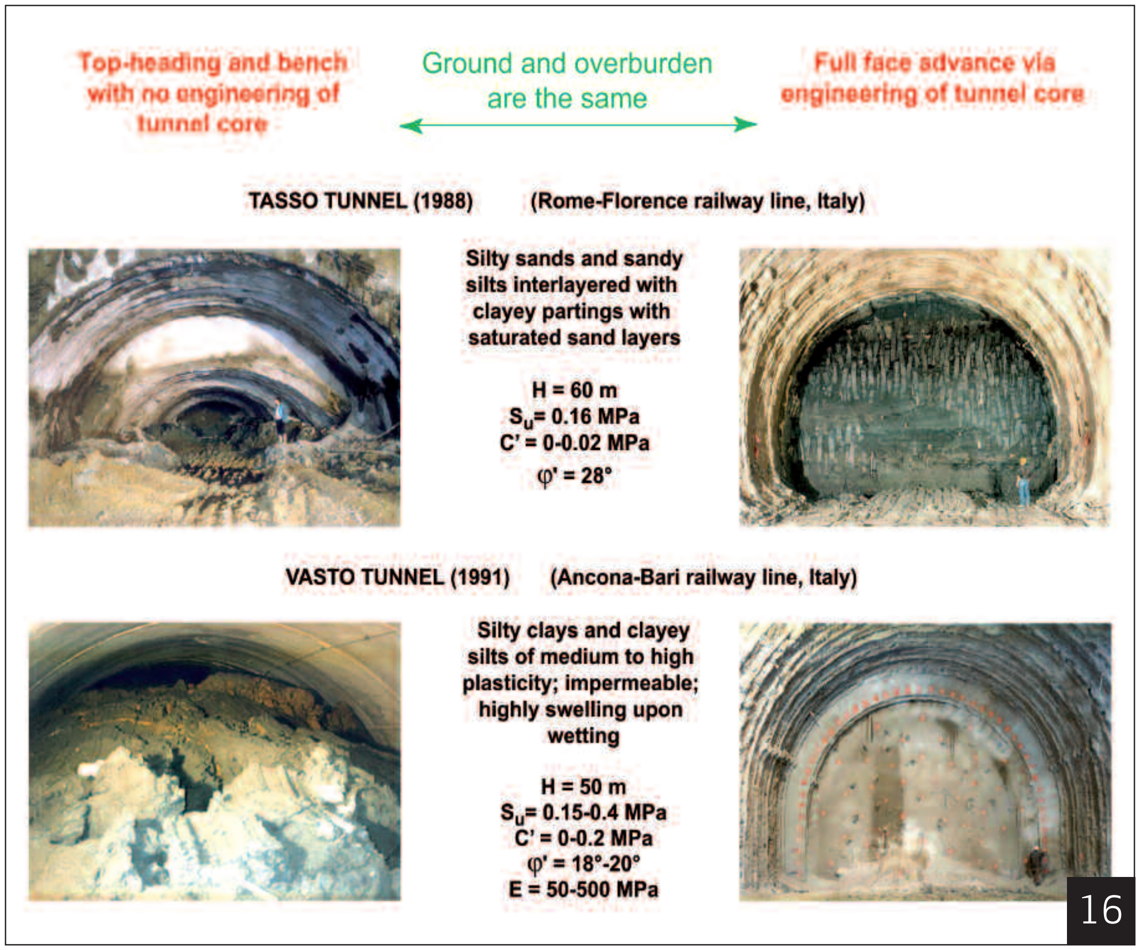
15. *NATM VS. ADECO*
AFTER LUNARDI (2008)

These results led Lunardi to the idea of engineering the core in order to use the core as a stabilization method for the cavity, the same way as rockbolts, shotcrete and steel sets are used to stabilize the cavity.
One of the most striking proofs of the central role of the core is given by the re-excavation of tunnels that failed when the core was ignored: Figure 16 offers two of many examples. The idea of engineering the core was implemented by developing new technologies, such as:

- Sub-horizontal jet-grouting (Campiolo tunnel, 1983).
- Pre-cut with full face excavation (Sibari-Cosenza railway line, 1985, evolution of the pre-decoupage used in the top heading in the Lille Metro, France).
- Fiberglass reinforcement of the core as a construction technology to be used systematically in full-face tunnel advance (1985, high speed railway line between Florence and Rome), and not only as an ad-hoc means to overcome unpredicted tunneling problems.

The ADECO is the culmination of these observations, experiments, and new technologies. The new technologies introduced with the ADECO can thus only be understood and properly used within the context of the ADECO approach.

16. *TUNNELS FAILED WHEN THE CORE WAS NOT USED AS A STABILIZATION METHOD (LEFT-HAND SIDES OF FIGURE 15 A AND B); AND RE-EXCAVATED BY USING THE CORE AS A STABILIZATION MEASURE (RIGHT-HAND SIDES OF FIGURE 15 A AND B)*



ADECO APPROACH

The ADECO (Analysis of COntrolled DEformations) workflow is illustrated in Figure 17. In the Diagnosis Phase, the unlined/unreinforced tunnel is modeled in its *in situ* state of stress with the aim of subdividing the entire alignment into the three face/core behavior categories: A, B, and C: these depend on the stress-strain behavior of the core (ground strength, deformability and permeability + *in situ* stress), not only on the ground class. The site investigation must be detailed and informative enough to carry out such quantitative analyses: this clearly defines what the investigation should produce.

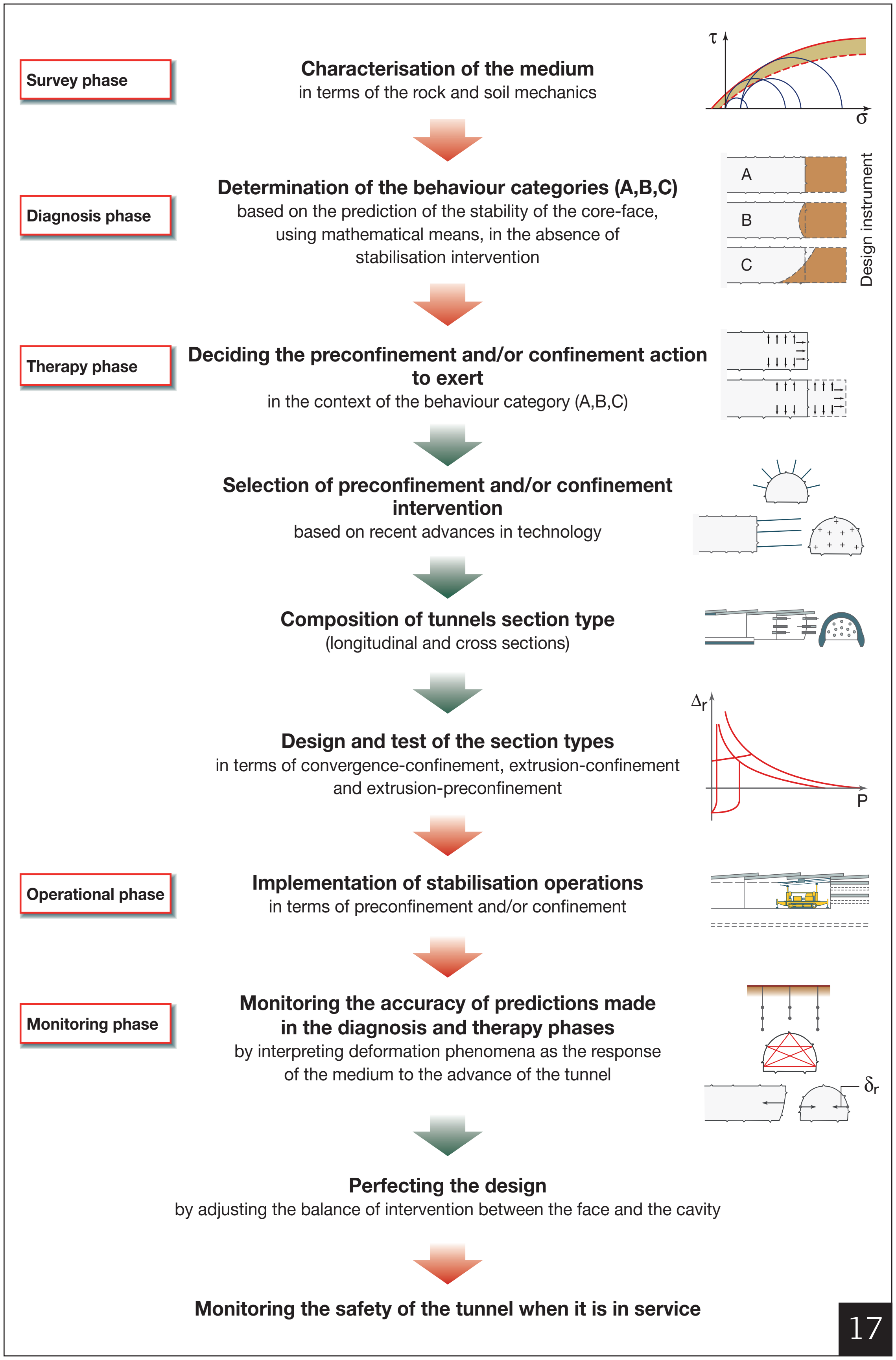
In the Therapy phase, the ground is engineered to control the deformations found in the Diagnosis Phase. For tunnel category A, the ground remains in an elastic condition, and one needs to worry about rock block stability (face and cavity) and rock bursts; typically, rock bolts, shotcrete, steel sets and forepoling are used to this effect. In categories B and C yielding occurs in the ground; an arch effect must be artificially created *ahead* of the tunnel face (pre-confinement) when a large yielded zone forms in category B, and in all cases in category C. By looking at the Mohr plane (Figure 18) two courses of action clearly arise:

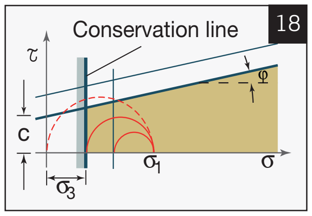
- Protecting the core by reducing the size of the Mohr circle: this can be achieved either by providing confinement (increasing σ_3) or by reducing the maximum principal stress (reducing σ_1).
- Reinforcing the core, thereby pushing up and tilting upwards the failure envelope.

The rightmost column in Figure 15 depicts the actual implementation of these two ideas as pre-confinement actions. The third line of action consists of controlling the convergence at the face by using the stiffness of the lining (preliminary or even final, if needed), which may also longitudinally confine the core. It is only in this context that the different technologies currently available and listed in Figure 19 take their appropriate role. Notice that, at difference with the NATM, the ADECO embraces tunnels excavated with and without a tunnel boring machine.

Once the confinement and pre-confinement measures have been chosen, the cross-section is composed both in the transverse and longitudinal directions, and then analyzed. In all cases, full face advance is specified in all stress-strain conditions, thus fulfilling Rabcewicz's dream.

For each cross-section, displacement ranges are predicted in terms of convergence and extrusion (Figure 20). Besides plans and specs, construction guidelines are also produced during the design stage. The construction guidelines are used at the construction site to make prompt decisions based on the displacement readings. If the readings are in the middle of the predicted ranges, then the nominal cross-section in the plans and specs is adopted; if reading values fall to the lower end of the predicted displacement ranges, then the minimum quantities specified in the guidelines are adopted for the stabilization measures (Figure 20). Likewise, if reading values are on the upper end of the predicted displacement ranges, then the maximum quantities specified in the guidelines are adopted.



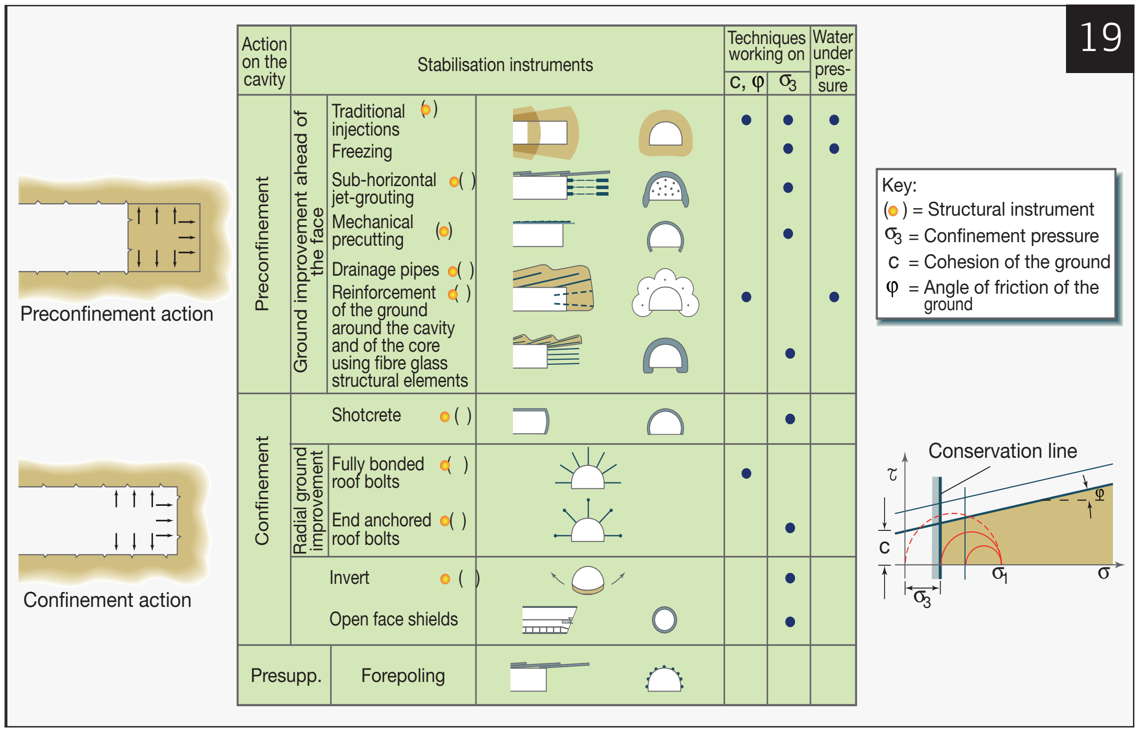


17. *ADECO WORKFLOW*
AFTER LUNARDI (2008)

18. *MOHR-PLANE EXPLANATION OF APPROACHES TO STABILIZE/STIFFEN THE CORE*
AFTER LUNARDI (2008)

19. *SUBDIVISION OF STABILIZATION TOOLS BASED ON THEIR ACTION AS PRE-CONFINEMENT OR CONFINEMENT*
AFTER LUNARDI (2008)

20. *DISPLACEMENT PREDICTIONS AND DESIGN GUIDELINES*
AFTER LUNARDI ET AL. (2008)



Section Types	Geology	Convergence (cm)	Extrusion (cm)
A	Monte Modino Sandstones	2-3	Negligible
B0		3-5	Negligible
B0V		5-10	< 3
B2	Scaly Clays	8-12	< 6
B2V		6-10	< 5
C2		10-14	< 10
C6		8-12	< 8

Section Types	Intervention	Variabilities		
		Minimum	*Nominal*	*Maximum*
C2	Steel rib step	1.2 m	1.0 m	0.8 m
	No. FTG face	50	70	90
	FTG face overl.	10.0 m	12.0 m	14.0 m
	Excavation	14.0 m	12.0 m	10.0 m
	Invert-face (°)	< 2.0∅	< 1.5∅	< 0.5∅
	Crown-face	< 3.0∅	< 5.0∅	< 7.0∅

20

Finally, if the readings are outside the predicted displacement ranges, the guidelines specify the new section to be adopted. In this way, ADECO clearly distinguishes between design and construction stages because no improvisation (design-as-you-go) is adopted during construction.

Monitoring plays a major role in the ADECO, but with two main differences with respect to the NATM:
- In categories B and C, not only convergence but also extrusion is measured because the cause of instability is the deformation of the core, and because stability of the core by pre-confinement actions is a necessary condition for the stability of the cavity.
- Monitoring is used to fine tune the design, not to improvise cavity stabilization measures, so that construction time and cost can be reliably predicted.

Tunnels are thus paid for how much they deform, which, unlike rock mass classifications carried out at the face, is an objective measure void of any interpretation. In addition, rock mass classifications are inapplicable to soils and complex rock mass conditions not included in classifications' databases.
Experience in over 500 km of tunnels indicates that, when the ADECO has been adopted and tunnels were paid for how much they deformed, claims have decreased to a minimum.

ADVANTAGES OF THE ADECO APPROACH OVER SEQUENTIAL EXCAVATION AND NATM

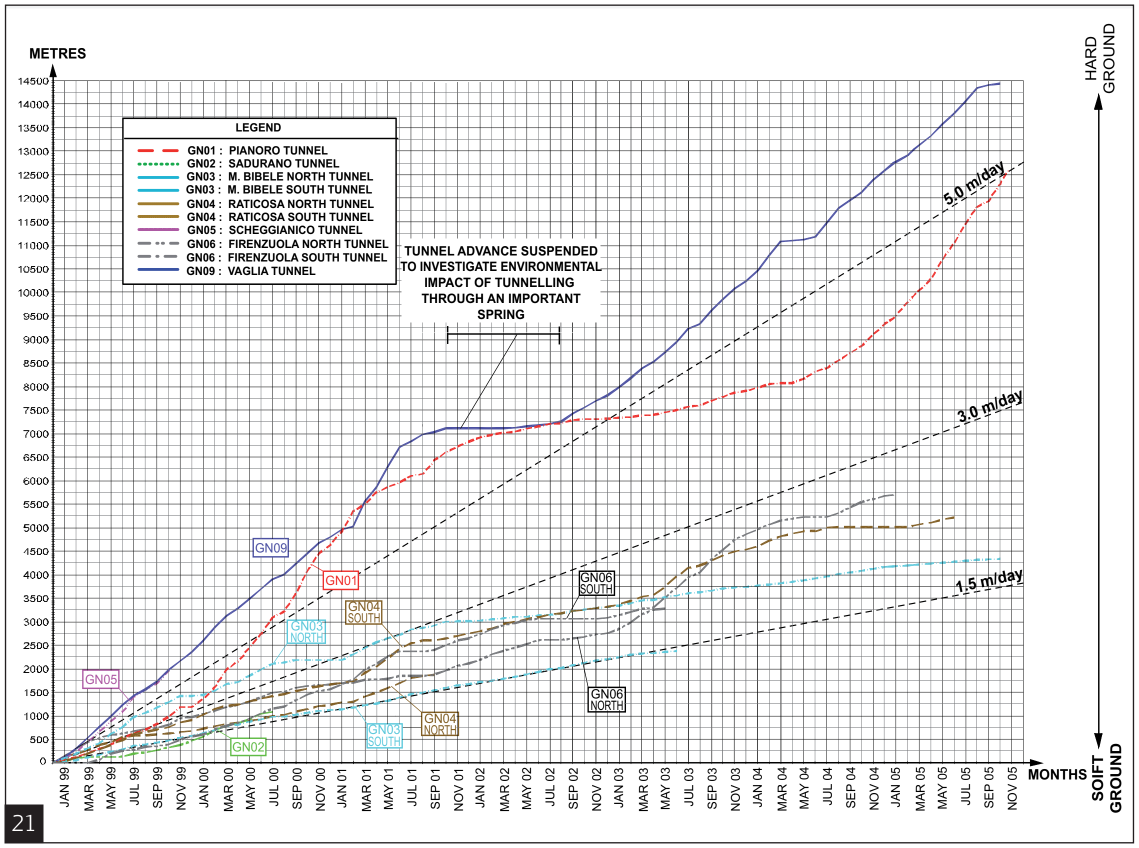
▸ ADECO fulfills Rabcewicz's dream of advancing full face in all stress-strain conditions, which allows risk, cost and construction time to be minimized.

▸ Tunnel construction is finally industrialized in all tunneling conditions because tunneling advance is no longer subject to the ground but the ground is made what it needs to be in order to proceed as fast as possible. This is illustrated in Figure 21 and Figure 24, where production rates are constant even in the most difficult stress-strain conditions (highly squeezing, and squeezing and swelling, respectively). In Figure 24, compare ADECO advance rates with sequential excavation rates, which are overall much smaller and are not constant.

▸ Industrialization entails that cost and time can be reliably predicted at the design stage. Figure 22 shows how predicted production rates were maintained during construction of 85 km of tunnels even under the most difficult stress-strain conditions (highly squeezing). Notice that these advance rates refer to the finished 140 m^2 face tunnel (including final lining), not top heading, or pilot drift. As stated in the introduction, NATM philosophy entails designing the cavity support/reinforcement based on monitoring results, which means that construction time and cost cannot be predicted.

▸ Constant production minimizes ground deformation, which minimizes squeezing and thus the loading on the final lining, which becomes cheaper.

21. PRODUCTION DATA IN THE BOLOGNE-FLORENCE HIGH-SPEED RAILWAY TUNNELS
AFTER LUNARDI (2008)

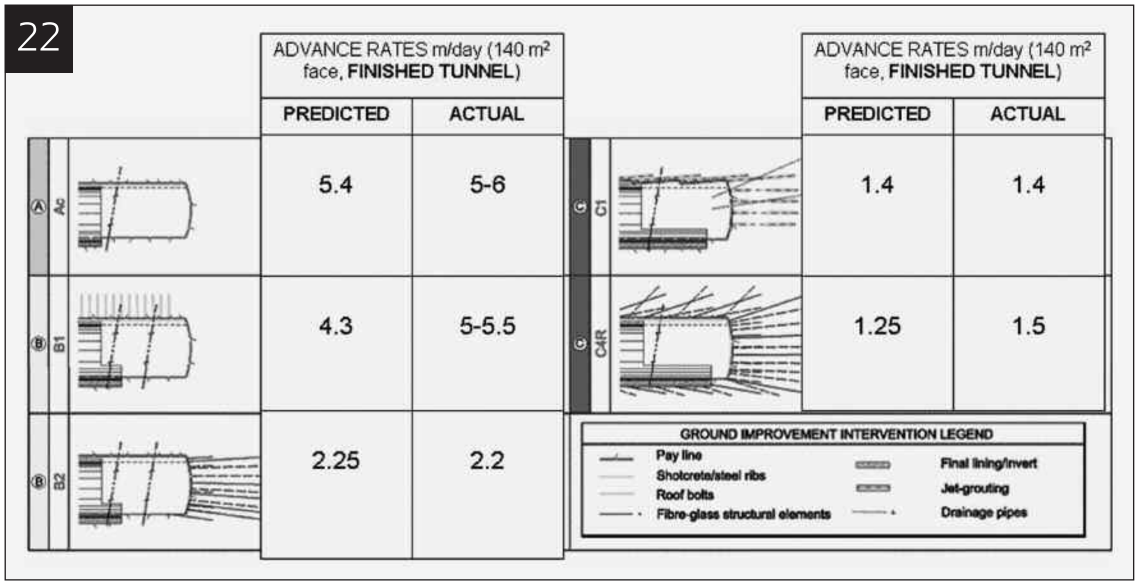


22. *PREDICTED VS. ACTUAL PRODUCTION RATES IN THE BOLOGNE-FLORENCE HIGH-SPEED RAILWAY TUNNELS*

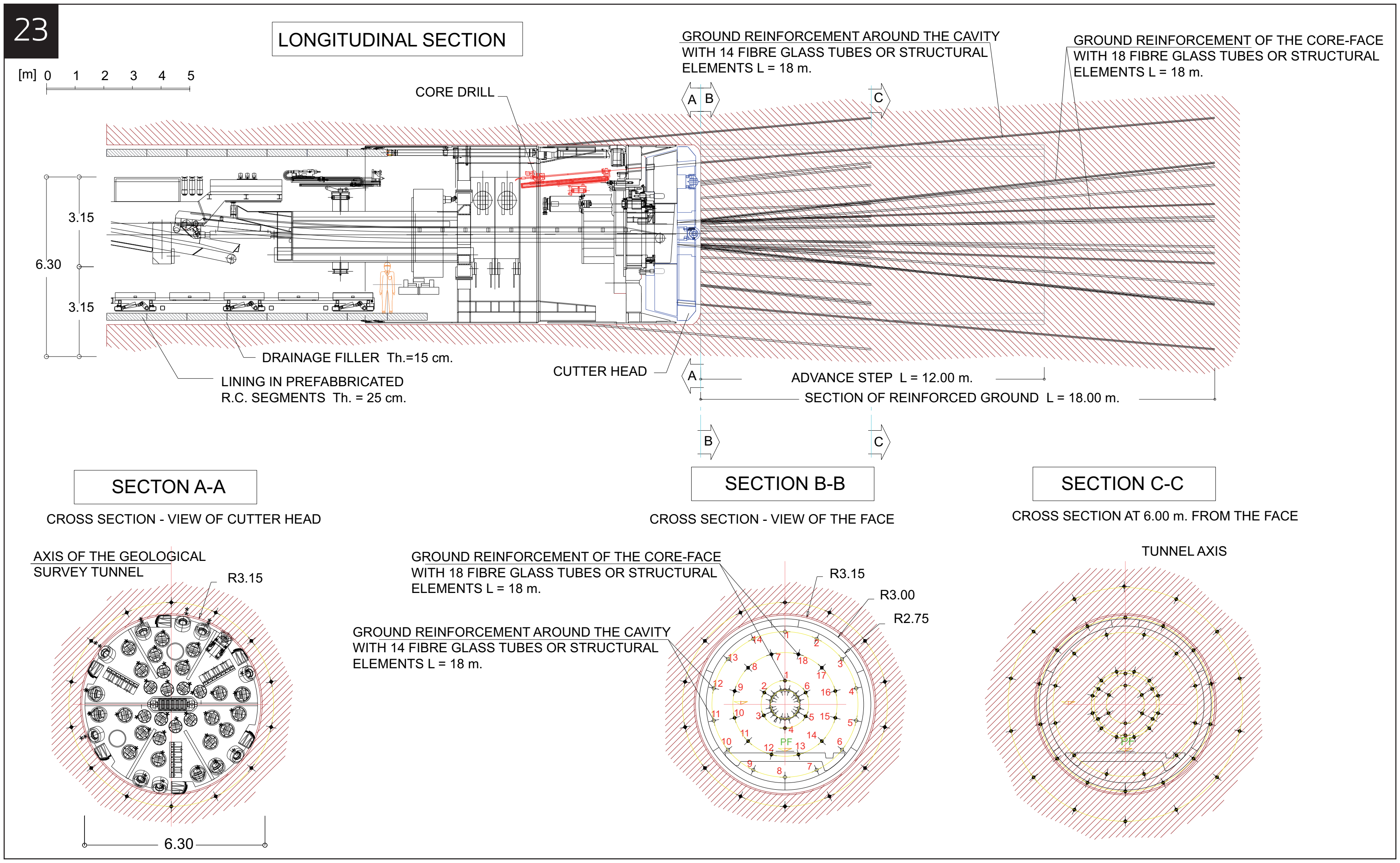
RECONSTRUCTED AFTER LUNARDI (2008)

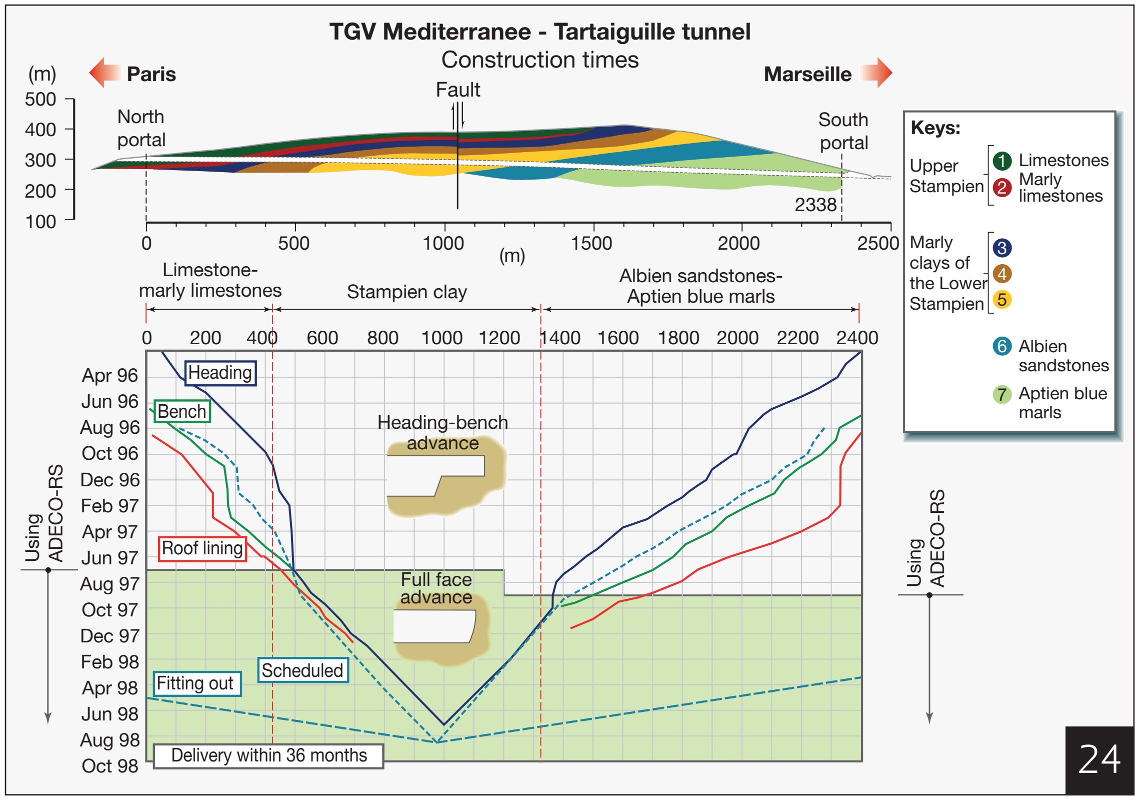
23. *BOLOGNE-FLORENCE HIGH-SPEED RAIL: TBM USED IN THE GINORI TUNNEL.*

AFTER LUNARDI (2008)



Section	ADVANCE RATES m/day (140 m² face, **FINISHED TUNNEL**) PREDICTED	ACTUAL	Section	ADVANCE RATES m/day (140 m² face, **FINISHED TUNNEL**) PREDICTED	ACTUAL
A Ac	5.4	5-6	C C1	1.4	1.4
B B1	4.3	5-5.5	C C4R	1.25	1.5
B B2	2.25	2.2			





24. TARTAGUILLE TUNNEL CONSTRUCTION TIME VERSUS GEOLOGY
AFTER LUNARDI (2008)

▸ By advancing full face under all conditions, large and powerful equipment can be used, which means that a lot of work can be done in a short time. This reduces cost and construction time.

▸ By concentrating all critical operations at the face, safety is greatly improved as opposed to sequential excavation, where many different (and critical, such as slashing the bench) construction operations spread out along the tunnel length.

▸ By advancing full face and minimizing squeezing, settlements are minimized, which, for example, is of paramount importance in urban area.

▸ Tunnels construction with and without a tunnel boring machine can be handled within the same approach (Figure 23).

CURRENT TUNNELING STATE-OF-THE-PRACTICE IN THE US

The Devil's Slide tunnel (Figure 25) illustrated in Figure 26 is used here to exemplify the current tunneling state-of-the-practice in the United States. The twin-bore road tunnel located in a rural area south of San Francisco along Interstate 1 is 1,280 m long and is designed to be accessible to bicyclists; each bore is 9 m feet wide and accommodates one vehicle lane and shoulder.

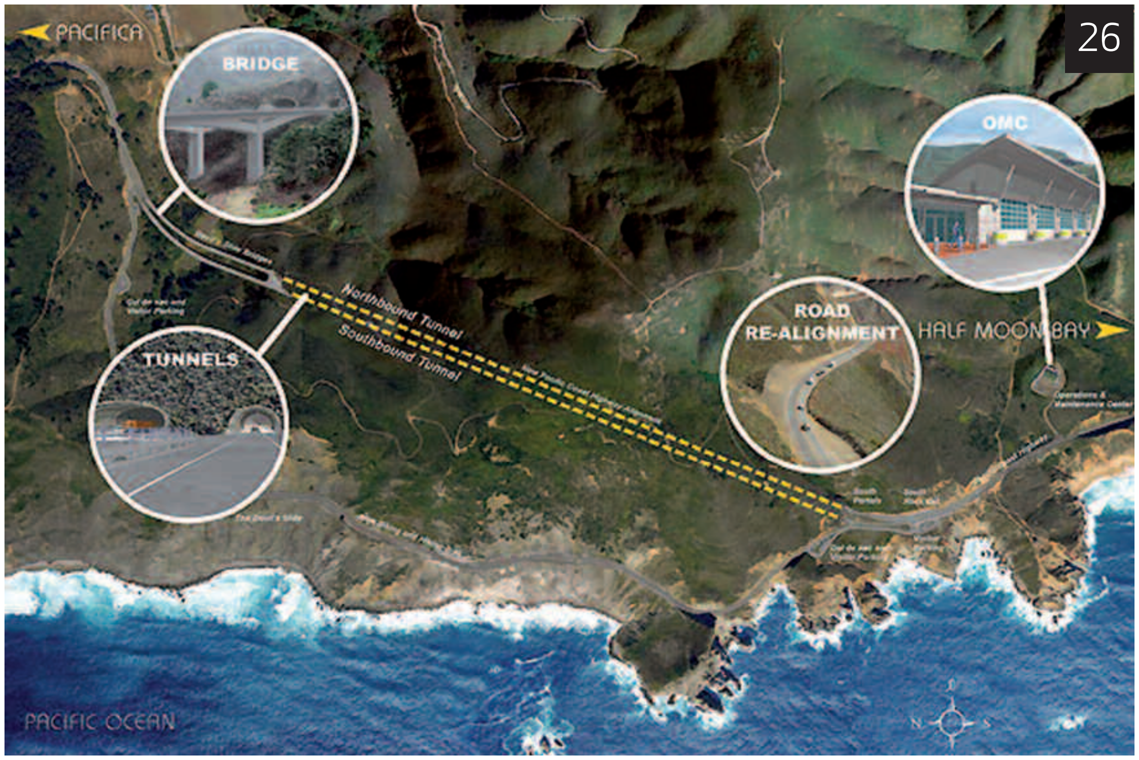
The tunnel contract also includes: 11 cross linking passages and 3 underground equipment rooms, day/night lighting system with brightness transition at each end, ventilation provided by 16 3.5-ft-diameter jet fans in each bore, and a control room. In 2007, the tunnel contract was awarded to the lowest bidder (Kiewit Pacific) for about $270 million; this figure is about 10/20 times the cost of a comparable tunnel in Italy.

As illustrated in Figure 27, the tubes are excavated top-heading and bench by using a "nose" of ground to stabilize the face (Figure 28) in the top heading. It is evident that such a nose:

- Faithfully applies Rabcewicz's dictates in his first NATM paper (1965): compare Figure 28 and Figure 29.
- Cannot apply large pressures because it only relies on gravity.
- It only applies pressure to the central part of the face, rather than uniformly to the entire face.
- Does not act in the core, but just on the face (cavity), hence it cannot help develop an arch effect ahead of the face and must be considered as a confinement measure (Figure 19), not as a preconfinement measure, and, as a result, the tunnel has to be excavated by using sequential excavation (see Rabcewicz's lack of understanding of the importance of the core);
- Cannot be engineered and fine-tuned as opposed to the preconfinement measures in Figure 19. For example: how can the contractor increase the applied pressure if convergence (only parameter monitored at the Devil's Slide) is greater than specified? Hence tunnel construction is not completely within the designer's control, and, as a consequence, construction schedule is not within the designer's control (see predicted vs. actual production rates below, Figure 22);
- It is a construction impairment because the roadheader used to excavate the tunnel is forced to move horizontally or in small vertical lifts, which defies the purpose of using a roadheader with a horizontally rotating drum (maximum productivity is achieved by full face excavation where the drum is moved vertically upwards and the upper ground is constantly undermined).

25. *DEVIL'S SLIDE TUNNEL PROJECT, CALIFORNIA, USA, REPRESENTS THE VERY BEST OF THE STATE-OF-PRACTICE IN THE UNITED STATES*

AFTER CALTRANS WEBSITE (WWW.DOT.CA.GOV/DIST4/DSLIDE), ACCESSED 16 OCTOBER, 2009



26. *DEVIL'S SLIDE TUNNEL PROJECT, CALIFORNIA, USA. THE BRIDGE CONTRACT IS SEPARATE FROM THE TUNNEL CONTRACT*

AFTER CALTRANS WEBSITE (WWW.DOT.CA.GOV/DIST4/DSLIDE), ACCESSED 16 OCTOBER, 2009

27. *DEVIL'S SLIDE TUNNEL PROJECT, CALIFORNIA, USA. TOP HEADING AND BENCH EXCAVATION*

AFTER CALTRANS WEBSITE (WWW.DOT.CA.GOV/DIST4/DSLIDE), ACCESSED 16 OCTOBER, 2009

28. *DEVIL'S SLIDE TUNNEL PROJECT, CALIFORNIA, USA. STABILIZATION OF THE TOP HEADING FACE BY USING A "NOSE" OF GROUND*

AFTER CALTRANS WEBSITE (WWW.DOT.CA.GOV/DIST4/DSLIDE), ACCESSED 16 OCTOBER, 2009

According to CalTrans website: "Tunnel boring progresses from south to north using excavation techniques that rely on inherent rock strength for support - known as the New Austrian Tunneling Method. Both tunnels are bored at the same time, one tunnel face approximately 60 yards ahead of the other. Staggering the bores reduces the likelihood of damage from blasting in alternate tunnels. The work schedule is 24 hours a day, 7 days a week resulting in a 24-month timetable to break through to the north portal".

In actuality, tunneling started on September 17th, 2007, and after 25 months (October 5th, 2009) progressed 2,684 ft (818 m) in the top-heading of the North-bound tube, and 2,487 ft (758 m) in the top-heading of the South-bound tube. Therefore, after 25 months, the top-heading of the North-bound tube was 62% complete, and the top-heading of the South-bound tube was 54% complete: compare this to the designer's predicted construction schedule of 24 months. This corresponds to an average advance rate of 1.07 m/day in the meager 65 m^2 North-bound tube top heading excavated in Class II or III according to Bieniawski in rural area, where surface settlement control is of no concern. Compare this advance rate for the top heading against the advance rates for 140 m^2 full face tunnel excavations constantly obtained with the ADECO in Figure 22. Many claims have been filed with the owner (Caltrans).

29. *RABCEWICZ'S STABILIZATION OF THE TOP HEADING FACE BY USING A "NOSE" OF GROUND*

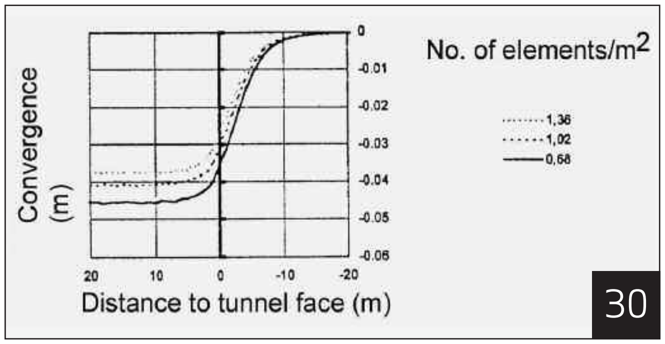
AFTER RABCEWICZ (1965)

OUTSTANDING ISSUES IN THE UNDERSTANDING OF PRECONFINEMENT MEASURES

Preconfinement Limits Convergence

It is often maintained that preconfinement measures (Figure 19) do not change the convergence curve of a tunnel (Pelizza and Peila 1993, Peila 1994, Peila et al. 1996, Oreste et al. 2004). This is equivalent to saying that the convergence measured at a large distance behind the tunnel face is not affected by the presence of preconfinement measures. As a consequence, when preconfinement is used, higher loads are predicted on primary lining/support and final lining than in the absence of preconfinement.

It is thus believed that preconfinement leads to more expensive tunneling solutions. However, Oreste's and Peila's (2000) 3D FEM elasto-plastic models in Figure 30 indicate that fiberglass elements inserted in the core reduce not only preconvergence, but also convergence in the cavity. The same conclusion is achieved from the analysis of Figure 31b, where a lined tunnel is modeled with different densities of fiberglass reinforcement in the core: preconfinement by fiberglass reinforcement of the core effectively controls convergence and therefore core reinforcement does change the convergence curve.



30. *CONVERGENCE VS. DISTANCE TO TUNNEL FACE FOR DIFFERENT DENSITIES OF FIBERGLASS REINFORCEMENT IN THE CORE*

MODIFIED AFTER ORESTE AND PEILA (2000)

31. EFFECT OF NUMBER OF FIBERGLASS ELEMENTS IN CORE ON: (A) EXTRUSION; (B) CAVITY CONVERGENCE AND PRECONVERGENCE.

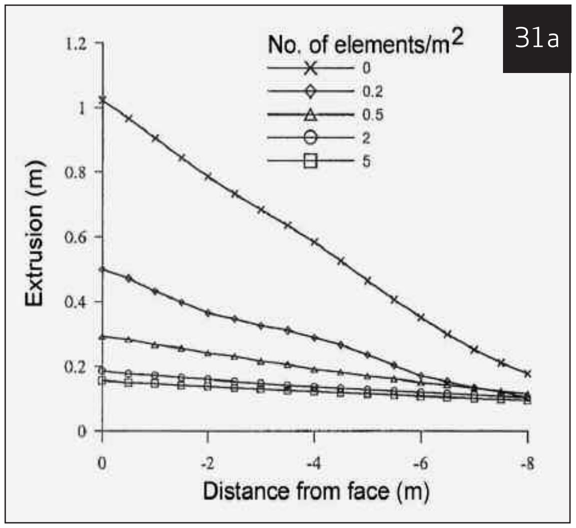
MODIFIED AFTER BOLDINI ET AL. (2000)

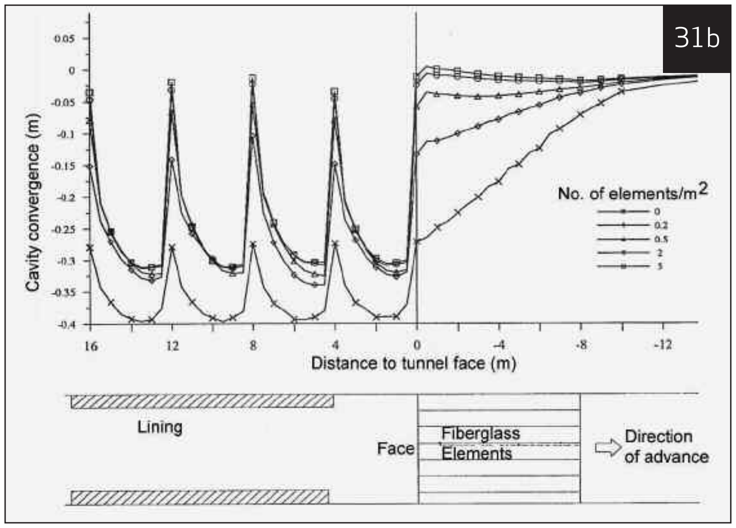
Likewise, field evidence gathered in over 500 km of built tunnels indicates that substantial savings have been achieved in the final lining when pre-confinement was adopted (Lunardi 2008). This contradictory evidence is currently being investigated by the author and his graduate students within the International Tunneling Consortium (ITC) that he established at the University of Texas at Austin (www.caee.utexas.edu/prof/tonon/ITC.htm). The objective of the International Tunneling Consortium (ITC) is to establish a collaboration between academia and the industry to foster research and education in tunneling by listening to the industry needs. The mission of the ITC is twofold:

- To carry out research on tunneling and underground construction as proposed by the members;
- to educate the next generation of tunnel engineers.

Comparison of Figure 31a and Figure 31b confirms that there is a 1-1 relationship between preconvergence and extrusion as already highlighted in Figure 9: as core reinforcement density increases, extrusion and preconvergence decrease together with maximum effect up to 1 element/m^2, beyond which small extrusion and preconvergence reductions are achieved for large increases in reinforcement density.

Finally, Figure 31b shows that convergence develops within only 1 m of the tunnel face, and this confirms the need to carefully design the preconfinement-confinement tran-





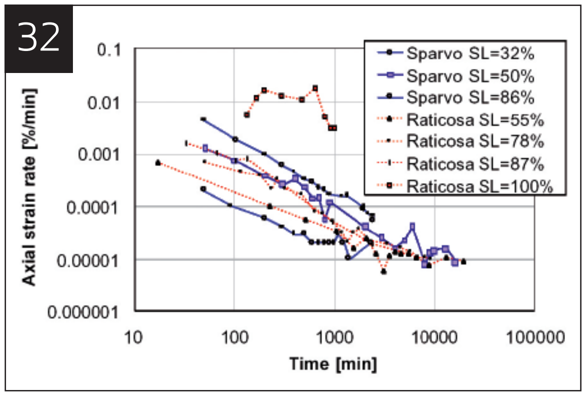
sitions in order not to waste the reduction in the preconvergence obtained with preconfinement ahead of the tunnel face as highlighted at the bottom of the rightmost column in Figure 15b.

Ground Time-dependent Behavior

Until now, all analyses of tunnels with preconfinement have used elasto-plastic models (e.g., Pelizza and Peila 1993, Peila 1994, Peila et al. 1996, Wong et al. 2000, Oreste et al. 2004, Marcher and Jiřičný 2005, Serafeimidis et al. 2007, Serafeimidis and Anagnostou, 2007). An exception is the study by Bonini et al. (2009): the authors, after a review of characterization and modeling of time-dependent behavior in rock, described the mechanical behavior of the Italian scaly clays, a structurally complex Clay Shale (CS) formation of the Apennines (Italy). Then, they identified the key factors involved in the selection of the constitutive model for CS and selected and discussed two constitutive models. Finally, they analyzed the Raticosa tunnel and compared the results of the models with the monitoring data in terms of radial convergence of the tunnel and extrusion of the tunnel face.

32. *ITALIAN SCALY CLAYS: (A) AXIAL STRAIN RATE VERSUS TIME FOR CREEP TESTS*

AFTER BARLA ET AL. (2004)



Although the study by Bonini et al. (2009) elucidates the applicability of the constitutive models proposed, the interaction between time-dependent ground behavior resulting in squeezing and/or swelling conditions and preconfinement measures has never been investigated in detail and currently the key geomechanical parameters and engineering preconfinement measures governing this interaction are not understood. Under these conditions, the author thinks that the effect of preconfinement is even more beneficial. Consider a classic rate-dependent viscoplastic model (Perzyna 1971, Simo and Hughes 1998), in which the strain rate is higher the farther the stress point is from the yield surface. For example, Figure 32 shows that, under sustained loading, the strain rate increases by nearly one order of magnitude even when the stress level (SL) increases from 50 to 86% of failure load. Because preconfinement allows stress points in the core and around the future cavity to remain closer to the yield surface, preconfinement should reduce strain rates in the ground and thus convergence and loads on primary support/reinforcement and final lining. In addition, full face advance allows for an immediate closure of the ring, and therefore even behind the face can stress points be kept as close as possible to the yield surface. Finally, fast rates of advance (Figure 22) maintained constant for the entire tunnel excavation minimize displacements. A thorough investigation needed to confirm/disprove these arguments is currently being undertaken by the author and his graduate students within the International Tunneling Consortium (ITC) at the University of Texas at Austin. In order to help guide the preconfinement design in these difficult stress-strain conditions, the ITC research will also develop quantitative understanding of:

- the interaction between time-dependent ground behavior resulting in squeezing and/or swelling conditions and preconfinement measures;
- the key geomechanical parameters and effect of engineering preconfinement measures in time-dependent ground.

CONCLUSIONS

Sequential excavation was started two hundred years ago; at that time, there was no electricity, horse and buggy were commonly used to move around (Figure 33); ladies wore crinolines and gentlemen wore top hats (Figure 34).

As originally conceived by Rabcewicz, the NATM did not necessarily embrace sequential excavation. Rather, Rabcewicz was completely in favor of full face advance but he realized that NATM did not allow him to advance full face in difficult stress-strain conditions.

33. *TYPICAL TRANSPORTATION MEANS IN THE EARLY 1800'S, WHEN SEQUENTIAL EXCAVATION WAS CONCEIVED*

The research and projects carried out by Lunardi indicate the reasons why Rabcewicz could not fulfill his dream in difficult tunneling conditions. He (and all his followers to date):

- Ignored the behavior of the advance core.
- Tried to counteract only convergence, which is the effect, instead of counteracting the very cause of instability, i.e. the deformation of the advance core.
- Used deformable linings, which allow the ground to deform and provide negligible confinement to the core.
- Let the ground deform and tried to mobilize the strength of the ground. In squeezing conditions, this practice allows the ground to start creeping, which is an irreversible phenomenon and is very difficult (if not impossible) to control by acting only on the cavity.
- Did not have the technology to pre-confine the core.

Ironically, continuing using the sequential excavation was a consequence of Rabcewicz's choices (not Rabcewicz's choice), which led him (and all of his followers to date) to give up on full face excavation, i.e. Rabcewicz's goal itself.

34. *OPENING OF THE TUNNEL UNDER THE TAMES IN THE EARLY 1800'S, WHEN SEQUENTIAL EXCAVATION WAS CONCEIVED*

We now know much more than in 1960s, we have much improved technology (in investigation, design and construction), we can deploy much more computational and construction power, and we have a complete design and construction approach that allows us to advance full face in all stress-strain conditions; it works with and without a tunnel boring machine. This approach has been proven in over 500 km of tunnels, the majority of which in difficult tunneling conditions. As for the United States, proceeding full face is just going back to the roots of early American tunneling. In the end, none of us

rides horse and buggy (Figure 33), nor wear crinolines or top hats (Figure 34) anymore. Let's update our tunneling approach as well!

We may still listen to the Beatles, but we do not take the risk and (fuel) cost of driving a 1964 Cadillac Fleetwood (Figure 35) across the US. Why should owners (and, eventually, taxpayers) across the US (and across most of the world) take the risk and pay the cost entailed in a 1964 tunneling approach? Will re-insurers still be willing to issue insurance policies to tunnel projects designed according to such a high-risk approach?

35. *1964 CADILLAC FLEETWOOD 60 SPECIAL SEDAN, PRODUCED WHEN THE NATM WAS CONCEIVED*

BIBLIOGRAPHY

BARISONE, G., PELIZZA, S. AND PIGORINI, B. (1982). Unbrella arch method for tunnelling in difficult conditions - analysis of Italian cases. Proc. IV International Association of Eng. Geol., New Dehli, Vol. IV, Theme 2, 15-27.

BERNAUD, D., ROUSSET, G.; (1992). La "Nouvelle Méthode implicite" pour l'Etude du Dimensionnement des Tunnels. Rev. Franç. Géotech. n° 60, pp.5-26 (juillet 1992).

BERNAUD, D., ROUSSET, G.; (1996). The New Implicit Method for Tunnel Analysis (Short Communication). Int. J. Numerical and Analytical Methods in Geomechanics, Vol. 20, pp. 673-690.

BOLDINI D., GRAZIANI A., RIBACCHI R., L'analisi tenso-deformativa al fronte e nella zona del retrofronte. In G. Barla, ed. "MIR 2000 - Lo Scavo Meccanizzato delle Gallerie". Patron Editore, Bologna, 2000.

BROWN, E.T. (1981). Putting the NATM into perspective. Tunnels & Tunnelling, November 1981, 13-17.

CARRIERI, G., FIOROTTO, R. GRASSO, P., AND PELIZZA, S. (2002). Twenty years of experience in the use of the umbrella-arch method of support for tunneling. International Workshop on Micropiles, Venice, Italy, May 30th-June 2nd 2002.

CORBETTA, F., BERNAUD, D., AND NGUYEN-MIHN, D.; (1991). Contribution à la Méthode Convergence-Confinement par le Principe de la Similitude. Rev. Franç. Géotech. n° 54, pp. 5-11 (janvier 1991).

GEOCONSULT; (1996). Bolu tunnel redesign technical report, Anatolian Motorway, Gümüova Gerede, Report no: 45.110/R/2118,KGM, Ankara. 10 pp.

GUO, C.; (1995). Calcul de Tunnels Profonds Soutenus, Mèthode Stationnaire et Mèthodes Approchèes. Ph.D. Thesis, ENPC, France.

KARAKUŞ, M. AND. FOWELL, RJ (2004). An insight into the New Austrian Tunnelling Method (NATM). KAYAMEK'2004-VII. Bölgesel Kaya Mekaniği Sempozyumu / ROCKMEC'2004-VIIth Regional Rock Mechanics Symposium, 2004, Sivas, Türkiye.

KOVÁRI K. (1994). Erroneous concepts behind the New Austrian Tunnelling Method, Tunnels & Tunnelling, November 1994, Vol. 26, 38-42.

LUNARDI, P. (2008). Design and Construction of Tunnels. Springer.

LUNARDI, P., CASSANI, G. AND GATTI, M.C. (2008). Design aspects of the construction of the new Apennines crossing on the A1 Milan-Naples motorway: the base tunnel. Proc. AFTES International Congress "Le souterrain, espace d'avenir"; Monaco October 6-8, 2008.

MARCHER, T. AND JIŘIČNÝ, F. (2005). ADECO RS versus NATM - A 3D numerical study. Tunnel, 2, 54-59.

NGUYEN-MINH, D. AND GUO, C.; (1993.A). A Ground-Support Interaction Principle for Constant Rate Advancing Tunnels. Proc. EUROCK '93, Lisboa, Portugal, pp. 171-177.

NGUYEN-MINH, D. AND GUO, C.; (1993.B). Tunnels Driven in Viscoplastic Media. Geotechnique et Environment Coll. Proc. Franco-Polonias coll., Nancy, France.

NGUYEN-MINH, D. AND GUO, C.; (1996). Recent Progress in Convergence Confinement Method. In Proc. EUROCK '96 (G. Barla ed.), 2-5 September 1996, Torino, Italy, Vol.2, pp. 855-860. A.A. Balkema, Rotterdam.

NGUYEN-MINH, D., GUO, G., BERNAUD, D., ROUSSET, G.; (1995). New Approches to Convergence Confinement Method for Analysis of Deep Supported Tunnels. Proc. 8th Congress International Society for Rock Mechanics, Tokio 2, pp. 883-887. A.A. Balkema, Rotterdam.

NGUYEN-MINH, D.; (1994). New Approaches in the Convergence Confinement Method for Analysis of Deep Supported Tunnels. Corso su "Trafori Alpini e Gallerie Profonde", Politecnico di Milano, Dip. di Ingegneria Strutturale, 19-23 Settembre 1994.

ORESTE P.P AND PEILA D., I consolidamenti come mezzo per permettere lo scavo meccanizzato in galleria. In G. Barla, ed. "MIR 2000 – Lo Scavo Meccanizzato delle Gallerie". Patron Editore, Bologna, 2000.

ORESTE, P.P, PEILA, D. AND PELIZZA, S. (2004). Face reinforcement in deep tunnels. Felasbau, 22, 20-25.

PANET, M., GUENOT, A.; (1982). Analysis of Convergence behind the Face of a Tunnel. Tunnelling '82. The Institution of Mining and Metallurgy. London.

PEILA, D. (1994). A theoretical study of reinforcement influence on the stability of a tunnel face. Geotechnical and Geological Engineering, 12, 145-168.

PEILA, D., ORESTE, P.P., PELIZZA, S. AND POMA, A. (1996). Study of the influence of sub-horizontal fiber-glass pipes on the stability of a tunnel face. In: North American Tunnelling '96, Balkema (NLD), ITA/AITES 1996 World Tunnel Congress, Washington DC, 425-432.

PELIZZA, S., PEILA, D. (1993). Soil and rock reinforcements in tunnelling. Tunnelling and Underground Space Technology 8 (3), 357–372.

PERZYNA, P. (1971). Thermodynamic Theory of Viscoplasticity. In Advances in Applied Mechanis, Academic Press, New York, 11.

PIEPOLI, G. (1976). La nuova galleria S. Bernardino della linea Genova-Ventimiglia. Ingegneria Ferroviaria, 10.

RABCEWICZ L. (1964). The New Austrian Tunnelling Method, Part one, Water Power, November 1964, 453-457, Part two, Water Power, December 1964, 511-515

RABCEWICZ L. (1965). The New Austrian Tunnelling Method, Part Three, Water Power, January 1965, 19-24.

ROMERO V. 2002. NATM in soft-ground: A contradiction of terms? World Tunnelling, 338-343.

SANDSTRÖM, G.E (1963). The history of tunnelling, underground workings through the ages. Barrie and Rockliff.

SERAFEIMIDIS, K., AND ANAGNOSTOU, G. (2007). The dimensioning of tunnel face reinforcement. In Proc. ITA-AITES World Tunnel Congress "Underground Space - the 4th Dimension of Metropolises", Prague May 2007.

SERAFEIMIDIS, K., RAMONI, M. AND ANAGNOSTOU, G. (2007). Analysing the stability of reinforced tunnel faces. In Proc. Geotechnical engineering in urban environments; XIV European conference on soil mechanics and geotechnical engineering, Madrid; Volume 2, 1079-1084; Millpress Science Publishers Rotterdam.

SIMO, J.C. AND HUGHES, T.J.R. (1998). Computational Inelasticity. Springer, New York.

WONG, H. , SUBRIN, D. AND DIAS, D. (2000). Extrusion movements of a tunnel head reinforced by finite length bolts - a closed-form solution using homogenization approach. International Journal for Numerical and Analytical Methods in Geomechanics, 24(6), 533-565.

Marc Panet

GROUND BEHAVIOUR AT THE TUNNEL FACE

The design of the excavation mode and of the support of a tunnel, especially in difficult grounds, must be based on a fair understanding of the behaviour of the ground in the vicinity of the tunnel face. The modern methods of analysis of the interaction between the ground and the support, such as the convergence-confinement method, rely on this understanding.

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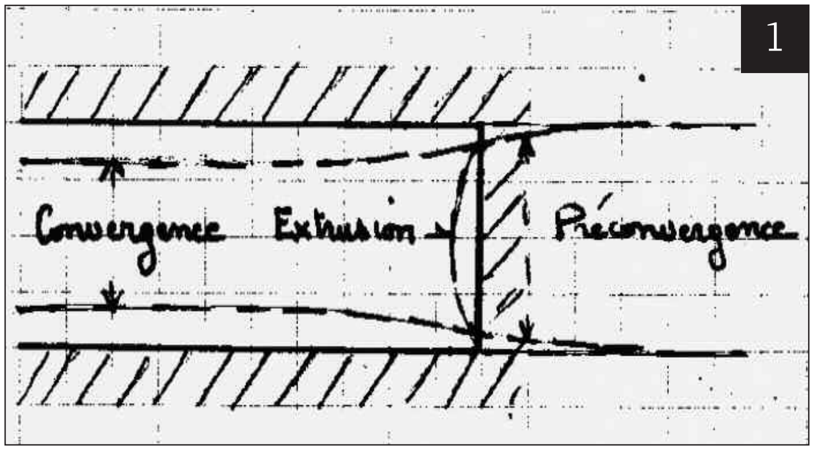
The construction method ADECO, successfully developed by Pietro Lunardi and his Rocksoil colleagues, takes into account not only the convergences which occur behind the face but also the preconvergence ahead of the face which is related to the face extrusion (fig. 1).
The monitoring during the tunnel excavation providing giving data on the convergences and on the face extrusion is a necessary tool to check the performance of the mode of support and of the presupport or to modify them, if necessary.



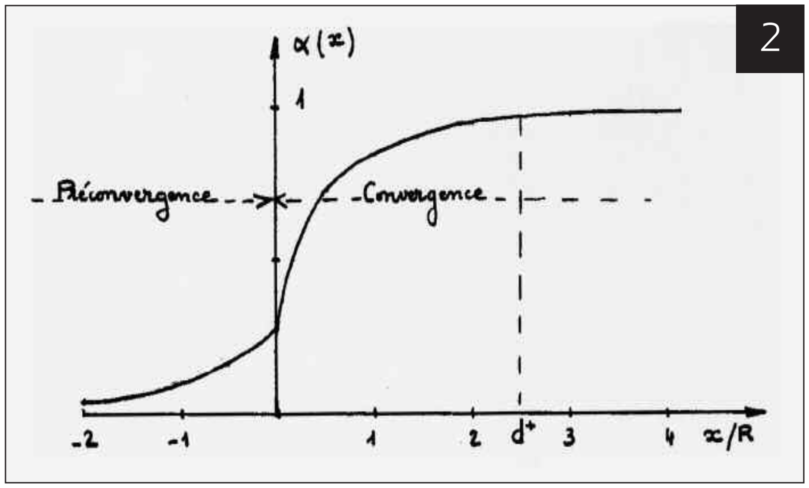
1. THE ELASTIC MODEL

Soils and rocks never behave elastically. However, the elastic model remains a useful reference for the analysis of the stresses and the displacements around a tunnel.

1.1 The convergence and the préconvergence

Let us consider the classic case of a circular tunnel driven in a homogeneous and isotropic ground with isotropic initial stresses σ^0.
The radial displacement far behind the tunnel face is given by the Lame formula:

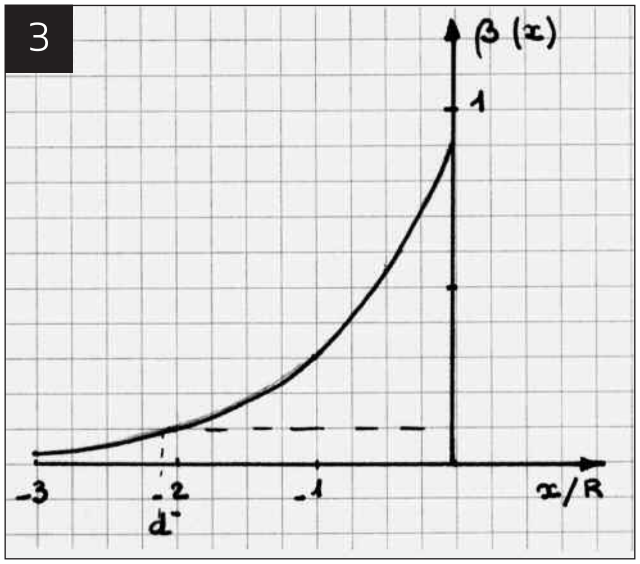
$$u_r(\infty) = \frac{\sigma^0 R}{2G}$$



If x is the distance to the tunnel face, x < 0 in the direction of the excavation advance,

$$u_r(x) = \alpha(x) u_r(\infty)$$

The graph of the function α (x) is given on fig. 2 for ν = 0,3.



1.2 The extrusion

The extrusion ahead of the tunnel face is the radial displacement u_x. For the circular tunnel excavated in a homogeneous and isotropic ground with isotropic initial stresses

$$u_x = \beta(x) u_r(\infty)$$

The graph of the function β (x) is given on fig. 3.

1.3 The distances of influence of the tunnel face

Two distances of influence of the tunnel face may be defined and have a practical importance:

- d+, behind the tunnel face ,defined by the expression:

$$\frac{u_r(\infty)-u_r(d^+)}{u_r(\infty)-u_r(0)}=0,9$$

- d-, ahead of the tunnel face, defined by the expression:

$$\frac{u_x(-d^-)}{u_x(0)}=0,1$$

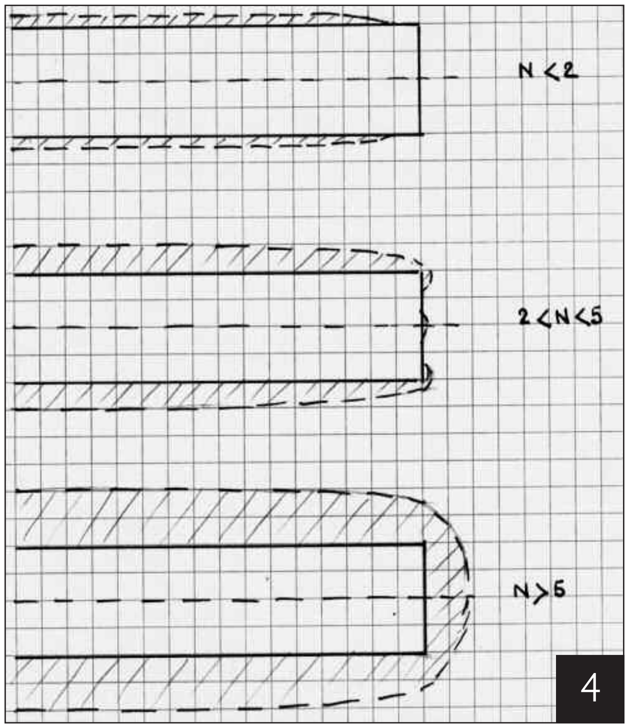
In the unsupported elastic case:

$$d^+ \approx 2,5R \qquad d^- \approx 2R$$

1.4 The stress distribution at the tunnel face

For a circular tunnel driven in a principal direction of the natural stress tensor, the maximal stress in the middle part of a plane face is given by the expression:

$$\max\sigma_f = C_1\sigma_1^0 - C_2\sigma_2^0 - C_3\sigma_3^0$$



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σ_1^0 and σ_2^0 are the principal initial stresses in the plane orthogonal to the tunnel axis ($\sigma_1^0 > \sigma_2^0$);

σ_3^0 is the principal initial stress in the direction of the tunnel axis;

The coefficients C_1, C_2 and C_3 depend on the Poisson's ratio. For $\nu = 0,2$.

$$C_1 = 1,3,\ C_2 = 0,075,\ C_3 = 0,7$$

The central part of the face is usually in compression and a rupture in compression is possible. If the face is concave the compression is larger.

2. AN ELASTO-PLASTIC GROUND BEHAVIOUR

When the initial stresses are sufficiently large, in the vicinity of the excavation permanent deformations with yielding or fracturing may occur.

The type of failure depends on the behaviour of the rock. It may be a real plastic deformation with shear surfaces, a damage zone with microfissuration, a brittle failure of the rocks which sometimes may be brutal (rock bursting), a failure by buckling of layers in orthotropic rocks when the discontinuities are parallel to a surface of excavation.

An elastoplastic behaviour is usually considered to analyze the stress and strain distribution around the excavation. This zone is usually called the "plastic zone".

The stability coefficient:

$$N = \frac{2\sigma^0}{\sigma_C}$$

is to be considered, where:
σ^0 is a representative value of the initial state of stress, most often computed from the weight of the overlying ground γH, e σ_c is the uniaxial compressive strength.

If the initial stress tensor is known, N may be taken equal to:

$$N = \frac{3\sigma_1^0 - \sigma_3^0}{\sigma_c}$$

σ_1^0 and σ_3^0 being the principal stresses in the plane orthogonal to the tunnel axis.
Three cases may be distinguished :
- N < 2, there is no plastic zone which includes the tunnel face;
- N > 5, the plastic zone is large and includes the completely the tunnel face;
- 2 < N < 5, the plastic zone includes only partially the tunnel face.

The difficult grounds which include plastic soils, squeezing rocks, or heavy rock bursting correspond to the case N > 5.

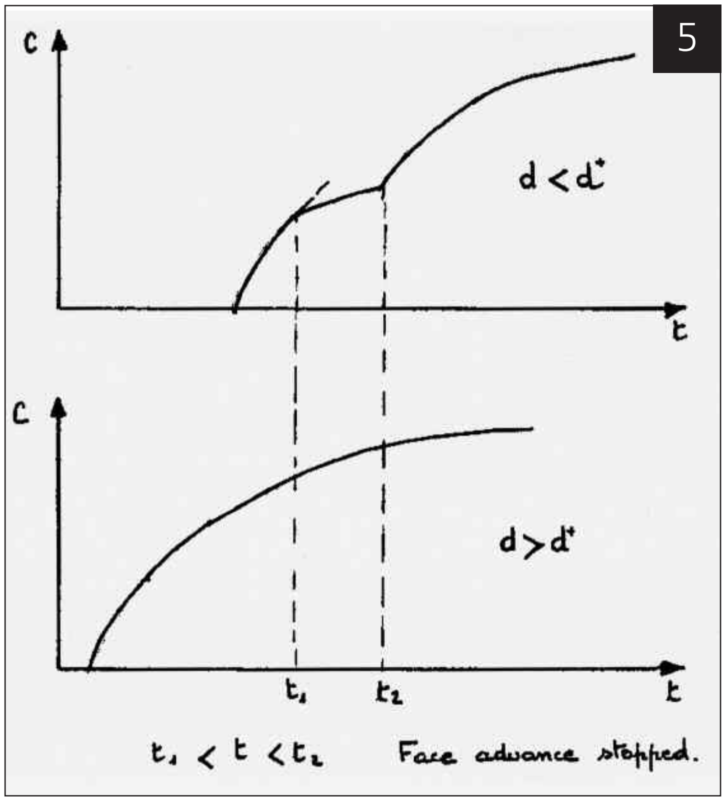
The distances of influence of the tunnel face increase with the extension of the plastic zone.
For a circular tunnel it may be estimated:

$$d^+ \approx 2{,}5R_p$$

$$d^- \leq R_p$$

Rp being the radius of the plastic zone.

The extension of the plastic zone may be evaluated. Two methods may be used in practice:
- The most precise one is to use the data of extensometric measurements in radial and axial boreholes , the boundary of the plastic zone is fixed by the deformation at yielding (about $1 \cdot 10^{-3}$ to $5 \cdot 10^{-3}$ according to the ground behaviour).
- The second is to consider the distance of influence of the



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face d+. It may be evaluated from the interpretation of the convergences after a stop of the advance of the face excavation. When the convergence measurement section is at a distance of the face d < d+ an increase of the convergence rate is observed at the resumption of the excavation.

The radial extension of the plastic zone is about d+/2,5.

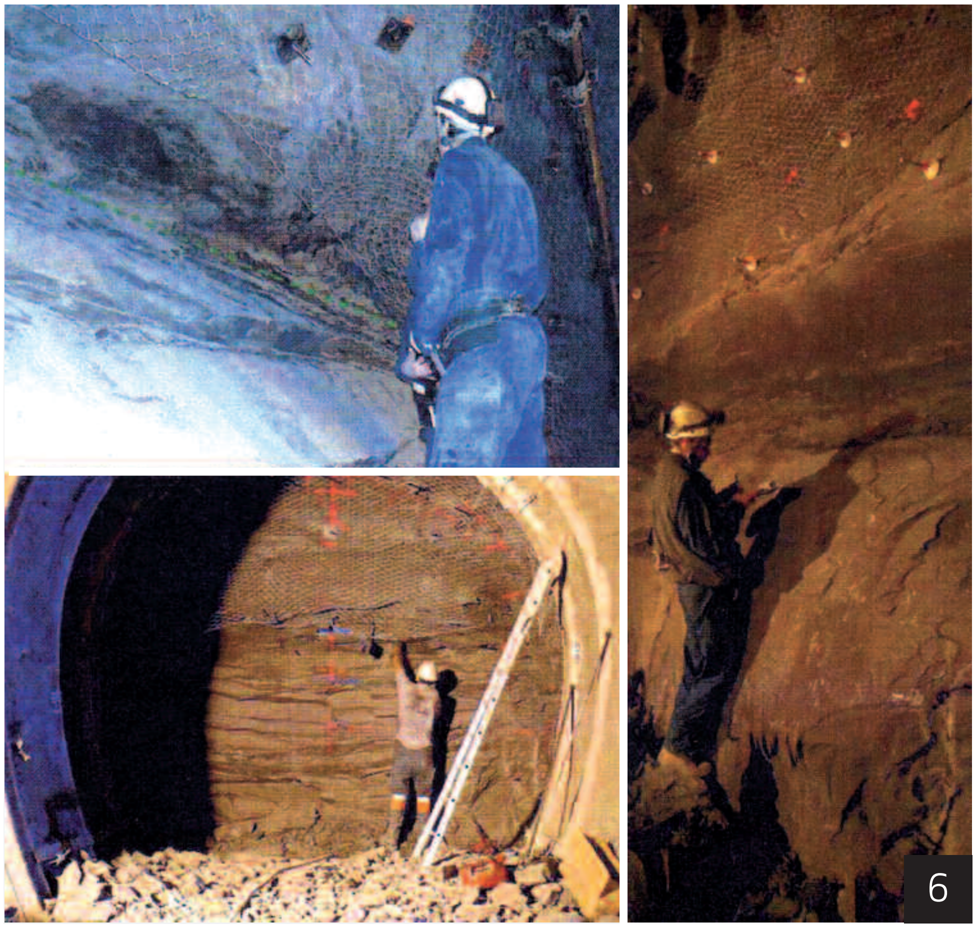
The elastoplastic models are more and more popular to analyse the ground behaviour around the tunnels and the ground interaction with the support. The possibilities of the numerical models bring about more and more sophisticated analyzes in 2D or 3D. They may now take into account various plasticity and damage criteria, hardening or softening laws. In practice, the true limit of these models is the actual possibility to determine the physical parameters. If sophisticated laboratory tests allow sometimes to face this complexity, these tests are rarely representative at the tunnel scale, especially in rock masses, or heterogeneous masses. However they may be useful for back analysis of in situ measurements.

3. THE FACE UNSTABILITIES

Various unstabilities may be observed on the tunnel face:

- In rock masses, some familiar block unstabilities result from the presence of intersection of discontinuities. They occur frequently in sedimentary or schistous formations with bedding planes or planes of schistosity dipping towards the excavation.
- In soft plastic rocks, shear surfaces ahead of the face due to the too large compression of the core may create failures. Such failures are very obvious in clayey rocks during the excavation of experimental galleries of the Bure Underground Laboratory for radioactive wastes (Meuse-Haute Marne in France) at a depth of 490 m.

In brittle rocks like granitic rocks, the failures may occur by scaling or rockbursting. Such violent phenomena occur during the driving of the Mont Blanc Tunnel between France and Italy. Among the165 000 rock bolts used on the French side to stabilize the walls of the excavation, 28 500 were installed at the face to allow the drilling of the blastholes. The technology of fibreglass rockbolts did not exist and the steel rockbolts brought about some difficulties for mucking.



6. UNSTABILITIES AT THE FACE OF A GALLERY OF THE UNDERGROUND LABORATORY AT BURE (ANDRA)

4. THE FACE REINFORCEMENT AND PRESUPPORT

The reinforcement of the face is an important factor in the tunnel design and construction approach in difficult ground. In the ADECO-RS approach it allows a full face advance. Full face excavation has many advantages for the work organisation and allows greater rates of tunnel excavation.

The reinforcement of the face may be achieved by bolting or jet-grouting ahead of the face. The presupport by umbrella vault may be realized by grouted pipes, jet-grouted columns or precut shells.

These various techniques have been put into practice in many works and have been described in a great number of papers.

The face reinforcement by fibre glass elements is more and more popular; it may be combined with various techniques of presupport (fig. 7).

Fibre glass bolts are made of a composite material of fibre glass and resin. Their shape is tubular, plate or that of a star with three branches. Their deformation modulus is about 40.000MPa, their tensile strength about 1.000MPa and the rupture deformation larger than 3%.

The density of face bolting is 2mx2m to 1mx1m. Their length vary from 15m to 24m; with an overlapping of one third of the length. The bolts are put in horizontal boreholes ∅ 100mm driven by specific equipment developed by Italian manufacturers. The correct grouting of the bolts is essential

However the actual performance of this technique to reduce the preconvergence and the extrusion of the face is not disputable but is still difficult to evaluate in the design.

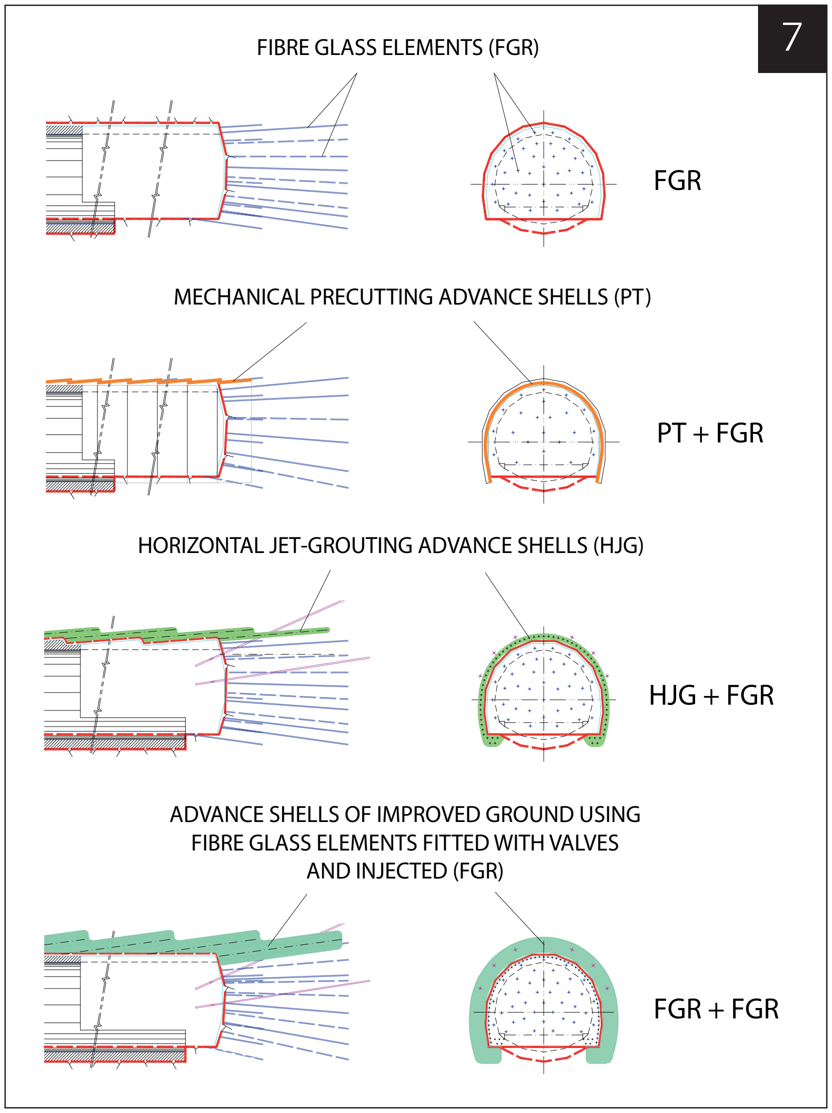
Presently there are four different approaches:

- The simplest and maybe the most usual way is to consider an equivalent pressure pf acting on the face:

$$p_f = \frac{\Sigma F_i}{S}$$

7. *VARIOUS TECHNIQUES OF PRESUPPORT ASSOCIATED WITH FIBER GLASS BOLTS*

(P. LUNARDI)



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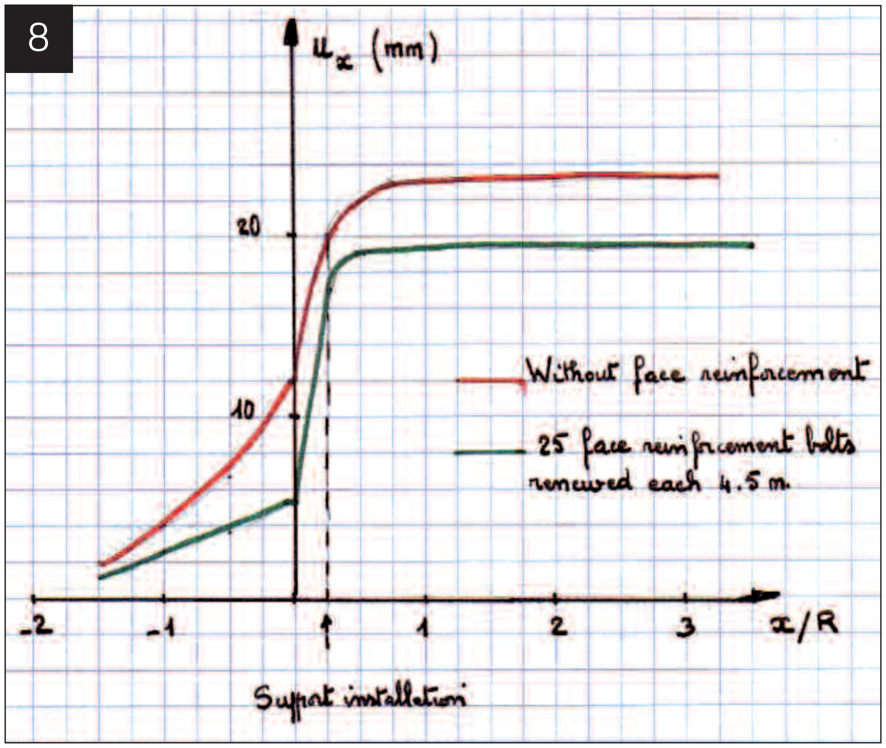


8. THE BOLTING INFLUENCE ON THE PRECONVERGENCE

- F_i, force which may be mobilized on the bolt i,
- S, section of the face.

Extrusion measurements allow having a better evaluation of F_i.

- In a convergence-confinement analysis, the rate of deconfinement at the face λ_f may be reduced. There is no well established rule to determine the rate of deconfinement at the face in relation with the bolting parameters. Usually λ_f is derived from p_f.
- It is possible to do a 3D numerical model where the modelization of the ground and of each bolt. These models are very complex and most of the time, the results are very deceiving. Such models must be considered as research tools.
- The characteristics of the bolted ground core ahead of the face may be determined by homogenization. Homogenization techniques of reinforced ground are now available and efficient.

Quest'ultimo approccio è stato utilizzato da Bernardet nel caso della canna sud del tunnel autostradale di Tolone, la cui sezione misura 120m^2.

This last approach was used by Bernardet in the case of the South tube of the highway Toulon tunnel the section of which is 120m^2. It is a shallow tunnel with a very rigid support installed close to the face to control strictly the surface settlements. The front is reinforced by 25 fibre glass bolts renewed every 4.5m. Fig. 8 shows clearly the bolting influence on the preconvergence.

L'ultimo promettente sviluppo della ricerca è una omogeneizzazione in più fasi, in cui il terreno armato è considerato come la combinazione di due mezzi continui, il terreno e l'insieme dei bulloni. Le interazioni terreno-bulloni sono trattate esplicitamente (Buhan).

5. A TUNNEL IN DIFFICULT GROUND: THE SAINT MARTIN LA PORTE ADIT

For the study of the future Lyon-Torino railway tunnel, an adit (section 80m^2, length 2km) has been driven at Saint Martin la Porte in highly fractured and heterogeneous Carboniferous rock mass. From back analysis, the uniaxial compressive strength of the rock mass has been estimated to be about 0.5MPa. With a cover between 250m et 300m, the stability ratio N is about 14. These very severe conditions gave a very heavy squeezing with measured convergences up to 2m (fig. 9).

The initial mode of excavation and support had to be adapted to these unexpected conditions:

- Modification of the cross section to an almost circular shape.
- Three phases of support:

1. As close as possible from the face, a closed steel set (TH 44/58) with a radius of 6.57m and 6 sliding joints, 37 bolts 8 metres long.
2. At a distance variable between 15m and 30m from the face, a steel set (TH 44/58) with a radius of 6;07m and 9 sliding joints, a 20cm shotcrete shell with 9 compressible elements (In some circumstances, it was necessary to take off the first steel set).
3. At about 80m from the face, the final support with a 1m thick concrete lining.
The core ahead of the face was reinforced by 40 fibre glass bolts 16m long which were renewed every 10m.
The monitoring of the excavation work: convergences, axial extrusions, radial displacements, lead to the following conclusions:

- At 60m from the face, the extension of the plastic zone is about 12m to 16m from the walls of the tunnel (R_p is about 18m to 22m)
- The distance of influence of the tunnel face d+ is about 50m (2.5 R_p).
- The extrusions are low. The extrusion at the face is about 5 and the distance of influence of the face d- is no longer than 4m.

The surprising low values of $u_x(0)$ and of d-are difficult to explain by the face reinforcement. It may be necessary to take into account an anisotropic behaviour of the rock mass.

9. THE SAINT-MARTIN LA PORTE ADIT

THESE VERY SEVERE CONDITIONS GAVE A VERY HEAVY SQUEEZING WITH MEASURED CONVERGENCES UP TO 2M

BIBLIOGRAPHY

BERNARDET A, H. OUFFROUKH, About the confining loss estimate with axisymmetric modelling

GUILLOUX A., B. KAZMIERCZAK, A. KURDTS, G. REGAL, WONG H, Stabilité et renforcement des fronts de taille des tunnels: une approche analytique en contraintes et déformations. Tunnels et Ouvrages Souterrains, 188, mars-avril 2005

GREUEL E., P. DE BUHAN, M. PANET, J. SALENÇON, Comportement des tunnels renforcés par des boulons passifs, XIII Int. Cong. Soil Mech. Found. Eng. New Delhi 1993

HOEK E., C. CARRANZA-TORRES, M. DIEDRICHS, The 2008 Kersten lecture: Integration of geotechnical and structural design in tunneling, Proc. University of Minnesota, 56^{th} Annual Geotechnical Geotechnical Engineering Conference, 2008

LECA E., L. DORMIEUX, Upper and lower bound solutions for facestability of shallow circular tunnels in frictional materials, Geotechnique, 40, 1990

LUNARDI P., La stabilité du front de taille dans les ouvrages souterrains en terrains meubles, études et expériences sur le renforcement du noyau d'avancement, Int. Symp. Renforcement des Sols, Paris 1993

LUNARDI P., Design and constructing tunnels – ADECO-RS approach, T&T, International special supplement, May 2000

LUNARDI P., R. BINDI, The evolution of reinforcement of the advance core using fibre-glass elements, Felsbau, 22, n° 4, 2004

PANET M., Le calcul des tunnels par la méthode convergence-confinement, Presses Ponts et Chaussées, 1995

RENAUD J., Le tunnel routier sous le Mont Blanc. Exécution des travaux de dérochement et de revêtement coté France, Travaux, déc.1962

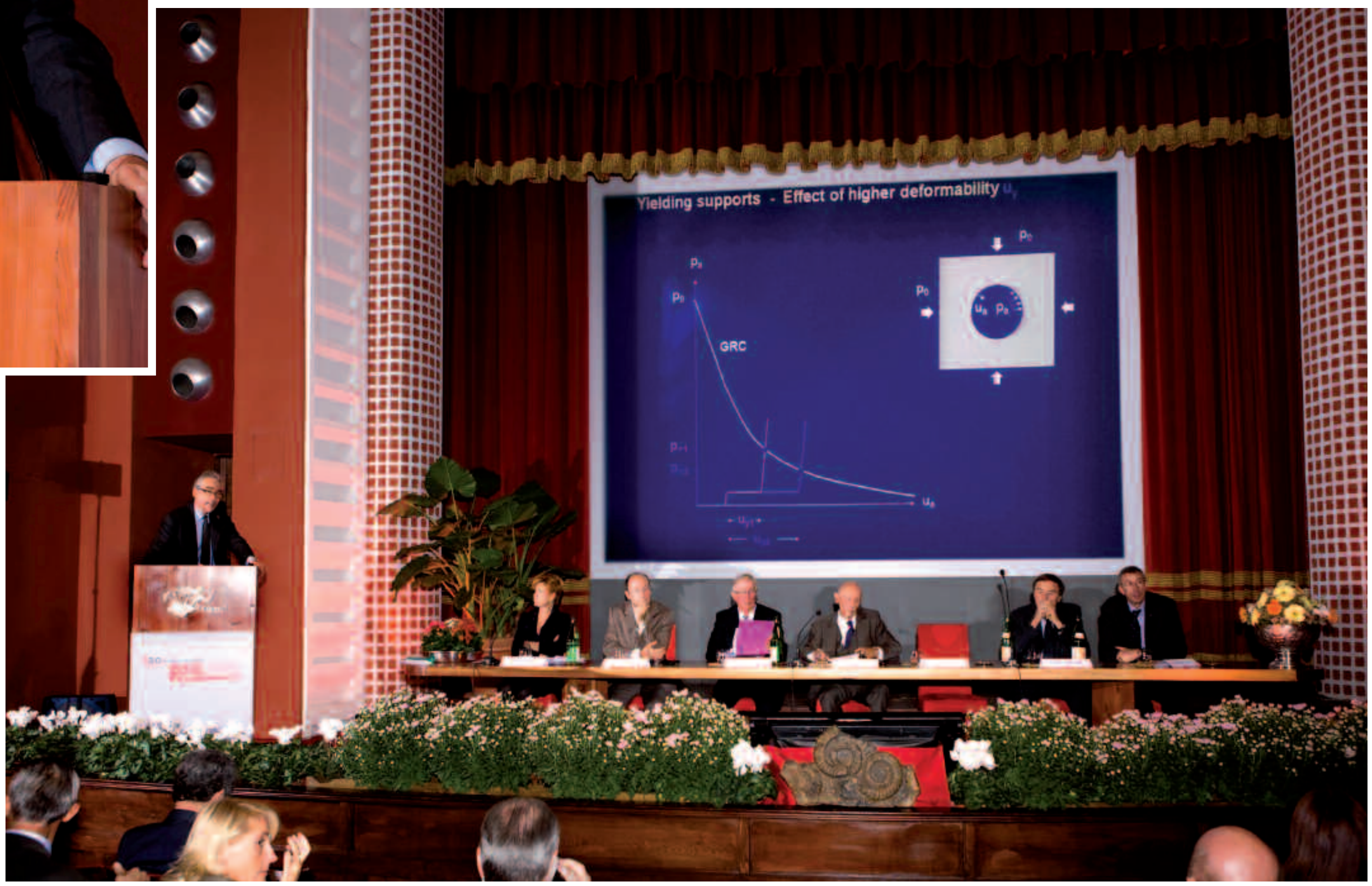
RETTIGHIERI M., J. TRICLOT, E. MATHIEU, G. BARLA, M. PANET, Difficultés liées aux fortes convergences rencontrées lors du creusement de la descenderie de Saint Martin la Porte, Int. Symp. AFTES, Monaco, 2008


Georgios Anagnostou

ABOUT THE ADVANCING FACE SPATIAL EFFECTS IN TUNNEL ENGINEERING

GEORGIOS ANAGNOSTOU, ETH ZURICH / LINARD CANTIENI, ETH ZURICH / MARCO RAMONI, ETH ZURICH

The paper illustrates the importance of spatial effects in tunnelling by addressing a number of practical design and analysis issues. It also provides an overview of recent research at ETH Zurich dealing with the evolution of the stresses and deformations around the advancing face. At the same time, it shows that taking due account of spatial effects leads to results which may be qualitatively different to those obtained through plane strain analyses.



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1. INTRODUCTION

In tunnel design it is essential for a number of questions to take due account of the evolution of the stress and deformation fields in the ground around the advancing tunnel face. It is well known, for example, that the main source of settlement in closed-shield tunnelling is localized in the shield area. The case of squeezing ground provides another example from a completely different context. When tunnelling with a shielded TBM, limited levels of convergence will not cause problems, thanks to the gap between the shield and the surrounding ground. If the gap closes, however, pressure will start to develop upon the shield. In this way, squeezing ground may slow down or even obstruct TBM advance. The quicker the development of the convergences, the higher the risk of shield jamming. The rate of convergence close to the tunnel heading is also important for conventional tunnelling; rapidly developing squeezing may slow down the advance rate considerably, as the installation of the support required for controlling the ground interferes with the actual excavation work.

The recognition of such spatial effects – and of the importance of taking them into account at the design stage – came relatively recently in tunnelling practice. Professor Lunardi was among the first engineers not only to attempt to understand what happens in the ground ahead of the tunnel face, but also to appreciate the importance of spatial effects in his tunnel designs. Today's celebration therefore gives us an opportunity to revisit some related issues that are relevant from a practical perspective and to give an overview of recent research at ETH Zurich into the evolution of stress and deformation around the advancing tunnel face.

The use of a planar statical system to estimate ground deformations or pressures in the vicin-ity of the face introduces uncertainties as a result of the need to make a priori assumptions about the development of deformations and stresses in the longitudinal direction. Plane models also fail to take account of important information concerning the ground response to exca-vation. In more specific terms, the current paper shows that if we take adequate account of the third dimension – the stress history of the ground together with the sequence of excavation and support installation – we will obtain results, which may be markedly different – both quantitatively and qualitatively – to those obtained through plane strain analyses. We might add that these results are surprising at first glance and cannot be reproduced from two-dimensional models.

The paper subsequently considers the development of surface settlement during the continu-ous excavation of a shallow tunnel crossing a low-permeability saturated ground (Section 4), and continues with the issue of shield jamming in overstressed rock (Section 5) before closing with some results obtained on the interaction between yielding supports and squeezing ground (Section 6). Before addressing these practical questions, however, some general issues are discussed concerning the computational mechanics of an advancing tunnel heading (Section 2) and a fundamental shortcoming of plane strain computational models (Section 3).

2. NUMERICAL MODELLING OF CONTINUOUS EXCAVATION

The numerical modelling of an advancing tunnel heading is particularly demanding in the case of time-dependent ground behaviour. The time-dependency of ground behaviour may be linked to consolidation, creep and, in some rocks, chemical processes as well. It manifests itself in a variety of ways depending upon both the type of ground and the construction method, and may have important implications for the construction process or the life of a tunnel.

Creep is associated with the rheological properties of the ground and becomes evident if the ground is overstressed – particularly as the failure state approaches. It is, therefore, of paramount importance in the case of weak rock under high stress (squeezing conditions).

The time-dependency of low-permeability soft ground is due mainly to transient seepage flow processes which are triggered by tunnel excavation and which develop slowly over the course of time. The long-term deformations of the ground generally include changes to its pore volume and water content (the latter requires more or less time depending on the seepage flow velocity and thus on the permeability of the ground). In a low-permeability ground, the water content remains constant in the short term. Tunnel excavation, however, generates excess pore pressures. As these will be higher in the vicinity of the tunnel than further away, seepage flow starts to develop. The excess pore pressures therefore dissipate over the course of time, thereby altering the effective stresses and leading to additional time-dependent deformations (consolidation). The permeability of the ground has a decisive effect on the rate of pore pressure dissipation and thus on the time-development of ground deformations or (where the latter are constrained by a lining) ground pressure.

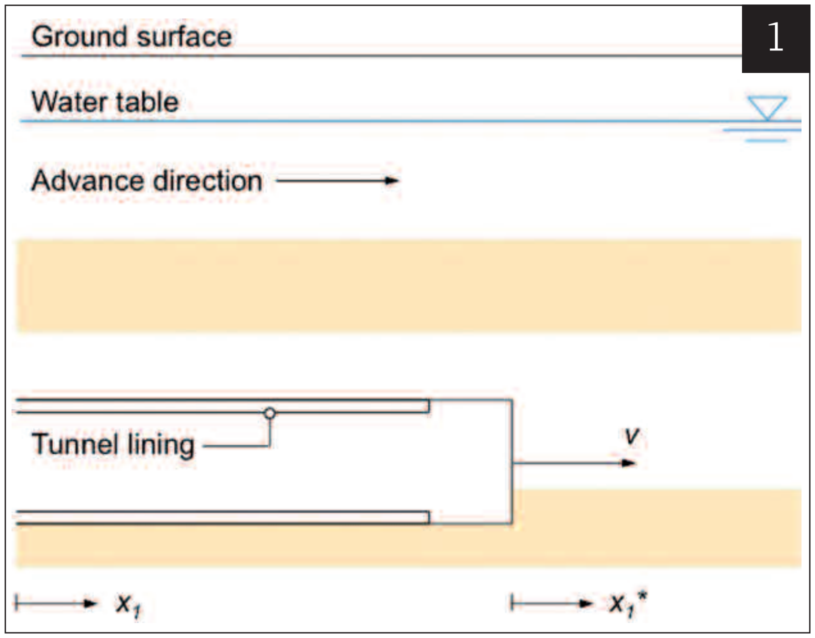
In geological conditions where there is pronounced time-dependent ground behaviour during construction (e.g., shallow tunnels through clay deposits or deep tunnels through weak rock), the advance rate greatly influences the development of ground pressure and deformation in the region around the working face because the deformations caused by creep or consolidation are superimposed upon those caused by the three-dimensional redistribution of stress that results from excavation.

As far as consolidation processes are concerned, the higher the advance rate and the lower the permeability, the less dissipation there will be in respect of pore pressures in the vicinity of the face (leading to so-called "undrained" conditions) and, consequently, the deformations will be smaller. If, on the other hand, the permeability is high and the advance rate low, drained conditions will prevail in the vicinity of the working face, and these are less favourable.
Depending on the ratio of advance rate to permeability, the ground response will be undrained, drained or somewhere in between. The ratio of advance rate v to permeability k governs stability, deformation and the ground pressure acting upon a lining or a shield in the vicinity of the face.

Similar considerations apply to creep (Ghaboussi and Gioda, 1977): the lower the advance rate, the greater the deformation. In the borderline case of a very high advance rate, only small, elastic deformations will develop in the vicinity of the face. A high advance rate is therefore advantageous, as the ground deformations close to the face will be smaller, regardless of whether the reason for the time-dependency is creep or consolidation.

An advancing tunnel heading is usually simulated (cf., e.g., Franzius and Potts, 2005) as a successive removal of soil elements and an addition of elements representing the lining or the shield, i.e. as a statical system that changes stepwise according to a sequence of excavation and support operations. Such an approach, although quite natural, can be very problematic in the case of time-dependent ground behaviour. The reason for this is the previously mentioned simultaneous redistribution of spatial stress around the advancing face, together with consolidation or creep processes in the ground. Since high deformation gradients and pore water pressure gradients prevail in the vicinity of the tunnel face, either the finite element mesh has to be fine everywhere along the tunnel axis or adaptive re-meshing must be carried out for each excavation step in the simulation. The analysis therefore becomes extremely time-consuming (even in the case of linear material behaviour) and generally presents serious problems in terms of both numerical accuracy and stability.

It should be noted, however, that in the case of uniform conditions in the direction of tunnelling (Figure 1), the stress fields and deformation fields are steady with regard to the tunnel heading, i.e. they "advance" together with the face in the direction of excavation. For this large class of problems, the step-by-step solution method is highly inefficient, as it approaches the steady state asymptotically after the simulation of several excavation steps. The advancing heading problem can instead be solved by means of a single computational step, i.e. without the need for integration in the time-domain. The central idea in respect of achieving this can be traced back to the work of Nguyen-Quoc & Rahimian (1981) on crack propagation in elastoplastic media and it involves eliminating the time-coordinate from the equations governing the steady state by carrying out appropriate transformations. Since the stress, pore pressure and deformation fields are apparently time-independent for an observer moving with the tunnel face, the obvious solution is to re-formulate the continuum-mechanical equations in a frame of reference that is fixed to the advancing heading (co-ordinate x_1* in Figure 1). One might almost say that this method, rather than attempting to model the advancing tunnel



__1.__ LAYOUT OF A PROBLEM HAVING CONSTANT CONDITIONS IN THE HORIZONTAL DIRECTION WITH RESPECT TO GEOLOGY, INITIAL STRESS, DEPTH OF COVER AND WATER LEVEL

(THE CO-ORDINATE X_1 IS SPATIALLY FIXED, WHILE THE CO-ORDINATE X_1 IS FIXED TO THE ADVANCING TUNNEL HEADING)*

face in a naturalistic way, considers the case of a mountain moving around a spatially-fixed tunnel face.

Corbetta (1990) and Anagnostou (1993) followed this approach when solving the viscoplastic tunnelling problem and when analysing transient seepage flow, respectively, while a recent work (Anagnostou, 2007) has extended this method for the coupled problem of tunnel excavation through a porous, saturated, elastoplastic ground. As shown in this work, almost all continuum-mechanical equations remain valid in a face-fixed co-ordinate system, the only exception being the mass balance equation, which is enhanced by one additional term incorporating the ratio v/k of advance rate to permeability as a parameter:

$$\nabla\nabla h = -\frac{v}{k}\frac{\partial \varepsilon_{vol}}{\partial x_1} \qquad (1)$$

where h and ε_{vol} denote the hydraulic head and the volumetric strain of the ground, respectively, while x_1 is the excavation direction (Figure 1). The numerical solution of the enhanced coupled seepage flow and stress analysis equations yields the steady state displacement and hydraulic head fields around the advancing tunnel face straightforwardly in a single step, i.e. without the need for a time-iteration. The difficulties associated with the accuracy and stability of a marching scheme in the time-domain therefore do not arise in this method.

3. THE NON-UNIQUENESS OF THE GROUND RESPONSE CURVE

Computational models based on planar statical systems consider a tunnel cross-section far behind the tunnel face and assume plane strain conditions.
Where there is rotational symmetry, the plane strain problem is mathematically one-dimensional. The so-called "characteristic line of the ground" – also referred to as the "ground response curve" (Panet and Guenot, 1982) – expresses the relationship between the radial stress p and the radial displacement u of the ground at the excavation boundary. Assuming equilibrium and compatibility between ground and support, the ground response curve can be employed in combination with the characteristic line of the support in order to estimate the radial convergence of the ground that must occur in order for the ground pressure to decrease to a chosen, structurally manageable value.

One fundamental problem of this approach is that all plane strain solutions (whether closed-form solutions for the ground response curve or numerical simulations involving a partial stress release before lining installation) assume that the radial stress at the excavation bound-ary decreases monotonically from the initial value (which prevails far ahead of the face) to the support pressure (which develops far behind the face). However, the actual load history in-cludes a complete unloading of the excavation boundary in the radial direction over the un-supported span and a subsequent re-loading of the tunnel boundary starting with the installa-tion of the lining. Cantieni and Anagnostou (2009a) have shown that the assumption of a mo-

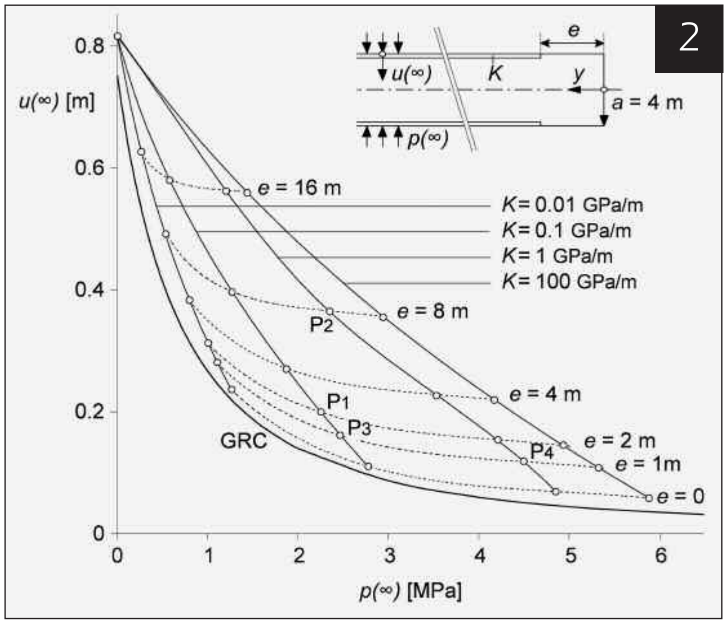
notonically decreasing radial stress may lead (particularly under heavily squeezing condi-tions and assuming an elastoplastic ground behaviour) to a more or less seri-ous underestima-tion of ground pressure and deformation.
The actual final equilibrium points – i.e., the tunnel wall displacement u(∞) and the ground pressure p(∞) prevailing far behind the face – are consistently located above the ground response curve.

Figure 2 shows the ground response curve obtained by a closed-form, plane strain solution (the solid line marked by "GRC"), as well as the results of axially-symmetric numerical cal-culations for the case of heavily squeezing ground (the points marked by circles, e.g. P_1, P_2, …). The mechanical behaviour of the ground was modelled as isotropic, linearly elastic and perfectly plastic according to the Mohr-Coulomb yield criterion, while the lining was taken as an elastic radial support with stiffness dp/du = k. All of the model parameters can be found in Table 1.

As illustrated by Figure 2, plane strain analysis systematically underestimates ground pres-sures and deformations. The deviation is due to the inability of any plane strain model to map, (i), the complete radial unloading of the excavation boundary over the unsupported span and, (ii), the subsequent increase in radial stress following the installation of the lining.

__2.__ GROUND RESPONSE CURVE UNDER PLANE STRAIN CONDITIONS (GRC) AND GROUND RESPONSE POINTS (P_1, P_2, …) FAR BEHIND THE TUNNEL FACE FOR DIFFERENT UNSUPPORTED SPANS E AND LINING STIFFNESSES K

AFTER CANTIENI AND ANAGNOSTOU, 2009A



(i) Let us consider first the effect of an unsupported span e for a fixed value of lining stiffness k. Points P_2 and P_4 in Figure 2 involve a stiff lining (k = 1 GPa/m) installed at e = 1 or 8 m, respectively.
The deviation from the ground response curve is larger in the case of a longer unsupported span. This is because the extent of the plastic zone and the magnitude of deformation are governed by the biaxial stress state (i.e. a radial stress equal to 0) prevailing over the long unsupported portion of the tunnel, rather than by the final stress state.

(ii) The effects of lining stiffness can be observed in points P_3 and P_4. The ground response point P_3 that results from the axisymmetric calculation for a soft support (k = 0.1 GPa/m, installed at e = 1 m) is closer to the plane strain ground response curve (GRC) than the ground response point P_4 that applies to a stiff support (k = 1 GPa/m, installed also at e

__TABLE 1.__ ASSUMED MODEL PARAMETERS (FOR THE EXAMPLES OF FIGURES 2 AND 15)

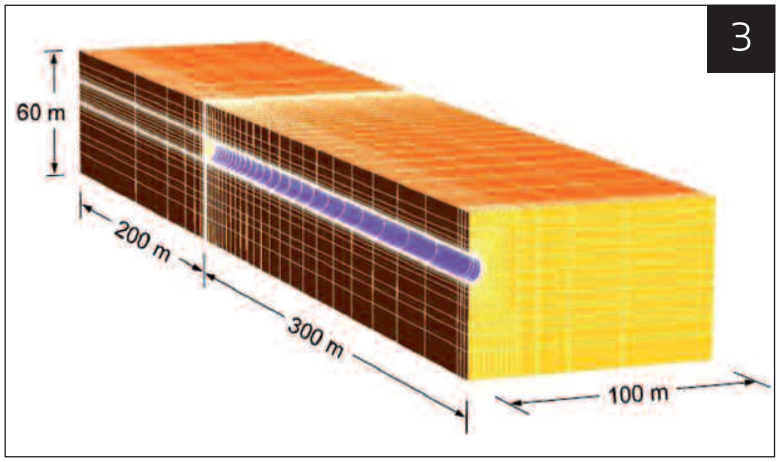
Initial stress	p_0	12.50 MPa
Tunnel radius	a	4 m
Young's Modulus (Ground)	E_R	1000 MPa
Poisson's ratio (Ground)	ν	0.30 -
Cohesion (Ground)	c	500 kPa
Angle of internal friction (Ground)	φ	25 °
Dilatancy angle (Ground)	ψ	5 °

= 1 m). In general, the stiffer the lining (the value of the unsupported span e being fixed), the larger will be the final ground pressure and thus, because of the more distinct stress reversal, the larger will be the deviation from the ground response curve. The net deviation is, of course, governed both by effects (i) and (ii). This is particularly apparent when considering long unsupported spans. The error introduced by the plane strain assumption decreases again at very large values of e (see, e.g., solid line for k = 1 GPa/m in Figure 2).

In conclusion, the actual relation between ground pressure and deformation is not unique. This conflicts with the plane strain results and suggests that a plane strain analysis cannot reproduce correctly at one and the same time both the deformations and the pressures. This is particularly relevant not only from the theoretical point of view but also with respect to practical design issues such as the assessment of a TBM-drive in squeezing ground (Section 5) or the design of a yielding support (Section 6). In both cases the tunnel engineer needs a reliable estimate of the ground deformations (in order to determine the amount of overcut or over-excavation) and of the ground pressures (in order to determine the required thrust force or the dimensions of the lining).

4. SETTLEMENT DEVELOPMENT IN SHALLOW SOFT GROUND TUNNELLING

Field measurements show clearly that settlements induced by tunnelling through low-permeability clay deposits may increase for several months after excavation. The settlement trough widens and deepens over the course of time (O'Reilly et al., 1991). As explained in Section 2, the advance rate and the ground permeability greatly influence the development of ground pressure and deformation when tunnelling through low permeability ground (e.g. clay deposits). Depending on the ratio of advance rate v to ground permeability k, the ground response will be undrained (favourable), drained (less favourable) or somewhere in between. As a rule, for a given low ground permeability k, a higher as possible advance rate v is advantageous. This Section investigates the opposed effects of consolidation and advance rate by means of numerical calculations for a cylindrical shallow tunnel crossing homogeneous ground (Figure 3). The advancing tunnel heading is taken into account as outlined in Section 2. Thanks to the economy and numerical stability of the method outlined in Section 2, it was possible to perform a comprehensive parametric study, clarifying the effect of advance rate on the transversal and longitudinal settlement troughs.



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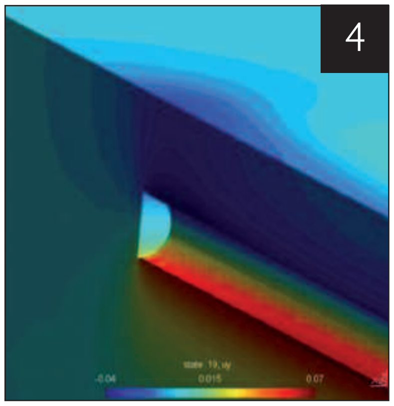
3. COMPUTATIONAL MODEL FOR A SHALLOW TUNNEL

In the numerical examples, both the ground and the lining have been modelled as linearly elastic materials. Table 2 summarises the material constants and the other model parameters. The tunnel heading and the unsupported span of 1 m were modelled as seepage faces under atmospheric pressure. We assume, furthermore, that the groundwater recharge from the surface (e.g. through rainfall or an adjacent river or lake) is sufficient to maintain the elevation of the water table at a constant level. The draining action of the tunnel then results in decreasing pore water pressures in the ground surrounding the tunnel. The decrease in the pore pressures may be temporary or permanent, depending on the permeability of the lining. We have chosen to consider the case of a practically impervious lining (i.e. a no-flow boundary condition will apply to the supported section of the tunnel boundary).

Figure 4 shows the contour lines of vertical displacement u_y for $v/k \rightarrow 0$ (i.e., either during excavation in a high permeability ground or at the steady state achieved during a long excavation standstill). A crater-like depression of the surface can be observed above the tunnel face (Figure 4): the soil surface experiences firstly a settlement as the tunnel face approaches ("Phase 1") and subsequently a heave ("Phase 2"). The Phase 1 settlement is caused by spatial stress redistribution and by the decreasing pore pressures (consolidation) in the vicinity of the heading. After the installation of the impervious lining, the pore pressures increase gradually and reach their natural values again at a certain distance behind the face. Consequently, the effective stresses decrease ("unloading", "swelling"). The simplified assumption of linear elastic behaviour (with the same levels of stiffness for loading and unloading) overestimates the swelling strains, with the result that the Phase 2 heave compensates for a considerable portion the Phase 1 settlement. With a more realistic material model (which takes account of the stiffer ground response to unloading), this effect would be less pronounced and the final settlement would be governed by the deformations around the tunnel heading.



4. *CONTOUR LINES OF VERTICAL DISPLACEMENT U_Y AT A STEADY STATE ($V/K \rightarrow 0$)*

Figure 5 provides a more complete picture of the surface settlement for different advance rates v, while Figure 6 shows the effect of advance rate v and of permeability k on the longitudinal settlement trough. According to Figure 6, the conditions in the vicinity of the heading are practically undrained for $v/k > 10^5$ and practically drained for $v/k < 10^3$. Depending on the ratio v/k, the ground response lies somewhere between the undrained response and the drained response. The higher this ratio, the

TABLE 2. *PARAMETER VALUES FOR THE NUMERICAL EXAMPLE*

Tunnel radius	R	5 m
Depth of cover (from tunnel axis)	H	20 m
Elevation of water table (from tunnel axis)	H_w	20 m
Advance rate	v	variable m/s
Coefficient of horizontal initial stress	K	0.50 -
Young's Modulus (ground)	E	50 MPa
Poisson's ratio (ground)	ν	0.15 -
Permeability coefficient	k	variable m/s
Total unit weight (ground)	γ	20 kN/m^3
Porosity	n	0.10 -
Unit weight of water	γ_w	10 kN/m^3
Compressibility of water	c_w	0.40 GPa^{-1}
Distance between lining and face	e	1 m
Lining thickness	d	0.20 m
Young's Modulus (lining)	E_L	15 GPa
Poisson's Number (lining)	ν_L	0.15 -

5. *CONTOUR LINES OF SURFACE SETTLEMENT FOR DIFFERENT VALUES OF THE ADVANCE RATE V*

(PERMEABILITY K = 10^{-8} M/S; EXCAVATION DIRECTION FROM RIGHT TO LEFT)

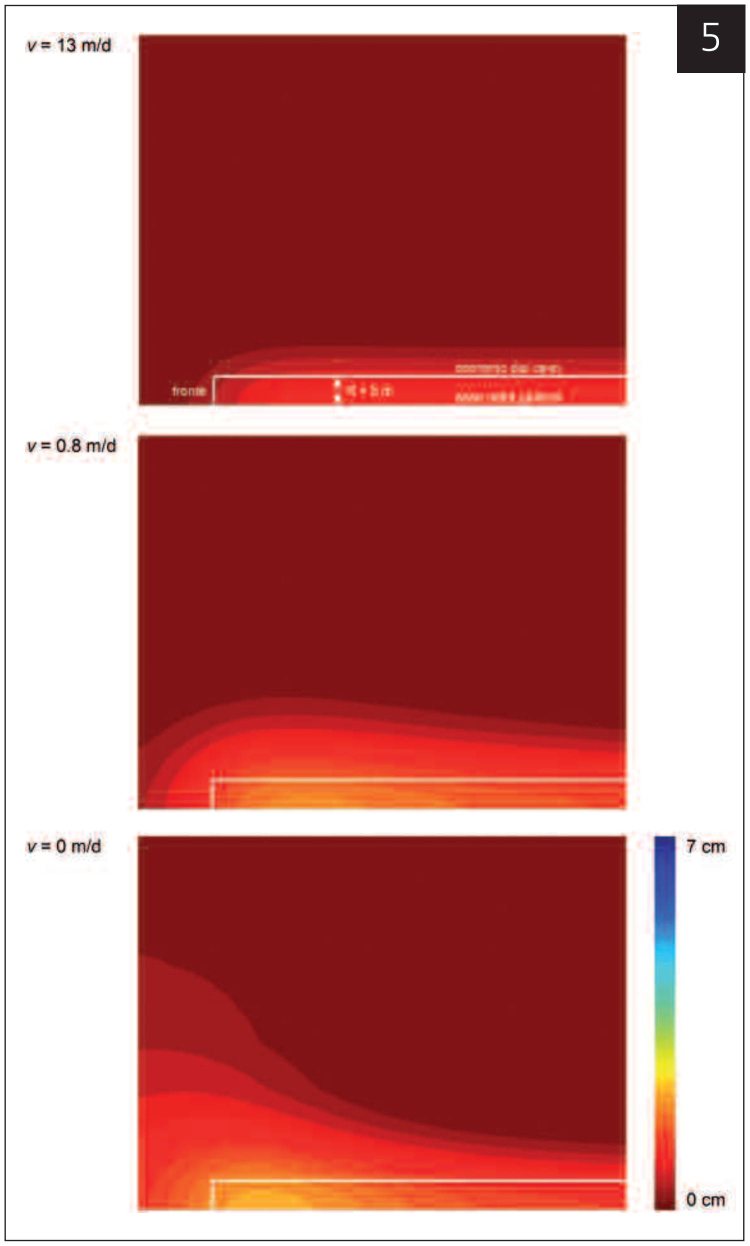
6. *LONGITUDINAL SETTLEMENT TROUGH FOR DIFFERENT VALUES OF THE RATIO OF ADVANCE RATE V TO PERMEABILITY K*

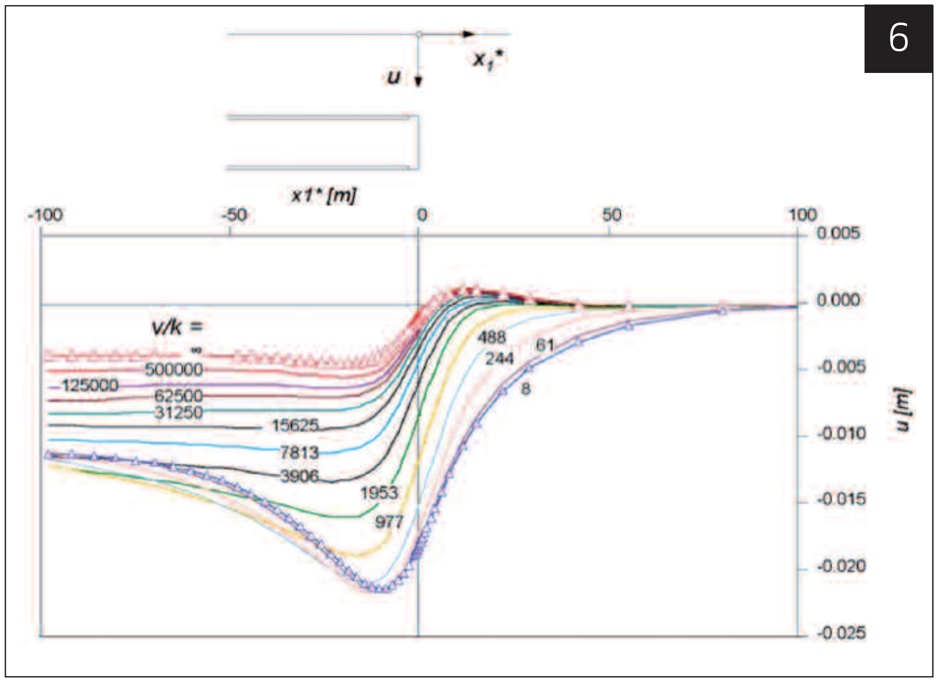
greater will be the distance from the advancing heading at which the deformations reach a steady state. In fact, the crater-like depression at the surface will occur only at high permeability values or, everything else being equal, at low advance rates.

High v/k-ratios reduce the time available for the consolidation process in the vicinity of the advancing face to have a positive influence on the settlement trough in the vicinity of the face (the higher the excavation rate, the lower the angular distortion at the surface) and to contribute to settlement limitation. This is particularly true for closed shield tunnelling as the defor-mations of the ground ahead of the face and around the shield (gap closure) represent by far the greatest sources of volume loss and surface settlement.

5. TBM TUNNELLING IN SQUEEZING GROUNDE

Squeezing ground may slow down or even obstruct TBM operation. Due to the geometrical constraints of the equipment, even relatively small convergences of one or two decimetres may lead to considerable difficulties with the machine (e.g., sticking of the cutter head, jamming of the shield, Figure 7) or in the back-up area (e.g., jamming of the back-up equipment, inadmissible convergences of the bored profile, damage to the tunnel support).





If occurring over frequent tunnel intervals or if persisting over longer portions of a tunnel these difficulties may undermine the economic viability and the feasibility of a TBM drive. In this context, a number of very difficult situations (including the complete loss of a TBM) have occurred in the past (Ramoni and Anagnostou, 2006, 2009a).

TBM performance is the result of a complex interaction between the ground, the tunnelling equipment and the support. The advance rate represents not only the "result" of this interaction, but at the same time influences the interaction too. Identifying the relevant interfaces between the main system components and understanding their interactions is essential to the assessment of critical situations (Ramoni and Anagnostou, 2009a). Moreover, as the TBM types are different with respect to the thrusting system, the type of support and the presence or absence of a shield, different hazard scenarios have to be considered depending on the ma-chine type (the advance rate, which plays an important role, also depends on the machine type).

In this respect, numerical analyses provide a valuable contribution to decision-making in the design process, as they indicate the magnitude of the key parameters. The application of axially symmetric or fully three-dimensional numerical models eliminates the uncertainties associated with the inherently three-dimensional nature of the problem under consideration. For example, such models allow the magnitude and the distribution of the ground pressure acting upon the shield and the lining to be determined, taking into account the support variation alongside the tunnel (shield, segments or otherwise) and the existence of a gap between the shield and the ground. Furthermore, as they take due account of the three-dimensional stress redistribution in the vicinity of the advancing face, they eliminate the errors introduced by as-suming plane strain conditions (Section 3).

7. *PHOTOGRAPH OF THE GROUND AND THE SHIELD TAKEN DURING OPERATIONS TO FREE A SINGLE SHIELDED TBM*

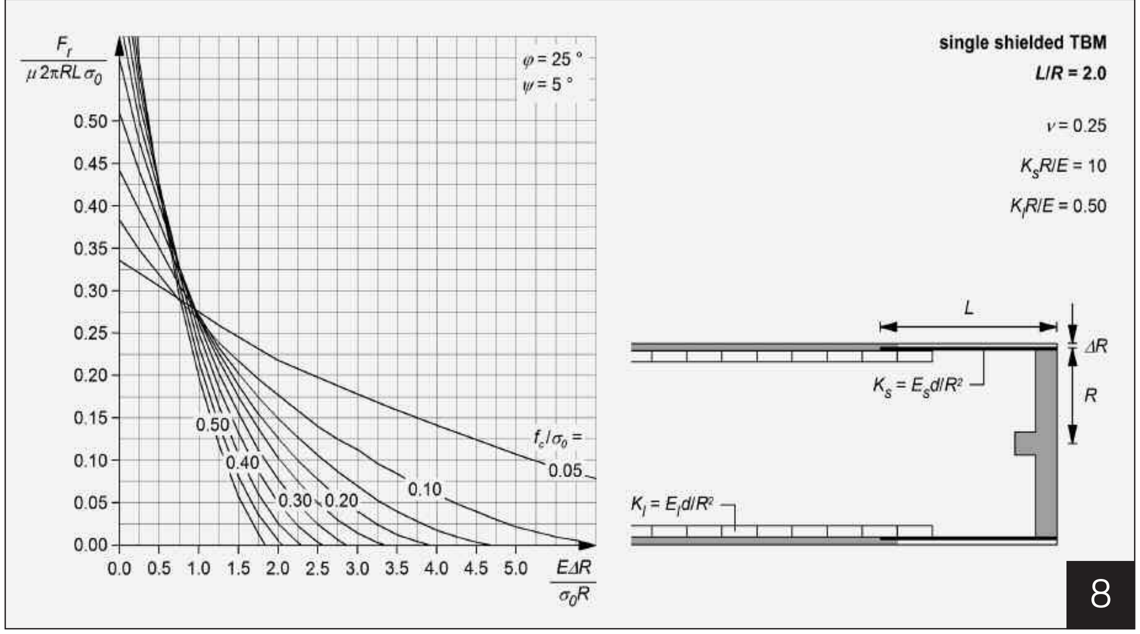
COURTESY OF W. BURGER, HERRENKNECHT AG

Concerning the risk of shield jamming, it is essential to have information on frictional forces when designing a new TBM and when assessing the feasibility of a proposed TBM drive. In the case of a second-hand TBM, checks also have to be made as to whether the installed thrust force is sufficiently high or whether the TBM has to be refurbished. Ramoni and Anagnostou (2009b) provided a number of theory-based decision aids which support rapid, initial assessments of thrust force requirements. A comprehensive parametric study has also been carried out using the finite element method and, based on the numerical results, dimensionless design nomograms have been worked out that cover the relevant range of material constants, in situ stress and TBM characteristics. This is the first time that such a systematic and thorough investigation of the combined effects of the parameters governing shield loading has been attempted. It should be noted that the parametric study involved about 12,000 numerical simulations and was possible only after developing efficient numerical solution methods (in terms of computer time and stability) specifically for this purpose (Section 2).

A total of 45 dimensionless nomograms were developed (see Figure 8 for an example). Each nomogram applies to a different TBM type and normalized shield length L/R, and to a different value of the angle of internal friction φ and includes a band of curves (each curve corresponding to another value of the normalized uniaxial compressive strength f_c/σ_0) showing the normalized required thrust force F_r as a function of the dimensionless product of E/σ_0 by $\Delta R/R$, where E, σ_0, ΔR and R denote the Young's modulus, the initial stress, the tunnel radius and the size of the gap between the bored profile and the extrados of the shield, respectively.

The nomograms make it possible to assess the feasibility of a TBM drive in a given geotechnical situation, to perform rapid sensitivity analyses with respect to the ground parameters and to evaluate potential design measures or operational measures such as reductions in shield length, the installation of a higher thrust force, increases in overcut or the lubrication of the shield surface, thus making a valuable contribution to the decision-making process.

Tunnelling practice has shown that interruptions in the TBM drive may be unfavourable in squeezing ground. In several cases, the TBM has become jammed only when there was a slowdown or standstill in the TBM drive, which suggests that maintaining a high advance rate and reducing standstill times may have a positive effect. Maintaining a high advance rate is of course a major goal for any TBM drive. Nevertheless, high advance rates should not be seen as a panacea for coping with squeez-



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8. *NOMOGRAM FOR DETERMINING THE REQUIRED THRUST FORCE F_R FOR OVERCOMING THE SHIELD SKIN FRICTION OF A SINGLE SHIELDED TBM*

RAMONI AND ANAGNOSTOU, 2010B

ing. Firstly, they are difficult to achieve (especially in the case of poor quality ground). Secondly, ground deformations may develop very rapidly and very close to the working face (Ramoni and Anagnostou, 2009a). In such a situation, the advance rate would play a secondary role (the TBM would become jammed even if operated at the highest feasible speed). Thirdly, standstills in TBM operation cannot be completely avoided.

In the remainder of this Section we would like to discuss in quantitative terms the positive effects of a high advance rate and of short standstills during TBM tunnelling in consolidating, squeezing ground by means of mechanically-hydraulically coupled numerical calculations. Consider, as an example, a 500 m deep, Ø 10 m tunnel driven by a 10 m long single shielded TBM in weak ground at a depth of 100 m beneath the water table (the model parameters are summarised in Table 3).

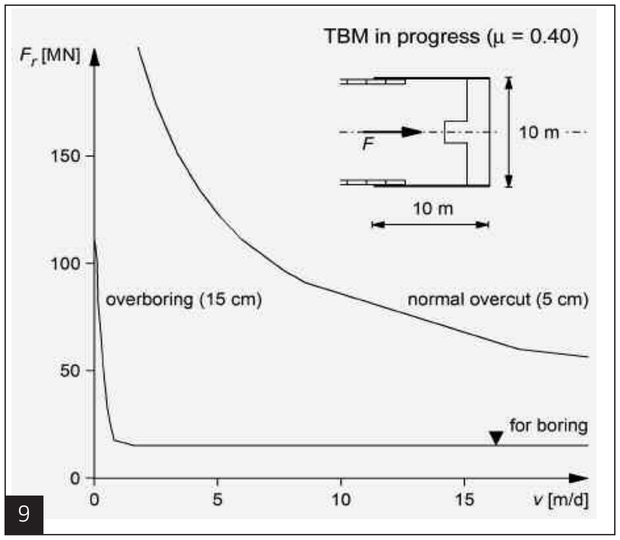
Figure 9 shows the thrust force F_r required to overcome shield skin friction as a function of the advance rate v in a low-permeability ground ($k = 10^{-9}$ m/s) during TBM boring. The favourable effects of a high advance rate and of overboring can be recognised immediately.
Figure 10 shows how the thrust force F_r required in order to restart excavation after a stand-still increases with the length of the standstill. During continuous excavation the machine has to overcome sliding friction, while directly after a TBM-stop (t = 0) static friction becomes relevant. A higher skin-friction coefficient μ was therefore considered.

The curves of Figures 9 and 10 have been calculated by integrating the ground pressure p over the shield length L. The ground pressure p acting upon the shield and the lining at different times t (i.e. the time that has elapsed since the standstill began, based on the assumption that the preceding rate of ex-

***9*. REQUIRED THRUST FORCE F_R AS A FUNCTION OF THE ADVANCE RATE V DURING CONTINUOUS EXCAVATION**
NUMERICAL RESULTS AFTER RAMONI AND ANAGNOSTOU, 2007A

***10.* REQUIRED THRUST FORCE F_R FOR RESTARTING OPERATIONS AFTER A STANDSTILL OF LENGTH T**
NUMERICAL RESULTS AFTER RAMONI AND ANAGNOSTOU, 2007B

***TABLE 3.* ASSUMED MODEL PARAMETERS FOR THE EXAMPLES IN FIGURES 9 TO 12**



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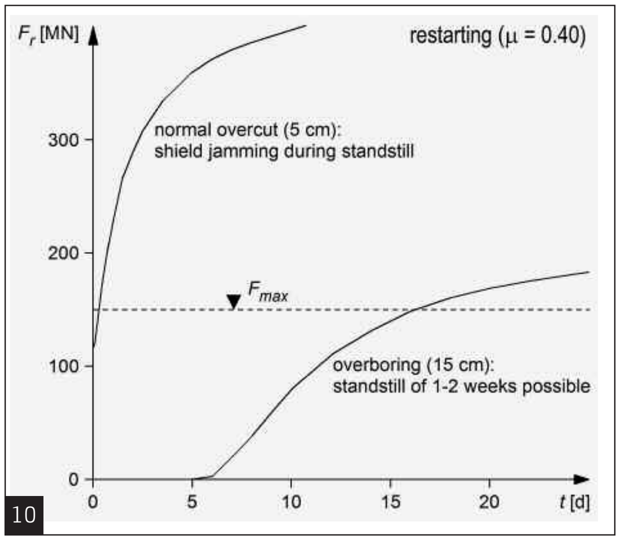
Initial stress (overburden 500 m)	σ_0	12.50 MPa
Initial pore pressure (water depth 100 m)	$p_{w,0}$	1 MPa
Tunnel radius	R	5 m
Permeability	k	10^{-9} m/s
Young's Modulus (ground)	E	1000 MPa
Poisson's ratio (ground)	ν	0.25 -
Cohesion (ground)	c	500 kPa
Angle of internal friction (ground)	φ	25 °
Dilatancy angle (ground)	ψ	5 °
Coefficient of friction (static)	μ	0.40 -
Coefficient of friction (dynamic)	μ	0.25 -



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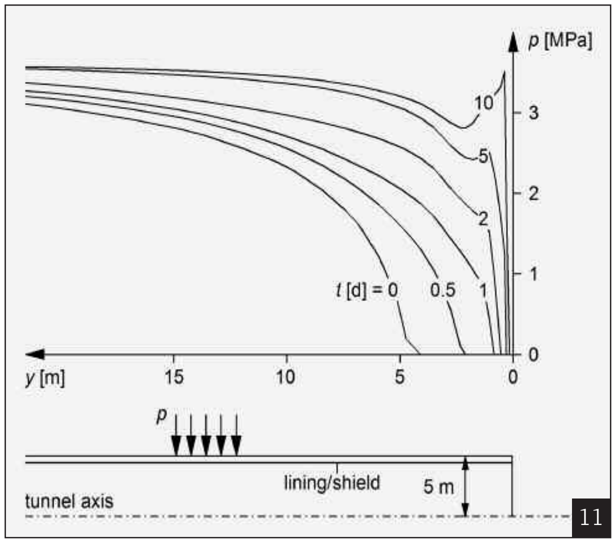
cavation was v =10 m/d) is shown in Figure 11. In this numerical example, the face was regarded as being unsupported. The core yields and extrudes freely (Figure 12) with the outcome that the radial stress ahead of the face decreases, additional load is transferred to the shield via arching in the longitudinal direction and, therefore, a pressure peak occurs close to the face (Figure 11). These results indicate that the behaviour of the core ahead of the face may also be important with respect to the loading of a TBM.

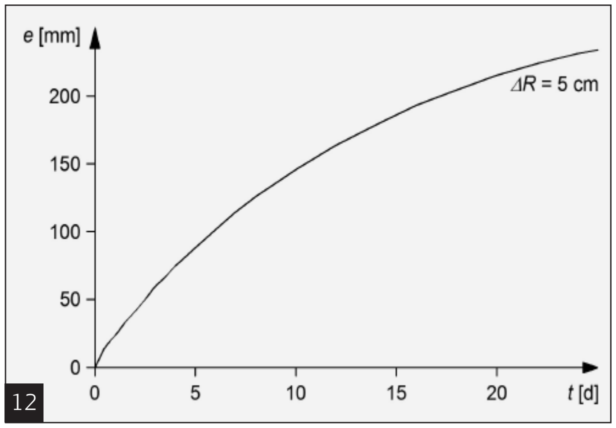
6. YIELDING SUPPORT IN SQUEEZING GROUND

Tunnelling through weak rocks under a high overburden can generate large deformations. This so-called "squeezing" can destroy the lining if an attempt is made to hold back the deformations by installing a rigid lining close to the face. The only feasible solution in heavily squeezing ground is a tunnel support that is able to deform without becoming damaged, in combination with a certain amount of over-excavation in order to accommodate the deformations (the so-called "yielding principle", Kovári, 1998). The so-called "yielding supports" are characterized by two main design parameters: the amount of over-excavation u_y and the yield pressure p_y. Figure 13 shows two examples of structural detailing.



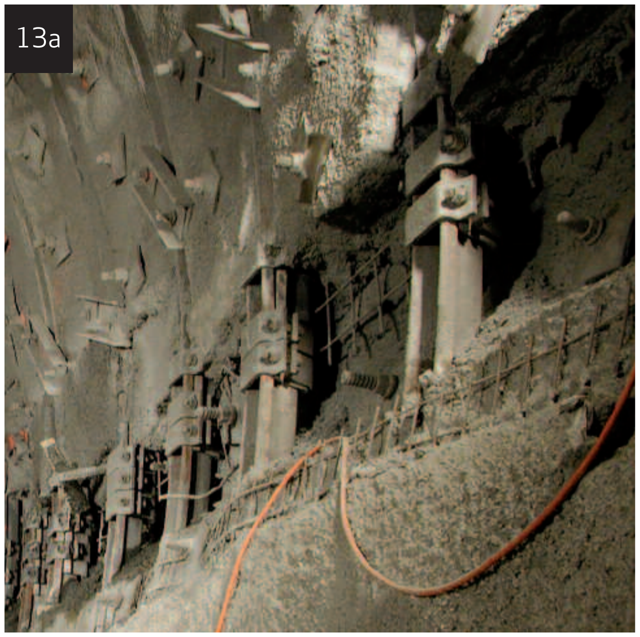
11. RADIAL PRESSURE P ACTING UPON THE SHIELD AND THE LINING AT DIFFERENT TIME POINTS T AFTER THE START OF A STANDSTILL

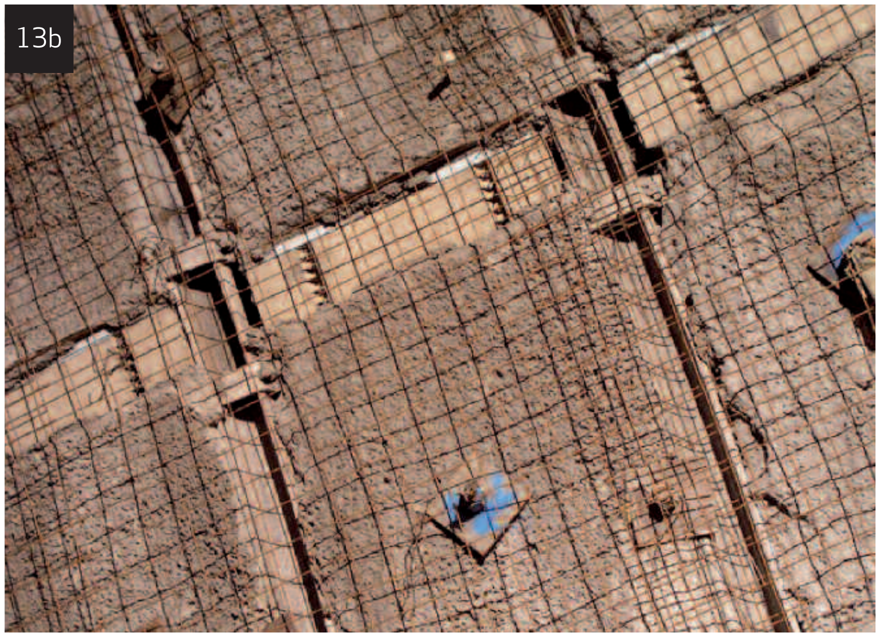
NORMAL OVERCUT OF 5 CM; NUMERICAL RESULTS AFTER RAMONI AND ANAGNOSTOU, 2007B



12. CORE EXTRUSION E OVER TIME T DURING A STANDSTILL

NORMAL OVERCUT OF 5 CM; NUMERICAL RESULTS AFTER RAMONI AND ANAGNOSTOU, 2007B



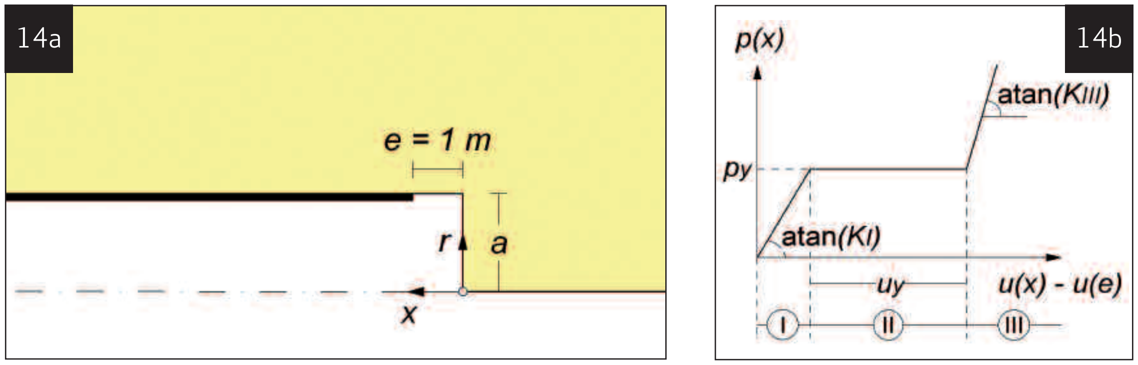


13. TUNNEL SUPPORT YIELDING, AT A LOW ROCK PRESSURE
(A) TH-STEEL SETS WITH SLIDING CONNECTIONS
(B) HIGH ROCK PRESSURE

HIDCON® ELEMENTS, SOLEXPERTS, 2007

The conceptual idea underlying all yielding supports is that the ground pressure will decrease if the ground is allowed to deform. Although it is beyond doubt that the squeezing pressure does decrease with deformation, the relationship between pressure and deformation is not unique, but depends on the support characteristics and installation point (Section 3).
The ground response curve (which describes the relationship between pressure and deformation assuming plane strain conditions and a monotonous reduction in pressure at the excavation boundary) represents the lower limit of actual pressures and deformations (Figure 2).
The reason for the deviation of the actual equilibrium points from the ground response curve is the complete radial unloading of the excavation boundary over the unsupported span and the subsequent increase in radial stress following the installation of the lining. The more pronounced this stress reversal, the bigger will be the deviation from the ground response curve.

This relationship has an important practical consequence for the design of yielding supports, because the magnitude of the stress reversal depends on the yield pressure p_y of the support. More specifically, the lower the yield pressure p_y, the more pronounced will be the stress reversal, the bigger will be the deviation of the final equilibrium point from the ground response curve and – the value of over-excavation u_y being fixed – the higher will be the final pressure.
Let us investigate the effect of the yield pressure quantitatively by considering the axisymmetric model of a 500 m deep, cylindrical tunnel in weak rock (Figure 14a). The mechanical behavior of the ground is modeled as isotropic, linearly elastic and perfectly plastic according to the Mohr-Coulomb yield criterion (see Table 1 for the material constants).
The initial stress field is taken to be uniform and hydrostatic with a pressure p_0 of 12.5 MPa. The yielding support is modeled as a radial support having a deformation-dependent stiffness (Figure 14b): Phase I is governed by the stiffness kI of the system up to the onset of yielding; in Phase II the support system deforms under a constant pressure py; when the amount of over-excavation uy is used-up, the system is made practically rigid (stiffness kIII), e.g. by applying shotcrete, with the consequence that an additional pressure builds up upon the lining (Phase III).



14. *(A) MODEL*
(B) CHARACTERISTIC LINE OF A YIELDING SUPPORT

Figure 15 shows the development of the radial pressure on the lining for four yielding supports that allow for a deformation u_y of 15 cm but yield at different pressures p_y (Table 4). For the reasons explained above, the final lining pressure decreases with the yield pressure of the support.

Figure 16 shows the equilibrium points – u(∞), p(∞) – of the ground as well as (for the purposes of comparison) the ground response curve under plane strain conditions (solid line GRC). Let us consider next how the equilibrium point changes in relation to the yield pressure p_y (in cases O, A, B and C) starting with a support that can endure

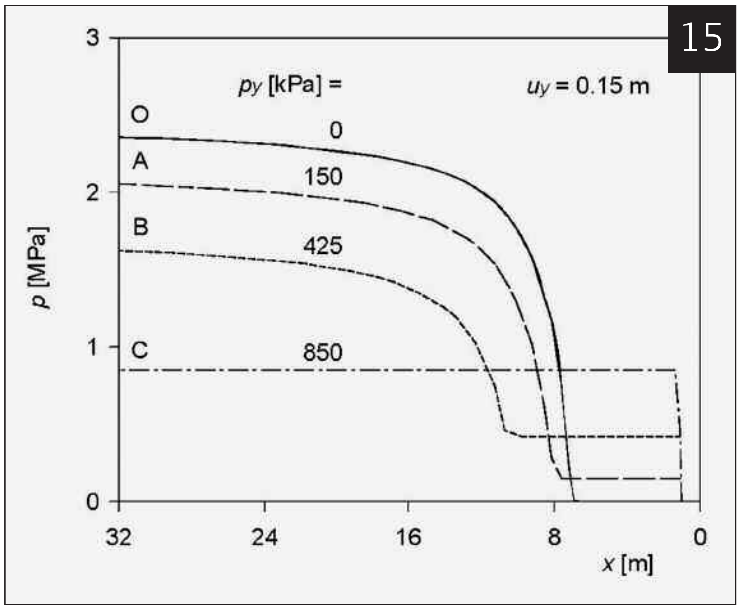
15. GROUND PRESSURE P VS. DISTANCE X BEHIND THE FACE FOR DIFFERENT VALUES OF THE YIELD PRESSURE P_Y
(SUPPORT CASES O, A, B AND C ACCORDING TO TABLE 4)
(NUMERICAL RESULTS AFTER CANTIENI AND ANAGNOSTOU, 2009B)

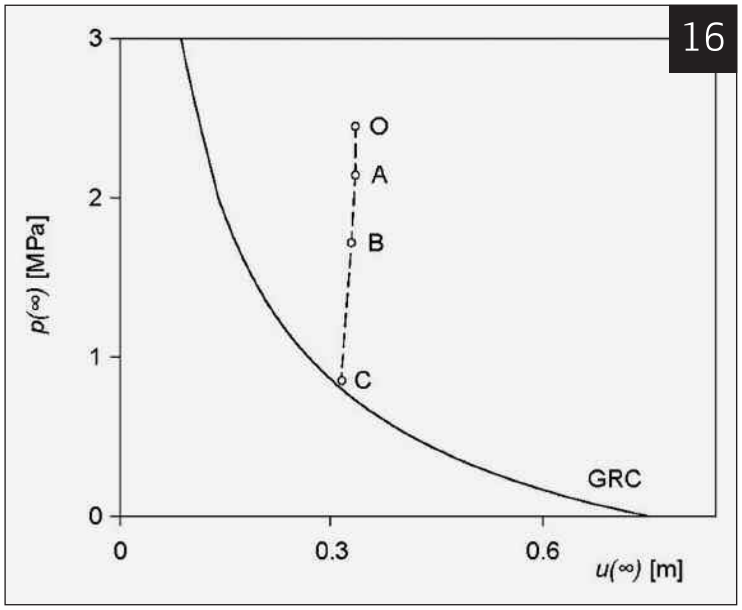
16. GROUND RESPONSE CURVE UNDER PLANE STRAIN CONDITIONS AND THE FINAL EQUILIBRIUM POINTS FOR THE SUPPORTS ANALYSED
(NUMERICAL RESULTS AFTER CANTIENI AND ANAGNOSTOU, 2009B)

TABLE 4. SUPPORT PARAMETERS
SEE ALSO FIGURE 14B

Case	Description	K_I (1) [MPa/m]	p_y [kPa]	u_y [cm]	K_{III} [MPa/m]
O	35 cm thick shotcrete lining with open longitudinal slots (E_L = 30 GPa (2))	n/a	0	15	656
A	Steel sets TH-44 (Figure 13a) spaced at 1 m with sliding connections by 4 friction loops each offering a resistance of 150 kN (3)	100	150	15	656
B	Like A, but additionally 20 cm shotcrete with highly deformable concrete elements inserted into the slots (yield stress 7 MPa, cf. Figure 13b and Solexperts, 2007)	100	425	15	656
C	Like B, but with higher yield strength elements (17 MPa, Solexperts, 2007)	100	850	15	656

Note:
(1) This parameter was kept constant in the numerical study as it is of subordinate importance. The actual values of k_I of the support systems described in the last columns of the table are 74 MPa/m (case A) and 114 MPa/m (cases B and C). A sensitivity analysis has shown that a variation of k_I in this range does not affect the numerical results.
(2) This value assumes that, by the time the longitudinal slots close, the shotcrete will have developed its final stiffness.
(3) After the yield phase, the support is set practically rigid (d_{III} = 35 cm, E_L = 30 GPa).





a radial displacement of u_y = 15 cm without offering any resistance to the ground (case 0). Due to the long unsupported span and the pronounced stress reversal, the equilibrium point for this case is located far above the plane strain response curve (Figure 16). At higher yield pressures, the stress reversal is less pronounced and, consequently, the equilibrium point is located closer to the ground response curve. This means – since the ground deformation is governed by the over-excavation u_y and is therefore approximately constant – that the final pressure decreases (points A, B and C in Figure 16).

It is remarkable that a similar reduction in the final ground pressure can be achieved not only by installing a support that is able to accommodate a larger deformation (which is a well-known principle), but also by selecting a support that yields at a higher pressure. A more detailed discussion of the interaction between yielding supports and squeezing ground (along with design nomograms that banish the shortcomings of plane strain analyses and enable rapid assessments of support requirements) can be found in Cantieni and Anagnostou (2009b).

7. CONCLUSIONS

Spatial considerations are important for a number of practical design and analysis issues arising in tunnel engineering. It is almost trivial – or even a tautology – to say that the prediction uncertainties of planar models can be reduced by taking account of the third dimension. It is more interesting to note that if we take adequate account of the stress history of the ground and of the sequence of excavation and support installation we may obtain results which are qualitatively different to those obtained through plane strain analyses, i.e. results which are surprising at first glance and cannot be reproduced from two-dimensional models.

BIBLIOGRAPHY

ANAGNOSTOU, G., 1993. Modelling seepage flow during tunnel excavation. Safety and environmental issues in rock engineering, EUROCK '93, Lisbon, Vol. 1, 3–10.

ANAGNOSTOU, G., 2007. Continuous tunnel excavation in a poro-elastoplastic medium. Tenth international symposium on numerical models in geomechanics, NUMOG X, Rhodes, 183–188.

CANTIENI, L., ANAGNOSTOU, G., 2009A. The effect of the stress path on squeezing behavior in tunnelling. Rock Mechanics and Rock Engineering 42 (2), 289–318.

CANTIENI, L., ANAGNOSTOU, G., 2009B. The interaction between yielding supports and squeezing ground. Tunnelling and Underground Space Technology 24 (3), 309–322.

CORBETTA, F., 1990. Nouvelles méthodes d'étude des tunnels profonds – calculs analytiques et numériques. PhD thesis, École des Mines de Paris.

FRANZIUS, J.N., POTTS, D.M., 2005. Influence of mesh geometry on three-dimensional finite-element analysis of tunnel excavation. International Journal of Geomechanics 5 (3), 256–266.

GHABOUSSI, J., GIODA, G., 1977. On the time-dependent effects in advancing tunnels. International Journal for Numerical and Analytical Methods in Geomechanics 1 (3), 249–269.

KOVÁRI, K., 1999. Scavo di gallerie in rocce spingenti. Quarry & Construction 36 (3), 59–72.

NGUYEN-QUOC, S., RAHIMIAN, M., 1981. Mouvement permanent d'une fissure en milieu élastoplastique. Journal de Mécanique Appliquée 5 (1), 95–120.

O'REILLY, M.P., MAIR, R.J., ALDERMAN, G.H., 1991. Long-term settlements over tunnels: an eleven-year study at Grimsby. Tunnelling '91, London, 55–64.

PANET, M., GUENOT, A., 1982. Analysis of convergence behind the face of a tunnel. Tunnelling '82, London, 197–204.

RAMONI, M., ANAGNOSTOU, G., 2007A. The effect of advance rate on shield loading in squeezing ground. Underground space – the 4th dimension of metropolises, ITA World Tunnel Congress 2007, Prague, Vol. 1, 673–677.

RAMONI, M., ANAGNOSTOU, G., 2007B. Numerical analysis of the development of squeezing pressure during TBM standstills. The second half century of rock mechanics, 11th Congress of the International Society for Rock Mechanics (ISRM), Lisbon, Vol. 2, 963–966.

RAMONI, M., ANAGNOSTOU, G., 2010A. Tunnel boring machines under squeezing conditions. Tunnelling and Underground Space Technology 25 (2), 139–157.

RAMONI, M., ANAGNOSTOU, G., 2010B. Thrust force requirements for TBMs in squeezing ground. Tunnelling and Underground Space Technology 25 (4), 433–455.

SOLEXPERTS, 2007. HIDCON-elements in tunnelling. Solexperts AG, Mönchaltorf (Schweiz).

NOTA FINALE
La versione originale del presente contributo è in lingua inglese.

ANAGNOSTOU, G., CANTIENI, L., RAMONI, M., 2009. About the advancing face – spatial effects in tunnel engineering. The evolution of design and construction approaches in the field of underground projects, Rocksoil's 30th Anniversary Conference, Milan, 1–20.

Norbert Vogt

SHOTCRETE EXCAVATIONS FOR THE MUNICH SUBWAY

COMPARISON OF DIFFERENT METHODS OF FACE SUPPORT IN SETTLEMENT SENSITIVE AREAS

J. FILLIBECK - N. VOGT ZENTRUM GEOTECHNIK, TECHNISCHE UNIVERSITÄT MÜNCHEN

For the construction of shallow tunnels in settlement-sensitive urban areas it is very important to reduce the settlements and to increase the stability of the tunnel face during the excavation. In the case of shotcrete excavation, the use of different methods of face support has become more and more common. These methods are: ground freezing, pipe roofs, jet grouting and injection support. The paper shows the experience made to due the installation of the above mentioned face supports, especially since the specific focus is related to the arising settlements. If only small deformations are allowed to occur, as the examples show, deformations that have to be considered during the construction process as well as those to establish the bearing load; they could be significant depending on the process. Suggestions have been made as to ways in which the deformations can be reduced by making additional measurements.

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1. INTRODUCTION

For the construction of safe shallow tunnels in settlement-sensitive urban areas, it is very important to reduce the settlements and to increase the stability of the tunnel face during the excavation. The use of different methods of face support in the case of shotcrete excavations is becoming increasingly common. These methods are ground freezing, pipe roofs, jet grouting and injection support.

The report presents the experience gained in the installation of the abovementioned working face supports, with particular focus on the induced settlements. Four different projects of Munich's subway are described. After a short project description the results of the measurements are illustrated (geodetic and borehole measurements) and evaluated. With this background, the different methods of face support are compared and the different advantages and disadvantages are discussed. Finally, special attention is focused on the installation process. Proposals are made for the reduction of settlements in future tunnel projects.

2. GEOLOGICAL AND HYDROGEOLOGICAL CONDITIONS

In the Munich subsoil the quaternary gravels follow under fillings of small thickness. The quaternary gravels can reach a thickness of more than 20 m. They predominantly consist of medium density layers, are laminated and have, depending on the deposit conditions and their age, a differing amount of sand and fine grain. The average permeability amounts to approx. $k = 5 \cdot 10^{-3}$ m /s. Tertiary layers lie below the quaternary gravels. They consist of changing layers of fine- to medium-grained sands with high density and clays or silts in stiff to firm consistency. The thickness of the layers can change excessively within a small distance. The average permeability of the sand amounts approximately from $k = 1 \cdot 10^{-4}$ to $k = 1 \cdot 10^{-5}$, the tertiary clay and silt can for all practical purposes be assumed impermeable.

The quaternary gravels possess a mostly free phreatic water level, which can reach ground level. There are still confined aquifers within the sand layers with fine-grained cover. The pressure of the groundwater approximately corresponds to that of the free phreatic surface in the quaternary gravels.

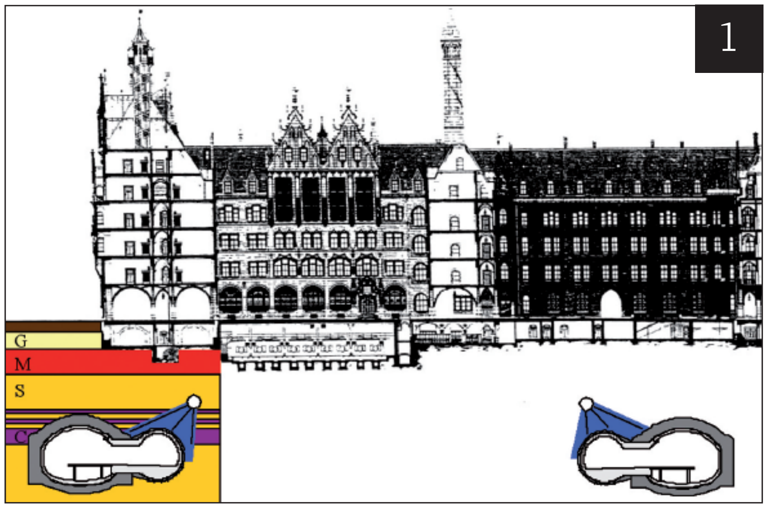
3. HEADING WITH GROUNDFREEZING UNDER THE CITY HALL OF MUNICH

3.1 Construction process

The extention of the station Marienplatz of the subway lines U3 / U6 under the Munich City Hall was built by the company Fa. Max Bögl GmbH & Co KG. The project was finished in 2006.

Parallel to the two existing platforms, two directly joining tunnels were built in shotcrete method under atmospheric conditions with a vertical distance of about 10 m to the city hall. In order to avoid damage to the landmark city hall, the deformations had to be strictly limited.

The construction company Fa. Bögl, planned freezing arches in the context of an alternate bid, in order to support the crown and to keep the retaining water away from the tunnel face. The freezing arches were provided for through pilot galleries above the crown (Figure 1).



1. CROSS SECTION WITH CITY HALL, TUNNELS AND GEOLOGIC SITUATION

2. OPERATING - CONTROL OF THE ARTIFICIAL GROUND FREEZING

The tunnels are embedded in the tertiary layers (figure 1). The water bearing sand layers had to be dewatered with the help of filter wells.

3.2 Measurements to reduce frost heave

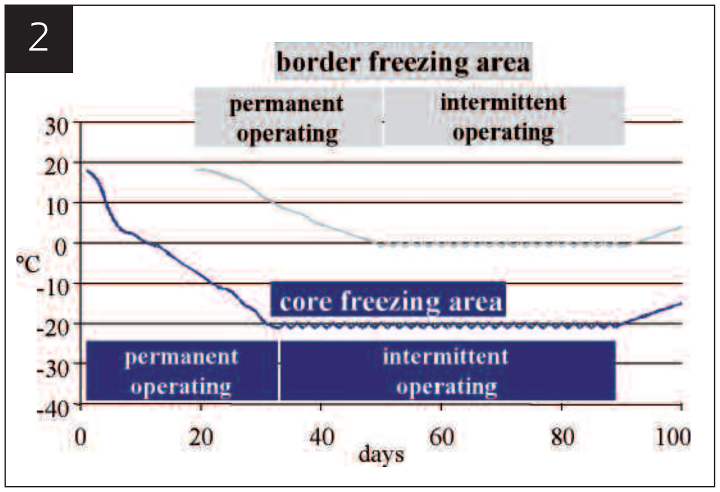
For the successful realization of the specific proposal it was crucial to reduce frost heave in such a way that no damage occur to the city hall. Frost heave can be essentially attributed to two reasons:

- homogenous frost heave (Δh_{vol}) because of a 9 % increase in volume caused by the changeover from water to ice.
- growing of ice lenses with corresponding frost heave (Δh_{icel}) because of the tendency of the soil to draw water near the interface of the frozen to the unfrozen soil (zero-degree-front). This frost heave increases with time.

The frost heave tests, which were performed in the laboratory of the Centre for Geotechnics at the Technical University of Munich (Zentrum Geotechnik, TU München), showed, that in the tertiary fine-grained soils frost heave Δh_{icel} still occurs at load-levels of more than 400 kN/m² if water can be drawn at the interface to the permeable sand layers. Therefore the alternating layers of permeable sands and frost-sensitive clays present a critical risk source when frost heave is considered.

In order to reduce frost heave to a minimum the following measurements had to be taken:

- measuring and controlling the temperature in the soil with the help of 5 measuring cross sections per tunnel. Every cross section includes 18 thermo couples.
- reducing the operation time of the frozen arches by dividing the tunnels into 3 different sections: north, middle and south.
- further partitioning within the freezing sections by the installation of groups of freezing tubes with separate control.

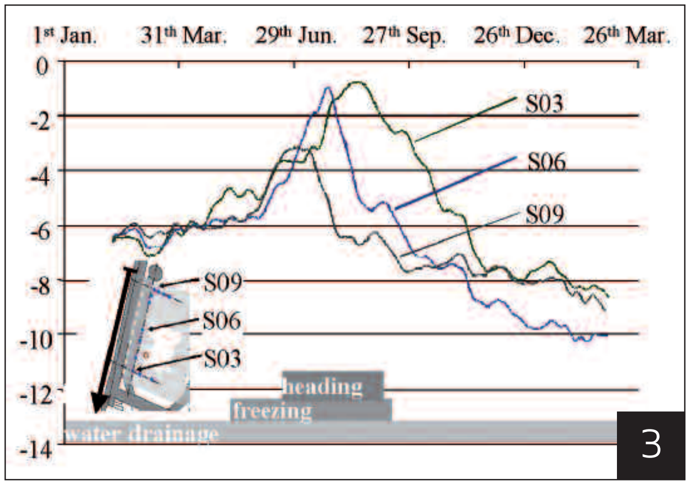


In figure 2 the temperature development of a section is shown schematically. Twenty days after starting the freezing process in the core of the freezing body the freezing of the border area started. After the frozen body reached approx. – 22 °C in the core area and 0 °C in the border area, the freezing tubes were operated intermittently, an average of 8 to 24 hours in the core area and 12 to 24 hours in the border area. Due to intermittent handling, the zero-degree-front does not move outside (enlargement of the frozen body), but stays in a narrow zone, which again and again gets frozen and defrosted. Thus frost heave reduces significantly. The operation of one section could be stopped after approx. 90 days. The adjacent defrosting process took about three months.

3.3 Measuring of the settlements

The deformations which occurred during the construction process were measured by a geodetic precise levelling system on the surface and a closed water levelling system in the 2nd basement of the city hall.

The closed water levelling system consisted of 10 measuring points with a resolution of 1/10 mm. The measuring results could be checked online at all times. Figure 3 shows the location of the measuring points S03, S06 und S09 of the closed water levelling system as well as the development of the settlement and heave depending on the time. The three measurement points were situated in the sections north, middle and south.



3. VERTICAL DISPLACEMENTS OF THE MEASURING POINTS S03, S06 AND S09

The settlements at the beginning of the freezing process result from groundwater drawdown. At the onset of freezing the expected frost heave started. It reached a maximum value of 3 to 5 mm.

The settlements due to the tunnelling process occurred after the heading had passed the measuring points and they still continued after the freezing process was stopped. The settlements slowed down continuously and stopped 3 months later with a maximum settlement of about 10 to 12 mm.

Figure 3 clearly shows the temporary displacement of the settlements according to the heading. The drive reached the measuring points in descending order, resulting in them reaching the maximum heave successively.

The measured deformations were approximately the same as the calculated ones. Thereby half of the settlements could be attributed to dewatering measures, which lead to large area settlements and correspondingly low differential settlements. Furthermore, no settlement damages were determined at the city hall, so it can be assumed that the heading was very successful. It was essential that for the success of the project, larger frost heave by ice lenses could be avoided by applying the above mentioned measures. Frost lenses would otherwise have led to a softening of larger soil areas and therefore to larger settlements and settlement differences.

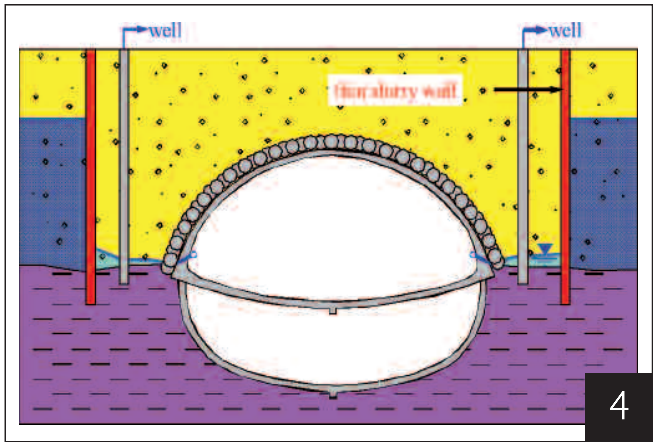
4. JET GROUTING COVER FOR A LARGE CROSS SECTION FOR U3 NORTH LOT 1

4.1 Construction process

The consortium Ed. Züblin AG / Max Bögl GmbH & Co KG carried out the construction of the subway lot U3 North – 1 in the north of Munich. The works were completed in 2006. The shotcrete headings with a total length of around 1950 m were driven with and without compressed air support and with several methods of crown support which will be introduced hereafter.

The headings W3 and W4 with a cross section area of up to 200 m² began from a starting shaft with a top heading. First watertight pits with thin slurry walls were produced (figure 4) in order to lower the groundwater table. The safety of the excavation face was increased by 13 jet grouting covers (total length of about 15.5 m each, overlap 4.3 m) as well as further jet grouting piles in the face of the crown.

The quaternary gravels were cut with suspension (simplex – method) at a pressure of up to 400 bar at the cone. At anytime during the making of a jet grouting pipe a controlled outflow of the suspension is required, assuring that the pressure does not lift the soil. The top heading followed after the installation of the jet grouting took place. The heading of the bench and invert began on finishing all top headings.



4. *HEADING W3 AND W4 WITH JET GROUTING COVER*

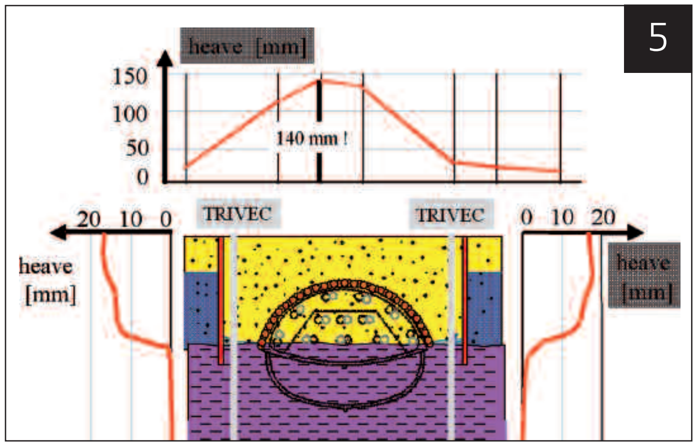
5. *DEFORMATIONS IN CROSS SECTION MQ 8 AFTER CONSTRUCTIONN OF THE JET GROUTING COVER*

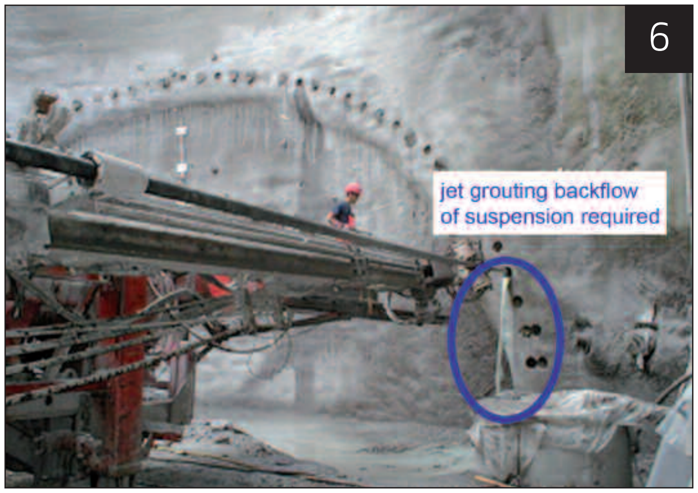
6. *REQUIRED SUSPENSION BACKFLOW DURING JET GROUTING*

4.2 Heave during the installation of the jet grouting cover

Figure 5 shows the deformation in the cross section at a distance of 50 m away from the starting shaft directly after the installation of the jet grouting cover.

The heave above the crown reached in MQ 8 about 140 mm and in total a maximum of 250 mm. At first, the heave was deemed uncritical because there were no buildings close to the tunnel, however they result particularly in soil strains in a narrow band directly above the interface to the tertiary soils. Because of the proximity of the thin slurry wall to the jet grouting cover, the heave led to a crack in the thin slurry wall, making the wall permeable.





The heave results from the fact, that the outflow of the suspension in the annular space of the jet grouting piles, which are faced upwards, can not be controlled in a suitable way (figure 6). Owing to the lack of backflow in the layered soil with strongly differing conductivity the overpressure spread over a greater area, resulting in a rising of the soil above the jet grouting cover.

The large heave could only be limited by reducing the overpressure through the installation of further boreholes from the ground level, resulting in high costs.

With increasing soil cover the heave reduced because of the increasing load. However, even under more than 12 m of soil cover and the installation of the jet grouting cover in tertiary clays, the heave still amounted to approximately 20 mm.

For further projects, where only very few deformations are allowed during the installation of the jet grouting cover, sufficient attention should be paid to the control of the suspension backflow during grouting. This problem

could for example be resolved by improving the technique of the grouting machine or with the help of a double tube, which is pulled a little ahead during jet grouting or likewise with the help of special valves which control the pressure of the backflow.
If the pressure gets to high during grouting, heave can be avoided or at least reduced by additional horizontal or vertical arranged boreholes.

4.3 Settlements during the heading

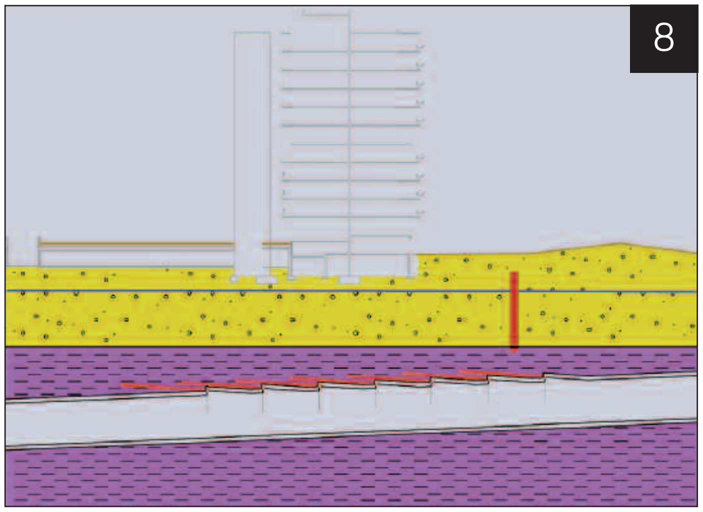
After the installation of the jet grouting cover, the top heading with temporary shotcrete invert followed step by step, over the whole heading distance. After this the heading of the bench and invert followed. The settlements which occurred during the headings amounted to a maximum of 26 mm in cross section MQ 8 and 30 to 40 mm in the area with larger soil cover.
In order to be able to judge the results, in figure 7 the above mentioned measurements are compared with those of atmospheric shotcrete headings having nearly the same soil cover but were driven in partial face advance without jet grouting cover.
Overall maximum settlements were measured as almost the same size, which means that the jet grouting cover does not reduce the settlements, in comparison to tunnels driven in partial face advance. As the sliding micrometer measurements show, the forces which were taken from the jet grouting cover, lead to concentrated high stresses in the small bedding area of the jet grouting cover. This stress concentration leads to comparatively high compressions and settlements. On the other hand partial face advance leads to less stress concentration, however the different delayed headings lead to multiple load rearrangements and therefore the surface experiences approximately the same settlements.
Finally it can be concluded that with the jet grouting cover the settlements are not reduced in comparison to those caused by partial face advances. However, the face stability clearly increases by using a jet grouting cover.

7. COMPARISON OF SETTLEMENTS OF DIFFERENT SHOTCRETE TUNNELS, DRIVEN UNDER ATMOSPHERIC CONDITIONS

7	U5/9 Ostbahn-hof	U5/9 Theresien-wiese	U3N1 W4 / W3
tunnelling cross section			
cross section area [m≤]	200	175	170 - 200
covering [m]	9,3	9,6	6,5 / 11
max. settlement [mm]	36	38	26 / 40

5. PIPE SCREEN COVER FOR THE UNDERPINNING OF A BUILDING IN U3 NORTH LOT 1

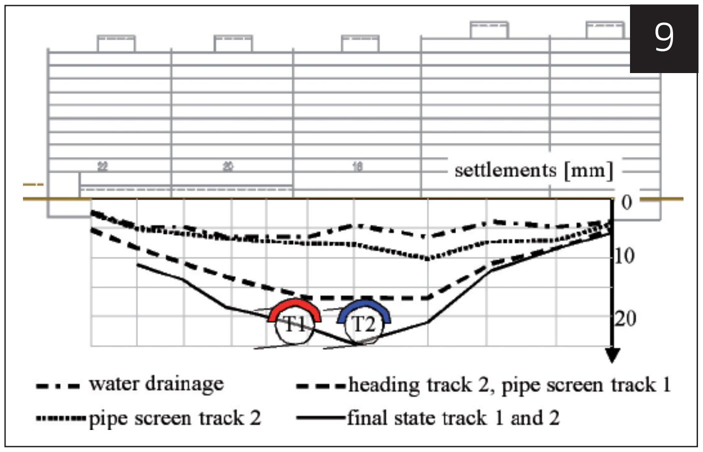
The two shotcrete headings of section W1 in the above described subway Lot U3 North-1 in Munich had a cross sectional area of A = 41 m^2 and were driven in the tertiary soils under atmospheric conditions with the help of wells, dewatering the tertiary sand layers.
In this section the underpinning of the Werner-Friedmann-Bogen, a building complex with 12 floors, is of special interest. The foundation pressure of the 3 m wide strip foundation, which lies in the centre of the building and carries the main loads, amounting to nearly 300 kN/m^2.
At a vertical distance of approx. 12 m between the foundation and the crown, a pipe screen cover was planned as an additional measure of protection (figure 8), because



8. CROSSING THE BUILDING WERNER-FRIEDMANN-BOGEN (LONGITUDINAL SECTION)

the tertiary soil cover amounted only 4 m and full water pressure was acting from the quaternary to the surface of the tertiary soils. At the southwest side of the Werner-Friedmann-Bogen an underground garage follows.

For every pipe screen 38 pipes were installed. The length of the pipes amounted to 12 m with an overlap of 4 m. The bore diameter amounted to 146 mm with a 6 mm annular space. In figure 9 the settlement trough along the Werner-Friedmann-Bogen is shown as dependent on the development of the heading.
Due to the dewatering of the tertiary sand layers settlements of 5 mm to 7 mm were recorded. The installation of the pipe screen and the forward directed settlements of the heading of track 2 increased the maximum settlements to approx. 10 mm. The largest settlements resulted from the 2 headings. Finally the maximum settlements amounted to 25 mm.



9. SETTLEMENTS OF WERNER-FRIEDMANN-BOGEN DURING TUNNELLING (CROSS SECTION)

As a comparison with measuring of further cross sections without pipe screen shows, the maximum settlements were measured under the Werner-Friedmann-Bogen. It is clear that the foundation loads lead to higher settlements and because of the smaller soil cover only limited arching develops. Furthermore, installation also cause settlements. However, it is crucial to settlements, that the pipe screens as well as the surrounding soil layers experience some deformation, before the system can carry the expected load in both the longitudinal and lateral directions. That is why the predominant settlements respectively occur shortly before and directly during the heading.
It can therefore be concluded, that the pipe screen primarily increases the safety of the tunnel face. For the installation and formation of the bearing effects, deformations are however necessary, which lead in this case to settlements of 25 mm. Pipe screens are only applicable for the reduction of settlements, if substantially higher settlements are expected without them.

6. HEADING WITH COMPRESSED AIR SUPPORT AND GROUTING IN U3 NORTH LOT 1

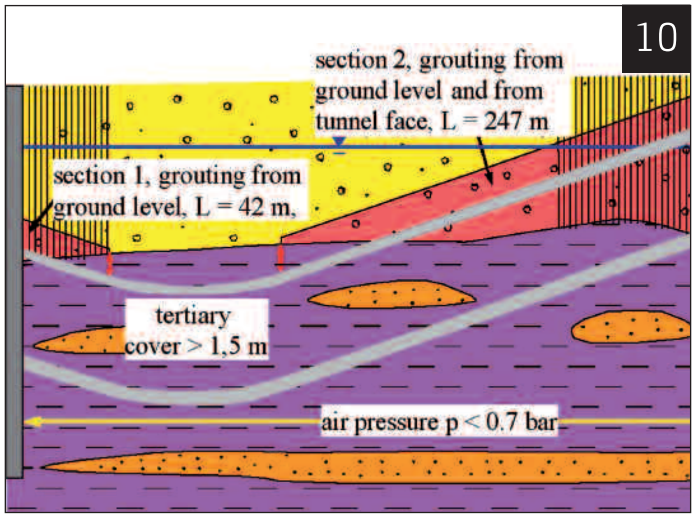
The geological conditions in the section 02 of the subway lot U3 north-1 are shown in figure 10. In this section a shotcrete heading with compressed air support was provided for. If the thickness of the tertiary soils above the crown reached less than 1.5 m, the overlaying quaternary gravels were grouted. In the grouting section 1 with a length of approx. 40 m the gravels were grouted from the surface. In section 2 the surface was not accessible. The gravel was therefore grouted from the tunnel. At the end of the heading the grouting was done again from the surface.

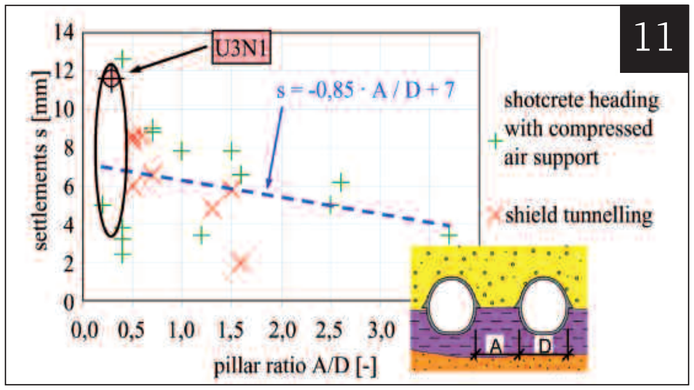
The aim of the grouting was to reduce the permeability of the gravel to k ≤ 5·10^{-5} m/s. This was controlled by permeability tests in the bore hole.

After the grouting activity the heading followed in shotcrete method with compressed air support with a maximum overpressure of 0.7 bar. Settlements were measured between 3 mm and 11 mm (without considerating of settlements due to water drainage). In order to assess this result, the results of settlement measurements of headings in Munich, with compressed air support and without grouting (shield tunnelling and shotcrete method) depending on the pillar ratio A/D are compared with the abovementioned result of the U3N1 measurement in figure 11.

It can be seen that the settlements measured if grouting was applied do not differ from those without grouting. It appears that the grouting did not reduce the settlements. Overall, the results confirm, that shotcrete headings with compressed air support lead only to very small settlements with small tangential inclinations, which cause no damage to conventional buildings.

Considering that the measured compressed air consumption was almost just as small as the calculated value, the very extensive grouting measure (21500 m grouting boreholes with more than 43000 grouting sleeves have been installed) can be determined as very successful.





7. CONCLUSINS AND FINAL REMARKS

In order to construct shallow tunnels in settlement-sensitive urban areas with the shotcrete method, measures have to be taken to increase the face stability and to reduce the settlements.

Besides the common measures (for example reducing the length of the advance step, etc.) special crown support measures are often used for this purpose. In this paper, the experiences of the authors demonstrated the purpose of using a frozen cover, a pipe screen cover, a jet grouting cover and a grouting cover. It has been shown, that the installation of crown supporting measures can have an extensive influence on the appearing deformations

As the settlements caused by shotcrete headings with conventional cross sections (approx. 40 m²) in the Munich underground are comparatively small (smaller than 20 mm to 25 mm for atmospheric headings and smaller than 10 to 20 mm for headings with compressed air support), the crown supporting measures do not have, as the examples show, decisive advantages regarding the deformations. However, if the crown supporting measures are used in a adequate way, a considerably higher safety potential occurs. This has to be considered if a decision has to be made, as to whether or not crown supporting measures are necessary in difficult sections.

10. *LONGITUDINAL SECTION OF THE GROUTING SECTION 02*

11. *SURFACE SETTLEMENTS DUE TO SHOTCRETE TUNNELLING USING COMPRESSED AIR IN MUNICH WITH DEPENDENCE ON THE PILLAR RATIO A/D*

SECOND SESSION
Adolfo Colombo

As the president of the Italian Tunnelling Society, which I believe is known throughout Italy and also internationally, I must congratulate and convey my best wishes to our past president Prof. Ing. Pietro Lunardi on this day of celebration and festivity, and I also express the hope that what he has sown over so many years may be carried forward and bear fruit also through his successors. Giuseppe is moving fast in this direction and with the help of his father, I believe he will inevitably reach the current and even higher levels of efficiency.
I would like here to recollect the figure of Pietro Lunardi at two particular moments in time which I feel it is important to recount. Firstly, he was, as you know, one of the founding members of the Italian Tunnelling Society. At a time when people talked only of the Subalpine Mining Association, because tunnels were seen as a mere appendage of mining activity, the experience of which was exported for those few road and rail tunnels

PROF. ING. ADOLFO COLOMBO,
CHAIRMAN OF THE ITALIAN TUNNELLING SOCIETY

CHAIRMAN'S TABLE
SECOND SESSION

which were conceded then, he had the insight to see that this new sector would develop, not just as a part of other activities, but as a science and technology which deserved promotion and investment in concepts and research.

It was an important insight, which together with the work and dedication of other founding members has led the Italian Tunnelling Society today to become universally recognised throughout the world as one of the most up-to-date and one of the most present and efficient associations in the sector.

The other moment that I would like to remember is that when we worked together on Venezia Station on the Milan Urban Railway Link Line which became in some ways a symbol of his activity. It was known almost everywhere because of the cellular arch method used to build it. With an overburden of just four metres over a tunnel diameter of over 30 metres, everybody felt it was impossible to build a station in that position and in that place in the city centre. However, by using this profoundly innovative method which he conceived of and designed, it was possible to build it on schedule as planned without almost anyone on the nearby surface noticing. That is because in Pietro Lunardi's vision, a point of equilibrium always exists as a reference. By passing from a pre-excavation situation to an excavation completed situation by means of a series of points of equilibrium and that is moments in time at which all the forces in play are in equilibrium, it is possible to construct works which until a few years ago were defined as not possible, or at least not without serious dangers to health, safety and existing structures.

In this sense it is also important to remember the International Conference on Tunnels held in Milan in 2001, which for better or for worse, only history will tell us, Pietro Lunardi was unable to inaugurate because he was required on that precise day to swear in as a Minister of the Republic of Italy. In this sense, it is my pleasure, together with you, to repeat our thanks to Pietro Lunardi. We are grateful to you for what you have done, for your desire for the modern and for research which has brought the recognition to the world of tunnelling that it enjoys today both generally and in technical and scientific terms.

After Japan, Italy is the country with the highest number of kilometres of tunnels in the world. At the moment, due in part to the programmes initiated when Pietro Lunardi was a minister, projects to cross the Alps, high speed rail projects and motorway expansion and modernisation projects are in progress and are finally being constructed and this again leads us to forecast the construction of a large number of new tunnels.

We are all committed to this and today's event, which has allowed me to see and greet many old friends and also some new ones, is evidence that our world is evolving and progress has certainly not come to a halt.

VENEZIA STATION
MILAN URBAN RAILWAY LINK LINE

Giovanni Barla

FULL FACE EXCAVATION AND REINFORCEMENT COUPLED WITH YIELD-CONTROL SUPPORT SYSTEMS TO COPE WITH SQUEEZING CONDITIONS

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POLYTECHNIC UNIVERSITY OF TURIN (ITALY)
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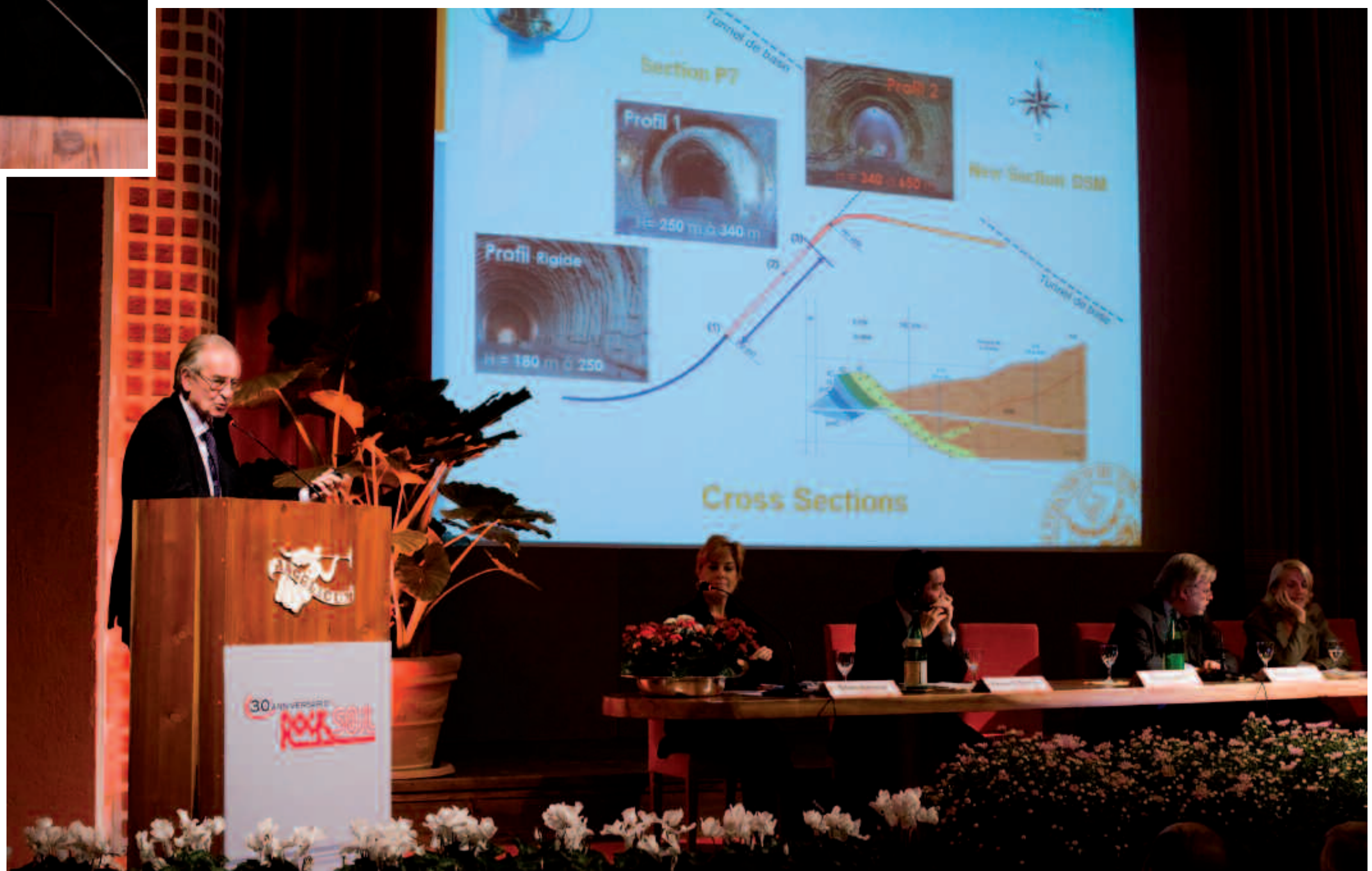
Recent innovations in yield-control support systems applied to conventional tunnelling and full face excavation in squeezing conditions are discussed in this lecture. The Saint Martin access adit along the Base Tunnel of the Lyon-Turin railway link is presented as a case study. Also discussed are in situ performance monitoring, laboratory investigations, and advanced modelling studies performed.

1. INTRODUCTION

The invitation to prepare this Lecture for Rocksoil's 30th Birthday Conference gives us the opportunity to describe some of the research work carried out in recent years at Politecnico di Torino, Department of Structural and Geotechnical Engineering, on the squeezing behaviour of tunnels. In doing this, we will add to the content of a Keynote



1

1. "SQUEEZING ROCK" REDUCES THE TUNNEL CROSS SECTION AS WELL EVIDENCED IN THE SAINT MARTIN LA PORTE ACCESS ADIT ALONG THE LYON-TURIN BASE TUNNEL

Lecture recently prepared for Eurock 2009 (Barla, 2009), which deals with innovations in support systems applied to conventional tunnelling in such conditions.
The choice of the topic for this lecture is indeed appropriate on this occasion as the support system recently implemented and described gives clear evidence of the importance, for successful tunnelling in squeezing conditions, of "understanding and controlling the behaviour of the core ahead of the advancing tunnel" as suggested by Lunardi (2000) through successful applications in engineering practice in Italy and abroad.
Tunnel construction in squeezing conditions is very demanding and reliable predictions at the design stage are difficult if not impossible, so that most often the use of the "interactive observational approach" is advocated (Alagnat, 2005; Barla, 2005). If consideration is given to the construction of deep tunnels (such as the new Alpine Tunnels, e.g. Lötschberg, Gotthard, Lyon-Turin, and Brenner Base Tunnels), alignment constraints, and uncertainties of geological exploration, it is not always possible to avoid the "difficult ground" which may result in squeezing conditions. Therefore, the selection of the most appropriate excavationconstruction method to be adopted is highly problematic and uncertain.
In mechanized tunnelling, due to the fixed geometry and the limited flexibility of the TBM (Tunnel Boring Machine), allowable space to accommodate ground deformations is restricted.
On the contrary, in conventional tunnelling, where a considerably larger profile can be excavated in order to allow for large deformations, inevitably excavation will take place with a low rate of advance. It is however true that, if the work is well planned and appropriate stabilization measures are implemented, excavation may proceed at an acceptable rate of advance even in severely squeezing conditions.

The case study presented in this lecture deals with the Saint Martin La Porte access adit (Lyon-Turin Base Tunnel) which is being excavated through the Carboniferous Formation. This tunnel has experienced severely squeezing conditions in some tunnel sections. Recent innovations in yield-control support systems adopted in conventional tunnelling are discussed together with in situ performance monitoring, laboratory investigations, and advanced modelling studies which have been carried out with the aim to gain insights into the understanding of squeezing behaviour.

2. SQUEEZING BEHAVIOUR

The term "squeezing rock" originates from the pioneering days of tunnelling through the Alps. It refers to the reduction of the tunnel cross section that occurs as the tunnel is being excavated (Figure 1). Based on the work of a Commission of the International Society for Rock Mechanics (Barla, 1995), which has described squeezing and the main features of this behaviour, it is agreed today that "squeezing of rock stands for large time-dependent convergences during tunnel excavation". This happens when a particular combination of material properties and induced stresses causes yielding in some zones around the tunnel, exceeding the limiting shear stress at which creep starts. Deformation may terminate during construction or continue over a long period of time.

The magnitude of tunnel convergence, the rate of deformation, and the extent of the yielding zone around the tunnel depend on the geological and geotechnical conditions, the insitu state of stress relative to rock mass strength, the groundwater flow and pore water pressure, and the rock mass properties. Squeezing is therefore synonymous with yielding and time-dependence, and often is largely dependent on excavation and support techniques being used. If the support installation is delayed, the rock mass moves into the tunnel and a stress redistribution takes place around it. On the contrary, if deformation is restrained, squeezing will lead to long-term load build-up of the support system.

The squeezing behaviour during tunnel excavation has intrigued experts for years, and often has caused great difficulties for completing underground works, with major delays in construction schedules and cost overruns.

There are numerous cases of particular interest in Europe and worldwide where squeezing phenomena have occurred, providing some insights into the ground response during excavation. A review of these case studies leads to the following remarks (Kovari, 1998):

- Squeezing behaviour is associated with poor rock mass deformability and strength properties. Based on previous experience, there are a number of rock complexes where squeezing may occur if the loading conditions needed for the onset of squeezing are present: gneiss, micaschists and calcschists (typical of contact and tectonized and fault zones), claystones, phyllites, flysch, clay-shales, marly-clays, siltstones, etc.
- Squeezing behaviour implies that yielding will occur around the tunnel. The onset of a yielding zone in the tunnel surround causes a significant increase in tunnel convergence and face displacements (extrusion); these are generally large, increase in time and form the most significant aspects of the squeezing behaviour.

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▸ Orientations of discontinuities, such as bedding planes and schistosities, play a very important role in the onset and development of large deformations around tunnels, and therefore also on the squeezing behaviour. In general, if the main discontinuities strike parallel to the tunnel axis, the deformation will be enhanced significantly, as observed in terms of convergence during face advance.

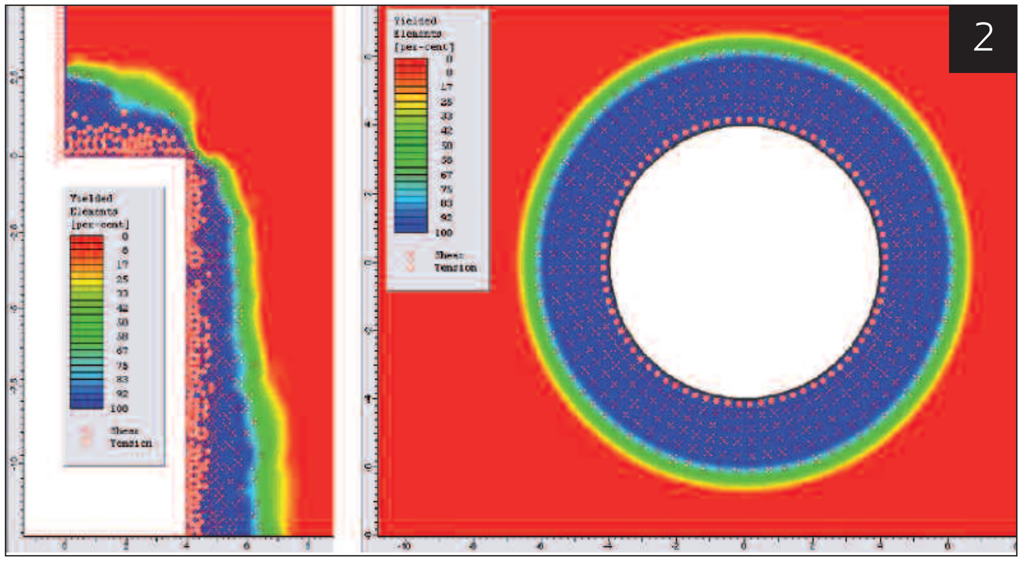
▸ The pore pressure distribution and the piezometric head also can influence the rock mass behaviour. Drainage measures that cause reduction in piezometric head both in the tunnel surround and ahead of the tunnel face help to reduce ground deformations.

▸ Construction techniques for excavation and support (i.e. the excavation sequences and the number of excavation stages which are adopted, including the stabilisation measures implemented) may influence the overall stability conditions of the excavation. In general, the ability to provide an early confinement on the tunnel periphery and in near vicinity to the face is considered to be the most important factor in controlling ground deformations.

▸ Large deformations associated with squeezing may occur in rocks susceptible to swelling. Although the factors that cause either behaviour are different, it is often difficult to distinguish between squeezing and swelling, as the two phenomena may occur at the same time and induce similar effects. For example, in over-consolidated clays, the rapid stress-relief due to the tunnel excavation causes an increase in deviatoric stresses with simultaneous onset of negative pore pressure. In undrained conditions, the ground stresses may be such as not to cause squeezing. However, due to the negative pore pressure, swelling may occur with a more sudden onset of deformations under constant loading. Therefore, if swelling is restrained by means of early invert installation, a stress increase may take place with probable onset of squeezing.

3. TUNNEL AND FACE STABILITY IN WEAK ROCK

In general, the major difficulties encountered when tunnelling in weak rock are associated with the stability of the tunnel and of the face (Lunardi, 2000). As illustrated in Figure 2, obtained with axi-symmetric and plane strain models by the Finite Element Method (FEM), a typical plastic yield zone develops in the rock mass surrounding the advancing tunnel.
Depending on the rock mass properties, the wall yield zone may interact or not with the face yield zone.
Of significant importance for the understanding of the tunnel response are both the radial displacements of the tunnel wall and the corresponding longitudinal displacements of the tunnel face as excavation proceeds. The tunnel



2. *PLASTIC YIELD ZONE DEVELOPING AROUND A 10 M DIAMETER CIRCULAR TUNNEL*
(LEFT: AXI-SYMMETRIC/FEM ANALYSIS
RIGHT: PLANE STRAIN FEM ANALYSIS; IN SITU STRESS P_0 = 5.0 MPA, DEFORMATION MODULUS E_D = 400 MPA, ROCK MASS STRENGTH σ_{cm} = 0.6 MPA, DILATION ANGLE Ψ = 0.0)

face follows the same deformational pattern as the tunnel itself, although the longitudinal displacements of the "core" ahead of the face are significantly smaller than the tunnel radial displacements. As shown by Hoek and Marinos (2000), this is well illustrated in squeezing rock conditions by plotting the radial strain of tunnel wall (ε_t) and axial strain of tunnel face (ε_f) against the ratio of rock mass strength (scm) to in situ stress (p_o). Note that ε_t is defined as the percentage ratio of radial tunnel wall displacement u_r to tunnel radius a and ε_f as the percentage ratio of longitudinal face displacement u_f to tunnel radius a.

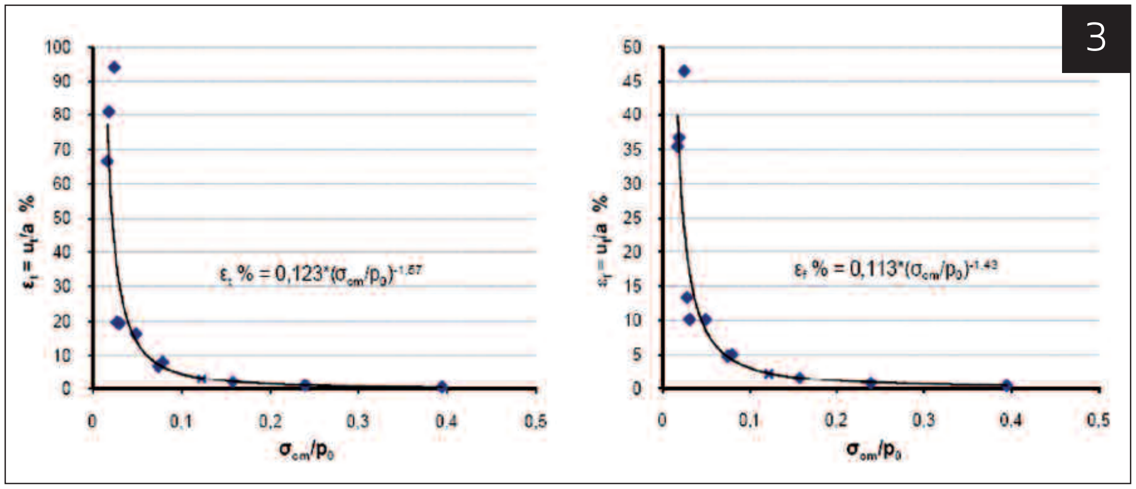
Figure 3 shows such a plot for a 10 m diameter circular tunnel at depth where the rock mass is represented as a continuum, isotropic, elastoplastic perfectly plastic model with a Mohr-Coulomb yield surface and by assuming a zero dilation. The results were generated with an axi-symmetric FEM model under no support pressure either at the wall or at the face. It is shown that the strains et and ef increase asymptotically when the ratio of the rock mass strength to in situ stress is smaller than 0.2, to indicate the onset of severe instability of the tunnel if no adequate support measures are implemented (Hoek, 2001).

Of the available options for conventional tunnel excavation and construction in squeezing rock (e.g. multiple headings, top heading and benching down, full face), the most recent method is the full-face method (Figure 4). A significant advantage of this method is the large working space available at the face, so that large equipment can be used for installing support/stabilization measures at the tunnel perimeter and ahead of the face (Figure 4 left).

However, in squeezing conditions, this method requires a systematic reinforcement of the face and of the ground ahead. Generally, the cross section is entirely open and a primary lining is installed as near to the face as possible, with the invert forming a "closed ring" as shown in Figure 4 (right).

One of the two methods, "heavy" and "light" methods, can be applied (Kovari, 1998). With the "heavy method" ("resistance principle"), the primary lining is designed to be very stiff (generally composed of steel fibre shotcrete and heavy steel sets) and the "ring is closed quickly" (Figure 4 right). The final concrete invert (first) and final con-

3. RADIAL STRAIN OF TUNNEL WALL (ε_t) AND AXIAL STRAIN OF TUNNEL FACE (ε_f) VERSUS THE RATIO OF ROCK MASS STRENGTH (σ_{cm}) TO IN SITU STRESS (p_0)



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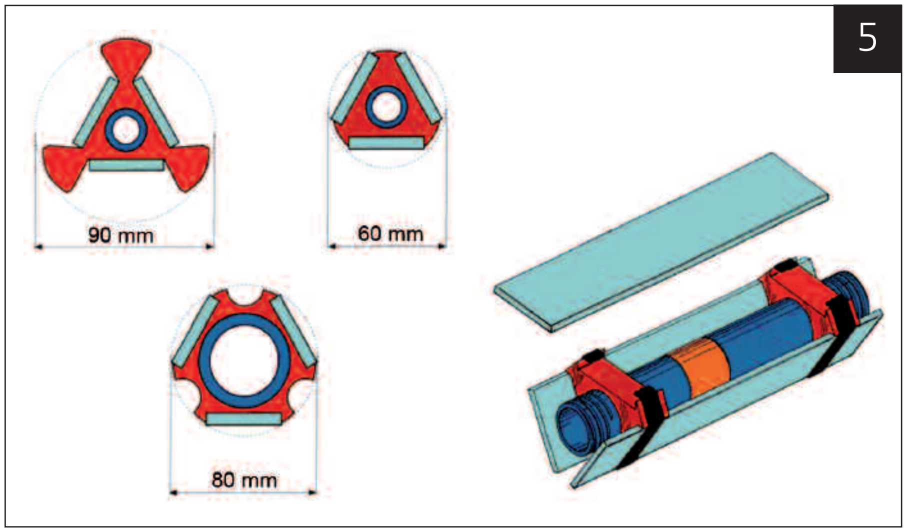
4. *FULL FACE EXCAVATION AND CONSTRUCTION METHOD: FACE REINFORCEMENT AND RING CLOSURE*

GIOVANNI BARLA

crete lining (second) are cast within a short distance from the face. It is apparent that if very high rock pressures are expected, as is the case of deep tunnels, this solution soon becomes impractical. With the "light method" ("yielding principle"), large deformations are allowed to develop around the tunnel with the expectation that rock pressure will decrease with increasing deformation. The excavation profile is chosen so as to maintain the desired clearance and to avoid the need for re-profiling. A key point is to be able to control the development of deformations. A suitable tunnel support system is to be adopted that will allow for accommodating deformations without damage of the lining.

In order to achieve face stability when driving a tunnel consists in reinforcing the rock mass ahead by means of grouted fibre-glass dowels (Lunardi, 2000). There are a number of fibreglass structural elements that may be adopted. Both smooth and corrugated tubes are available. More recently, flat elements (Figure 5) are being used which can be assembled in situ in a wide variety of types; they are very easy to inject and transport, and they allow reinforcement advance steps up to 25 m. In typical reinforcement schemes the fibreglass elements are used to reinforce the "core" ahead of the face and in cases to provide a "reinforced ring" around the tunnel. In these cases, in line with the "resistance principle", closely spaced steel sets are incorporated in a thick shotcrete shell to form the primary lining which is installed close to the face as early as possible (Figure 4).



5. FLAT FIBRE-GLASS STRUCTURAL ELEMENTS ADOPTED FOR FACE REINFORCEMENT IN THE FULL-FACE EXCAVATION AND CONSTRUCTION METHOD

4. YIELDING SUPPORT SYSTEMS

A yielding support system which has been originally applied in mines and is still used today consists of providing sliding joints in top hat steel sets (TH, Toussaint-Heintzmann type) embedded in the shotcrete lining. The tangential force in the steel sets is controlled by the number and by the pre-tensioning level of the sliding joints in the steel sets. Open gaps are provided with dense rock bolting of the tunnel cross section. After a pre-defined amount of convergence these gaps are filled with shotcrete (Figures 6 and 7).

Various design options (Figures 8 to 10) have been proposed to better deal with severely squeezing conditions such as:

- LSC element (Lining Stress Controller): developed by the Geotechnical Group Graz between 1996 and 1999 (Schubert, 1996; Schubert et al., 1999) is distributed by Alwag, Austria, a Dywidag-System International Unit. In its most recent design, each unit consists of coaxial cylinders which are loaded in the axial direction and buckle in stages under a stress of 4.2 MPa with a limit strain of 20 % approximately (typical).

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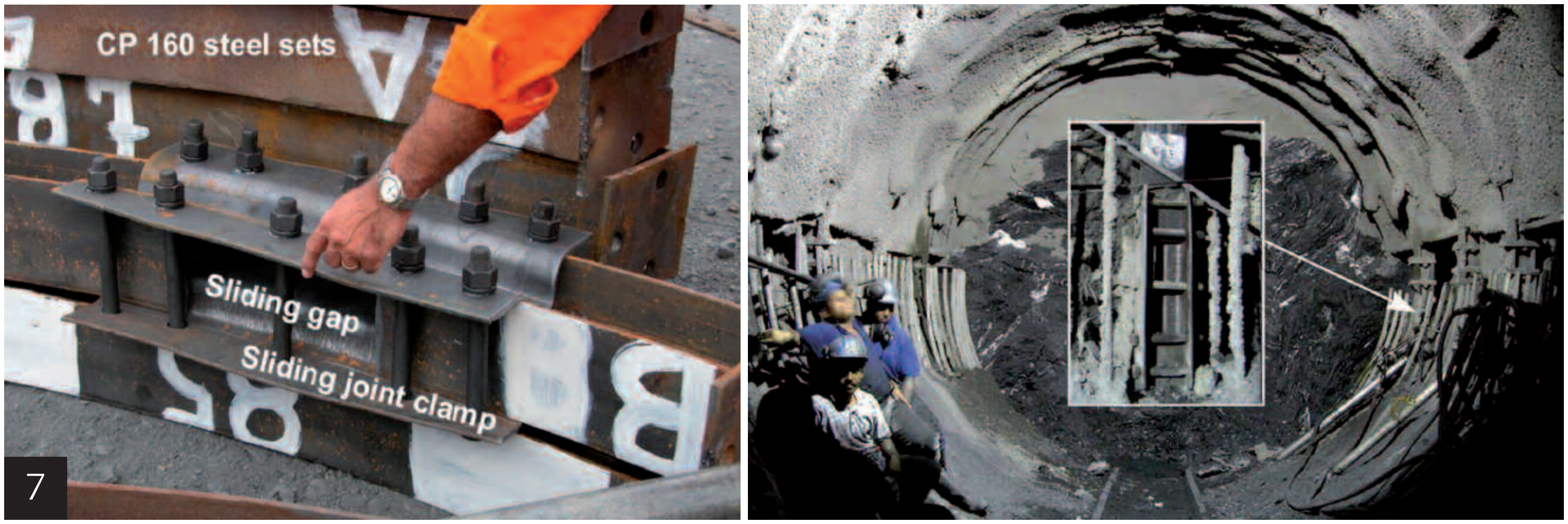
6. YIELDING SUPPORT SYSTEM WITH TH STEEL SETS AT THE GOTTHARD BASE TUNNEL
(COURTESY OF ALPTRANSIT GOTTHARD AG)

7. YIELDING SUPPORT WITH CP 160 STEEL SETS AND SLIDING JOINT CLAMPS AT THE YACAMBÚ-QUIBOR TUNNEL
(HOEK E GUEVARA, 2009)

▸ WABE honey-comb element: available from Eisenhütte Heintzmann GmbH & Co. KG, Bochum, Germany, consists of steel tubes inserted between steel plates to form a number of layers (Geomechanics & Tunnelling, 2009). Its deformational response to loading may change based on tube characteristics (diameter, wall thickness, length, material and steel quality), insertion of smaller diameter tubes into existing tubes, etc. The reported yield strength is 2.5 MPa for 40 % strain approximately.



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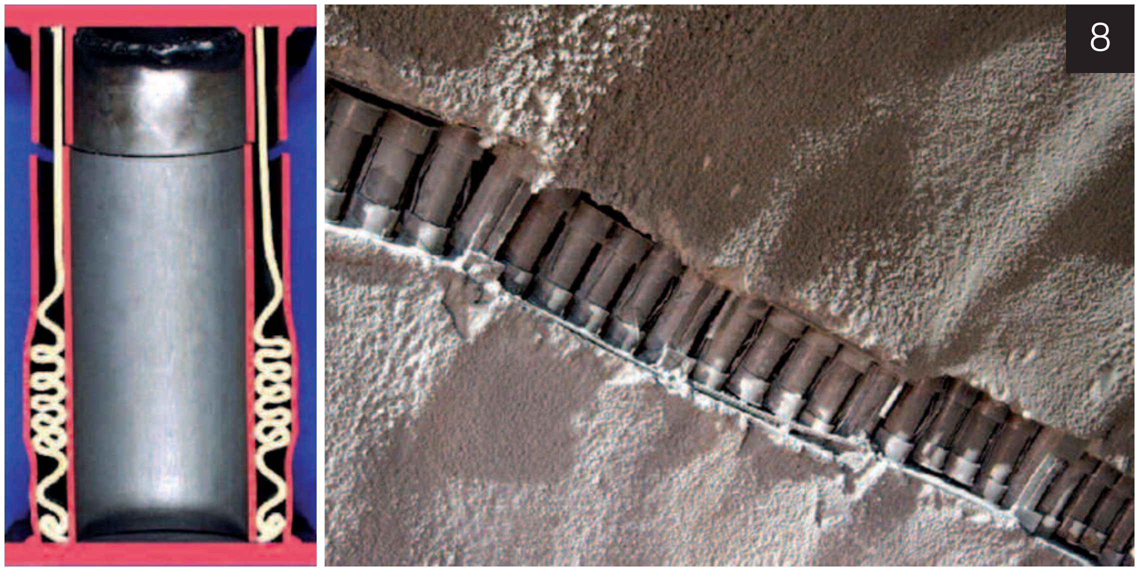
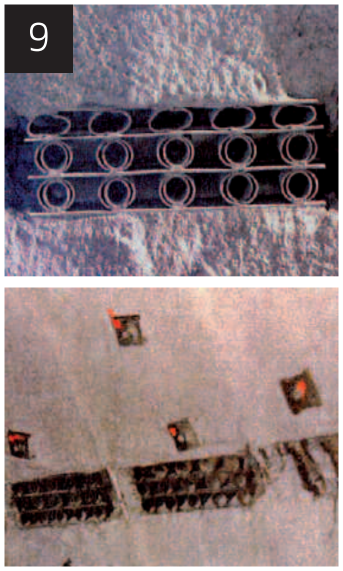




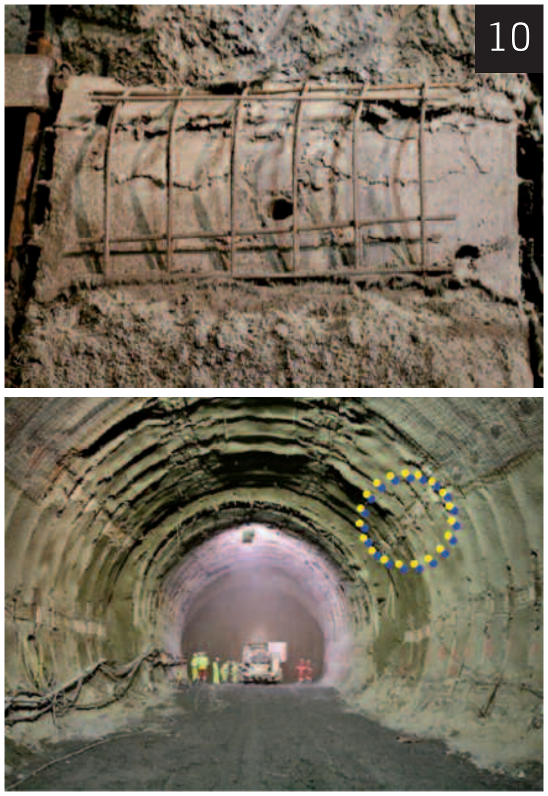
8. YIELDING SUPPORT WITH LSC ELEMENTS
(SCHUBERT, 1996; SCHUBERT ET AL., 1999)

9. YIELDING SUPPORT WITH WABE ELEMENTS
(GEOMECHANICS & TUNNELLING, 2009)

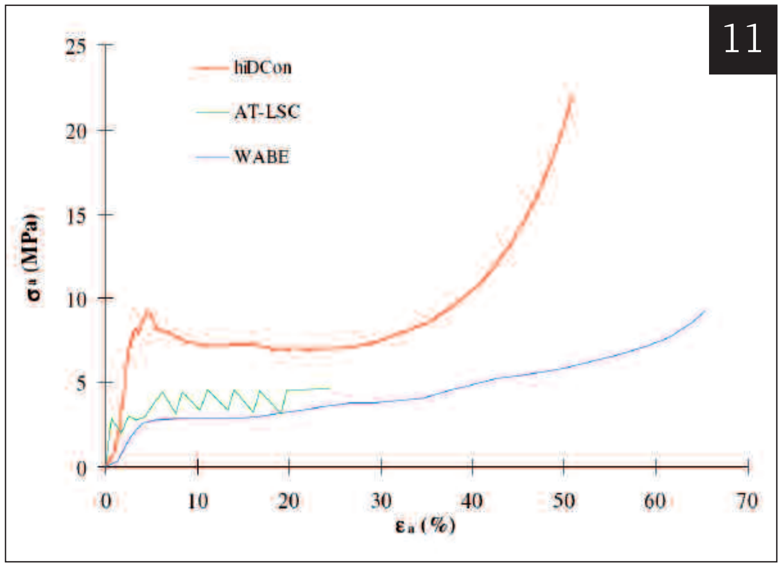
10. YIELDING SUPPORT WITH HIDCON ELEMENTS
(KOVARI, 2005; THUT ET AL., 2006)

11. STRESS - STRAIN CHARACTERISTICS OF LSC, WABE AND HIDCON ELEMENTS

▸ hiDCon element (highly Deformable Concrete): was originally developed by Solexpert (Kovari, 2005; Thut at al., 2006). It is composed of a mixture of cement, steel fibres and hollow glass particles. The glass particles increase the void fraction of the mixture and collapse at a predetermined compressive stress. Depending on the composition of the mixture the yield strength ranges from 4 to 10 MPa with a maximum allowable strain approximately equal to 40 - 50 %.



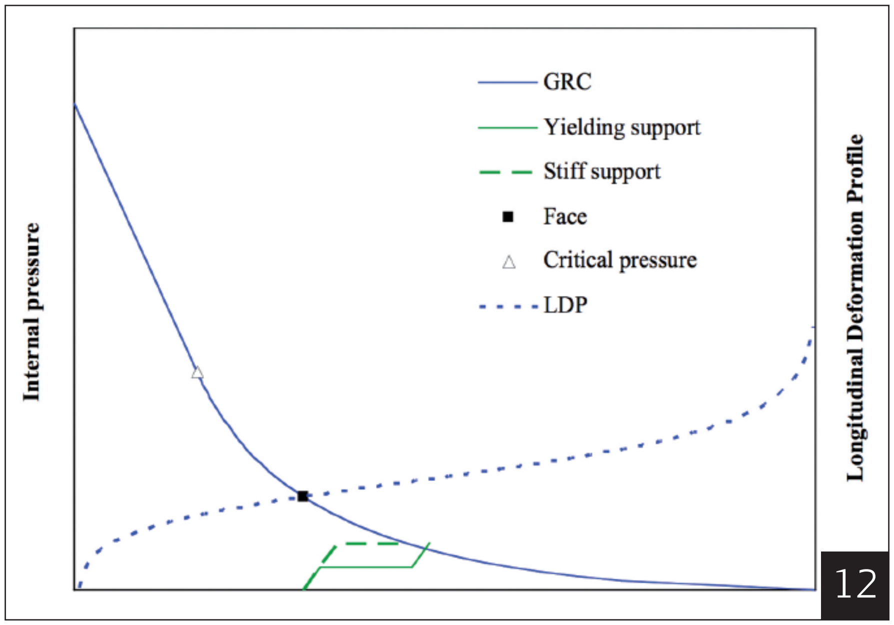
Typical stress-strain characteristics of these elements are given in Figure 11. It is shown that a yielding element initiates yielding at a specified stress level as it undergoes a very small strain (smaller than 5% strain approximately).



Deformation continues with the element length shortening significantly before the stress in the element increases. The level of yielding stress is to be selected to allow the yield-control support system to deform as desired, without any damage of the shotcrete lining. The amount of desired deformation depends on the amount of over-excavation which is chosen for the tunnel section, in order to avoid re-profiling.
To illustrate in a diagrammatic form the tunnel response, when a "yield-control support system" (which is formed by a composite shotcrete lining which incorporates the top hat steel sets placed with a given longitudinal spacing and a number of yielding elements) is installed at the face, the Convergence-Confinement (CC) method with its basic components, i.e. the Longitudinal Deformation Profile (LDP), the Support Characteristic Curve (SCC), and the Ground Reaction Curve (GRC), can be used. As shown in Figure 12, a yielding support installed immediately behind the face will allow the tunnel to converge in a controlled manner while maintaining a support pressure which is within its capacity.
On the contrary, if one is to install a "stiff support system" at the face in similar rock mass conditions (i.e. with the same GRC/LDP curve), as shown again in Figure 12 this would fail.
One could however delay support installation, but this would be highly undesirable and very dangerous in a weak rock mass as one would have to work at the face in an unsupported tunnel section. The introduction of the yielding elements in the support to form a "yieldcontrol support system" can instead overcome this, "since the activation of the support is delayed but the support system is in place to catch runaway if this should occur" (Hoek, 2006).
Therefore, a yield-control support system is intended to allow the tunnel to converge (DR) in a "controlled manner" while keeping the tangential stress sJ in the lining nearly constant and applying a confinement stress sr to the surrounding rock mass (Figure 13). It has been proven that higher is the yield stress of the support, the lower it will be the final ground pressure. This is like to say that "a targeted reduction in ground pressure can be achieved not only by installing a support that is able to accommodate a larger deformation (which is a well known principle), but also by selecting a support that yields at a higher pressure" (Cantieni & Anagnostou, 2009).

12. *DIAGRAMMATIC REPRESENTATION OF THE BEHAVIOUR OF A YIELDING SUPPORT BY USING THE CONVERGENCE-CONFINEMENT METHOD*

13. *DIAGRAMMATIC REPRESENTATION OF THE BEHAVIOUR OF A YIELD-CONTROL SUPPORT SYSTEM WITH YIELDING ELEMENTS INSERTED IN THE SHOTCRETE LINING*



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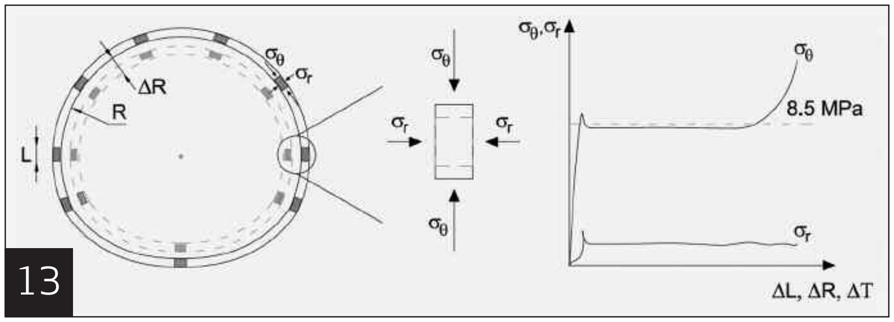

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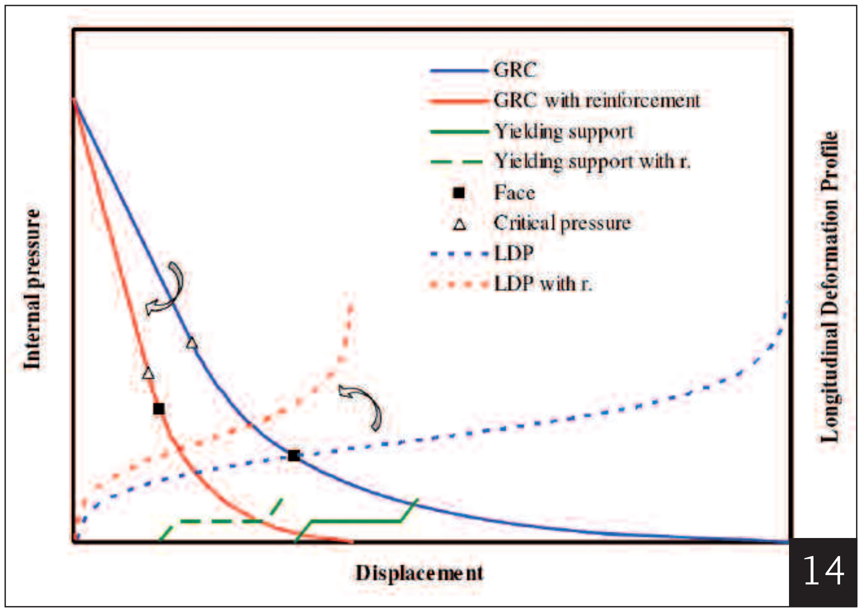
5. FULL FACE EXCAVATION AND REINFORCEMENT COUPLED WITH YIELD-CONTROL SUPPORT SYSTEMS

As discussed above, in squeezing conditions, the understanding of the tunnel response as the radial displacements of the tunnel wall and the corresponding longitudinal displacements of the tunnel face increase, and excavation proceeds, is essential for an appropriate selection of both the tunnel support and face reinforcement. When the full face excavation method is adopted, an usual measure implemented in order to ensure stability of both the face and the tunnel, consists in reinforcing the core ahead of the face. An additional measure is the immediate installation of a very stiff support, in line with the "heavy method" ("resistance principle"). Also, the final concrete invert (first) and the final concrete lining (second) are cast in sequence within a short distance from the face Experience shows however, as already noted, that in severely squeezing conditions and at great depth such a "heavy method" is not applicable as the tunnel support cannot cope with the heavy loading conditions expected with the result that this may be badly damaged, which is very dangerous for the workers in the tunnel. As described above, the alternative has been to adopt the "light method" ("yielding principle") which consists in the use of a yield-control support system which is designed to allow the tunnel to deform as desired, according to the amount of over-excavation needed in order to avoid re-profiling of the tunnel perimeter. In such cases the most care is needed in order to keep the face stable as the tunnel perimeter undergoes large deformations.



__14.__ DIAGRAMMATIC REPRESENTATION OF THE BEHAVIOR OF A YIELDING SUPPORT COUPLED WITH FULL FACE EXCAVATION AND REINFORCEMENT

With the above in mind, in severely squeezing conditions, "full face excavation and reinforcement coupled with yield-control systems" seem to be the real way to go. In order to illustrate this in a diagrammatic form, we can use once again the Convergence-Confinement (CC) method as depicted in Figure 14. Let us start with the same rock mass conditions and initial hydrostatic far field stress acting on the rock mass as considered in Figure 12, i.e. by taking the same Ground Reaction Curve (GRC) and the same Longitudinal Deformation Profile (LDP).

If it is assumed that the core ahead of the face is reinforced significantly (e.g. by means of fibre glass dowels), both the GRC and LDP will be modified as illustrated, i.e. the radial displacement that occurs in a given cross section as the tunnel advances as well as the extent of the yield zone that develops around the tunnel and ahead of the face will be reduced.

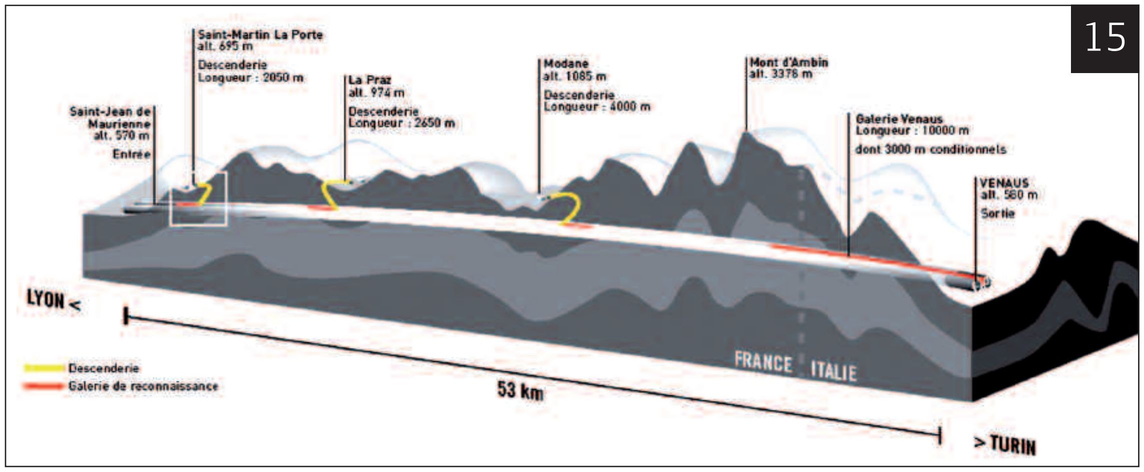
The diagram indicates that, when the yield-control support system is installed at the face, the effect of the reinforced core will be such as to reduce the radial and longitudinal displacements that occur at the face. As a consequence, the yielding support system will allow the tunnel to undergo larger deformations in controlled conditions than is the case without reinforcing the core, given that only a prescribed amount of deformation is permitted by the yielding elements inserted in the shotcrete lining, before they start to become less effective and make the support to increase in stiffness.

6. CASE STUDY

A yield-control support system coupled with full face excavation and reinforcement has been adopted successfully in order to cope with the severely squeezing conditions encountered during excavation through the Carboniferous Formation in the Saint Martin La Porte access adit (Lyon-Turin Base Tunnel). Such a case study will be considered in the following by giving a description of in situ performance monitoring, laboratory investigations, and advanced modelling studies carried out.

6.1 Project background

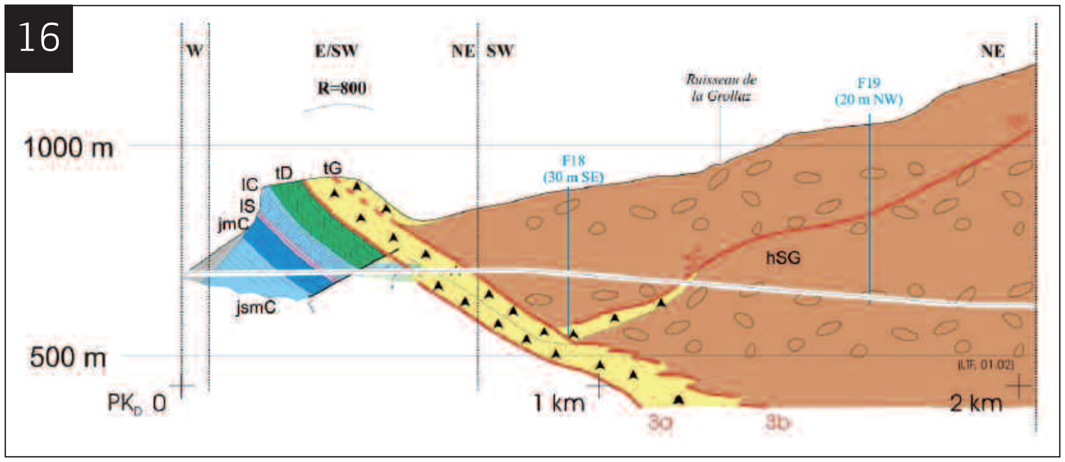
The Saint Martin La Porte access adit is a vital part of the early works for the Lyon-Turin Base Tunnel, which is at the centre of the axes linking the North and South, and East and West Europe and is to be excavated between the portals in Italy and France (Figure 15).



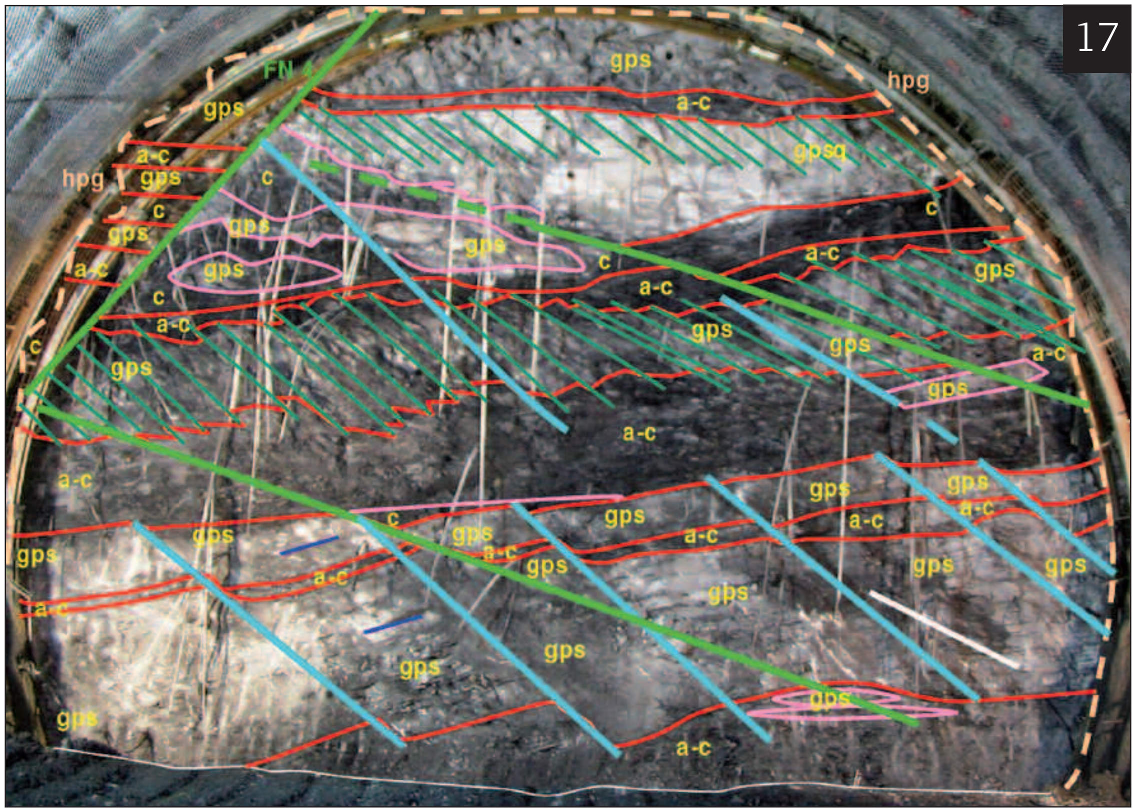
15. *THE BASE TUNNEL BETWEEN SAINT-JEAN DE MAURIENNE (FRANCE) AND VENAUS (ITALY) INCLUDING THE THREE ACCESS ADITS*

AT PRESENT, CONSIDERATION IS BEING GIVEN TO EXTEND THE TUNNEL LENGTH FROM 53 TO 57 KM, WITH THE SOUTH PORTAL LOCATED NEAR TO SUSA (ITALY)

At present two access adits (La Praz and Modane) are complete whereas the Saint Martin La Porte adit is being excavated. These adits are essential in understanding the geological, geomechanical, and hydro-geological conditions along the alignment and for the selection of the excavation method to be used. They will also provide multiple faces for construction,



16. *GEOLOGICAL PROFILE ALONG THE ACCESS ADIT*



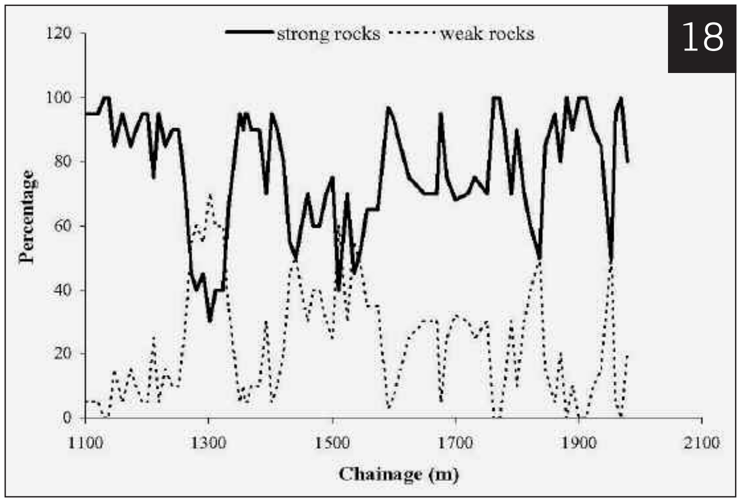
17. *TYPICAL GEOLOGICAL CONDITIONS*
(GPS - SANDSTONE, A - CLAY-SHALES, C-COAL, ETC.)

18. *PERCENT DISTRIBUTION OF "WEAK" ROCKS AND "STRONG" ROCKS*

and be used for ventilation access for maintenance and rescue teams if necessary.

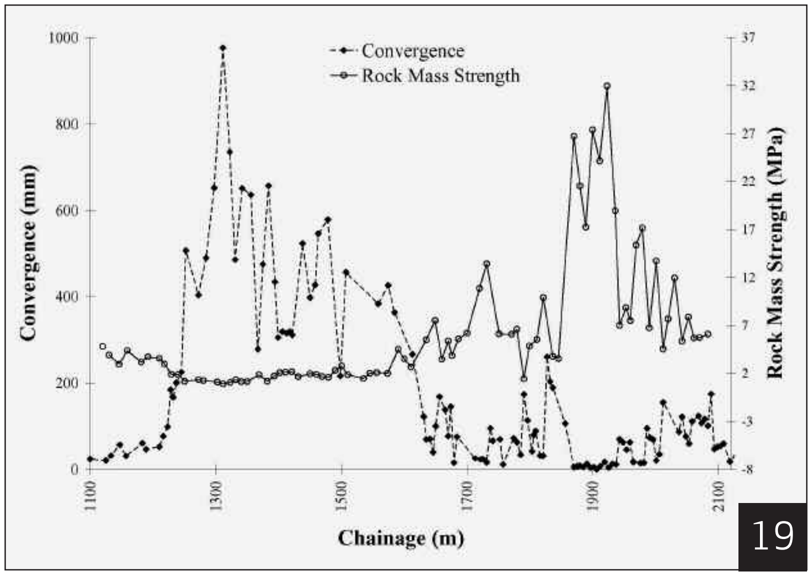
The Saint Martin La Porte access adit (Figures 16 and 17) is being excavated through the Carboniferous Formation, "Zone Houillère Briançonnaise-Unité des Encombres" (hSG in Figure 16), which is composed of black schists (45 to 55%), sandstones (40 to 50%), coal (5%), clay-like shales and cataclastic rocks.

A characteristic feature of the ground observed at the face during excavation (Figure 17) is the highly heterogeneous, disrupted and fractured condition of the rock mass. This formation is often affected by faulting that results in a degradation of the rock mass conditions. The overburden along the tunnel in the zone of interest ranges from 300 to 650 m. Excavation takes place in dry conditions.



In order to assess the rock mass quality during excavation detailed mapping of the geological conditions at the face is undertaken and the percent distribution of "strong" (sandstones and schists) and "weak" (coal and clay-like shales) rocks at the face is determined as shown in Figure 18. Also adopted is the Geological Strength Index (GSI) classification according to Hoek and Marinos (2000).

With interest in the potential for squeezing, the percentage strain in the rock mass surrounding the tunnel, defined by tunnel convergence/tunnel diameter was computed (see paragraph 6.3 below for the convergence data). By knowing the in situ stress and assuming the degree of difficulty associated with tunneling in squeezing rock to depend on the level of percent strain as discussed above, the distribution of the rock mass uniaxial compressive strength along the tunnel length was determined as shown in Figure 19.



19. CONVERGENCE AND DISTRIBUTION OF ROCK MASS STRENGTH

6.2 Support systems used

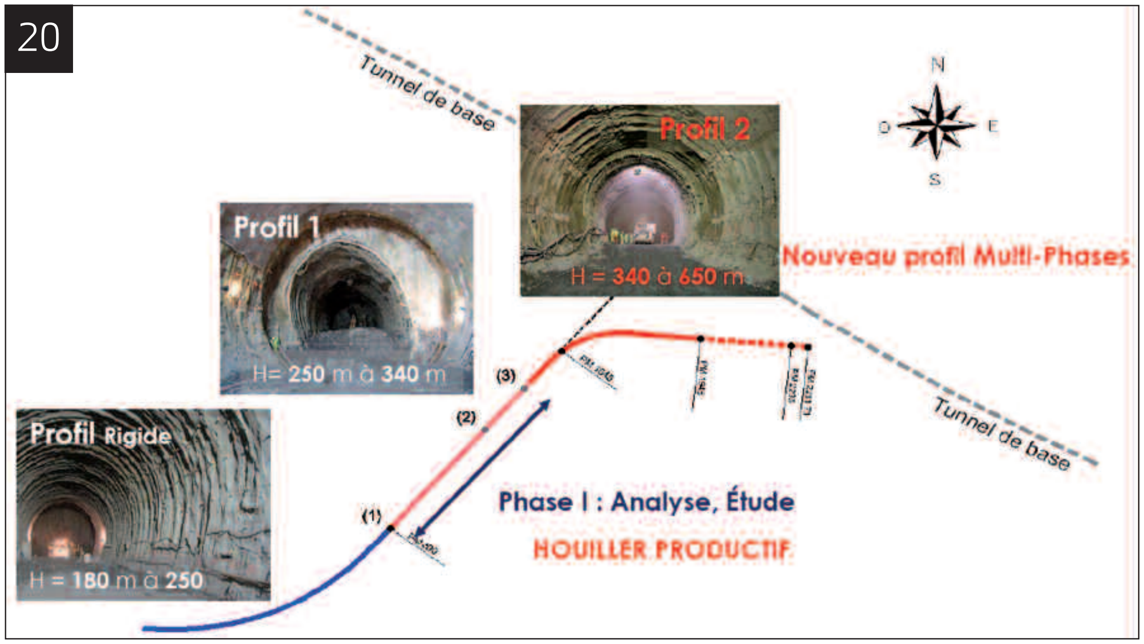
Several support systems were used in the Carboniferous Formation along the Saint Martin La Porte access adit as illustrated in Figure 20. It soon became apparent that a stiff support ("Profil Rigide") as initially adopted would not be feasible to cope with the severely squeezing conditions encountered.

A yielding support system with face-reinforcement by means of fibre glass dowels, top heat steel sets with sliding joints, rock anchors and a shotcrete layer with a horseshoe cross section was then adopted. However, the sections of the tunnel where this support system was installed underwent very large deformations with convergences up to 2 m and needed to be reprofiled ("Profil 1" in Figure 20). Design details of this support system are shown in Figure 21.

The design concept finally implemented consisted in the systematic use of full face excavation and reinforcement coupled with a yield-control support system by using a near circular cross section ("Profil 2" in Figure 20).

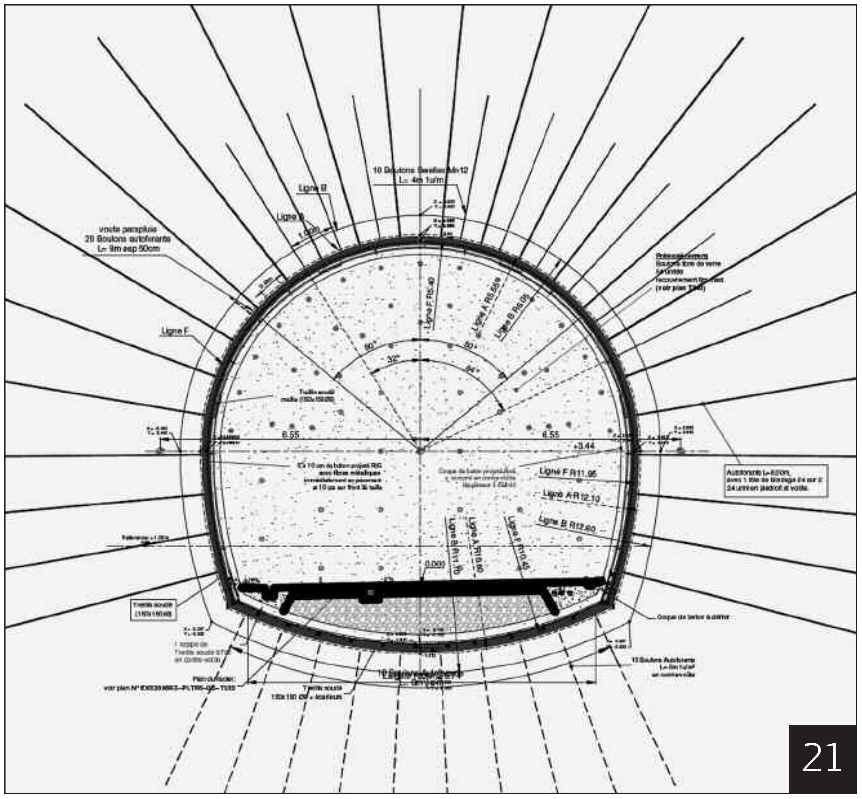
20. ILLUSTRATION OF THE SUPPORT SYSTEMS USED IN CARBONIFEROUS FORMATION ALONG THE SAINT MARTIN LA PORTE ACCESS ADIT

"PROFILE RIGIDE", DEPTH 180-250 M; "PROFILE 1", DEPTH 250-340 M; PROFILE 2", DEPTH 340-650 M



As shown in Figures 22 and 23, the excavation-construction sequence adopted is as follows:

- Stage 0: face reinforcement, including a ring of grouted fibre glass dowels around the tunnel perimeter, designed to reinforce the rock mass over a 2 to 3 m thickness.
- Stage 1: mechanical excavation carried out in steps of one meter length, with installation of un-tensioned rock anchors (length 8 m) along the perimeter, yielding steel sets with sliding joints, and a 10 cm thick shotcrete layer. The tunnel is opened in the upper cross section to allow for a maximum convergence of 600 mm.
- Stage 2: the tunnel is opened to the full circular section at a distance of 30 m from the face, with application of 20 cm shotcrete lining, yielding steel sets with sliding joints fitted with hiDCon elements. The tunnel is allowed to deform in a controlled manner so as to develop a maximum convergence which should not exceed 400 mm.
- Stage 3: installation of the coffered concrete ring at a distance of 80 m from the face.



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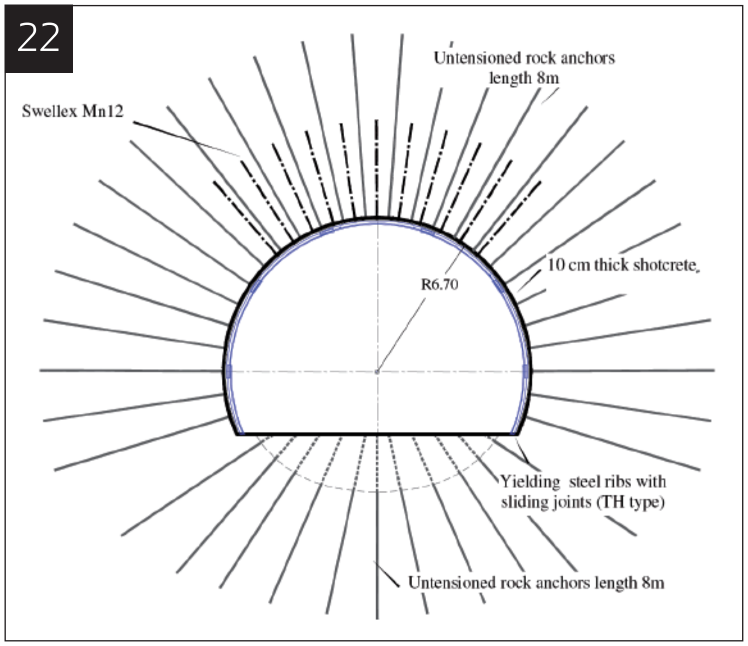
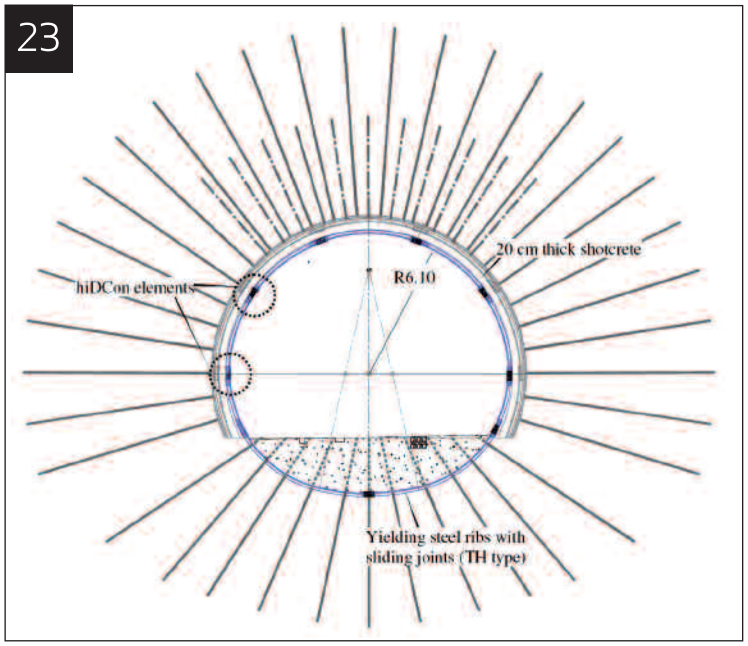
21. *YIELDING SUPPORT SYSTEM INITIALLY ADOPTED WITH HORSE-SHOW CROSS SECTION*

(SECTION P7 - "PROFIL 1" IN FIGURE 20)



22. *YIELD-CONTROL SUPPORT SYSTEM USED WITH NEAR CIRCULAR CROSS SECTION. STAGE 1*

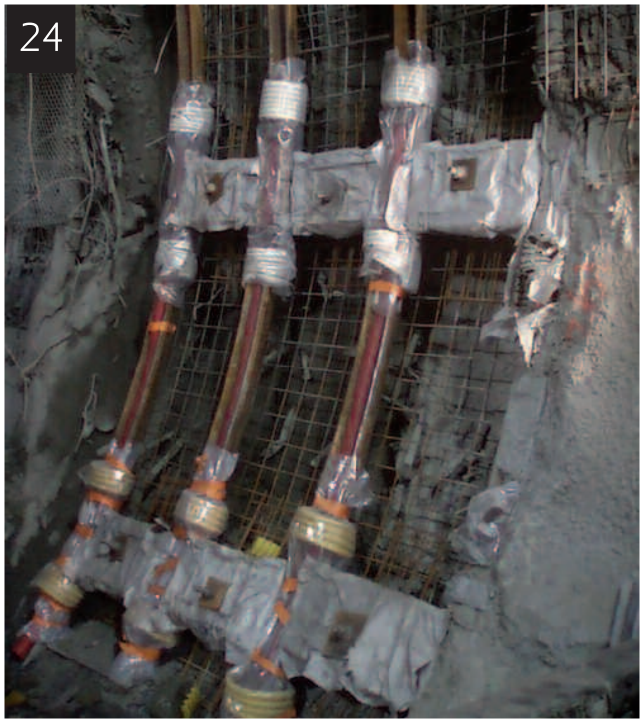
(SECTION DSM - "PROFIL 2" IN FIGURE 20; NOTE: FACE REINFORCEMENT NOT SHOWN)



23. *YIELD-CONTROL SUPPORT SYSTEM USED WITH NEAR CIRCULAR CROSS SECTION. STAGE 2.*

(SECTION DSM - "PROFIL 2" IN FIGURE 20; NOTE: FACE REINFORCEMENT NOT SHOWN)

As shown in Figure 24, the most important component of such a yielding support system is the hiDCon element. A total of 9 such elements (one in the invert) are installed in slots in the shotcrete lining between the steel sets. In the Saint Martin access adit the hiDCon elements have height of 40 cm, length of 80 cm, and thickness of 20 cm. They have been designed to yield at approximately 40-50 %, strain with a yield stress of 8.5 MPa. This means that with 9 elements installed, if one takes for simplicity a circular tunnel, under the assumption that each element may attain a 50 % strain, the maximum allowed radial displacement ΔR is equal to 20 cm approximately, resulting in a total tunnel convergence of 40 cm. Also, if one takes a yield stress of 8.5 MPa, the radial confinement stress on the surrounding rock results to be 0.3 MPa approximately.



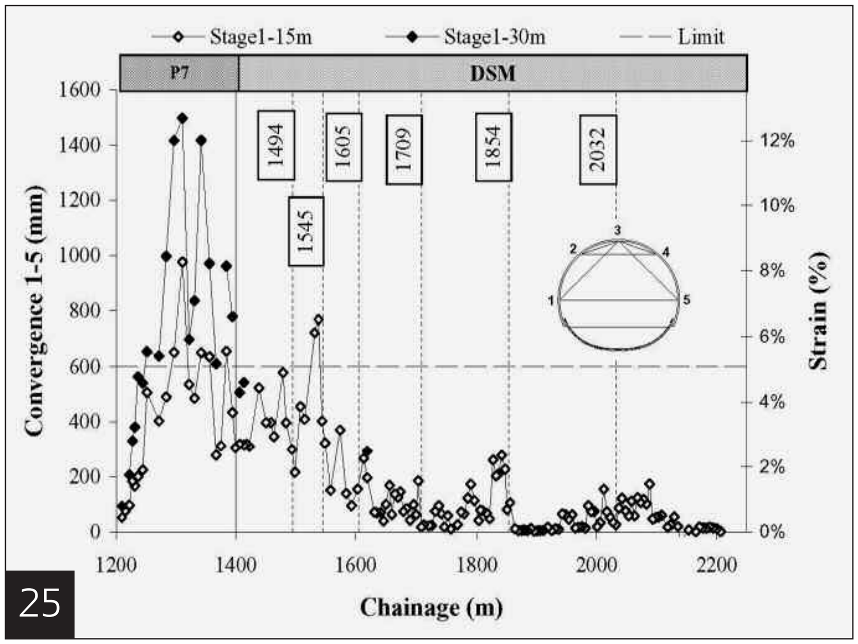
24. DETAILS OF THE HIDCON ELEMENTS INSTALLED BETWEEN THE SLIDING JOINTS OF THE STEEL SETS BEFORE SHOTCRETE PLACEMENT

6.3 Performance monitoring

Monitoring of tunnel convergence has been underway along the tunnel where the support system described above has been installed systematically, however with adaptations in the level of stabilization/support measures and the number of hiDCon elements used depending on the rock mass conditions encountered and the level of squeezing being experienced.

Convergences are measured by means of optical targets placed along the tunnel perimeter. A number of special sections have been equipped with multi-position borehole extensometers.

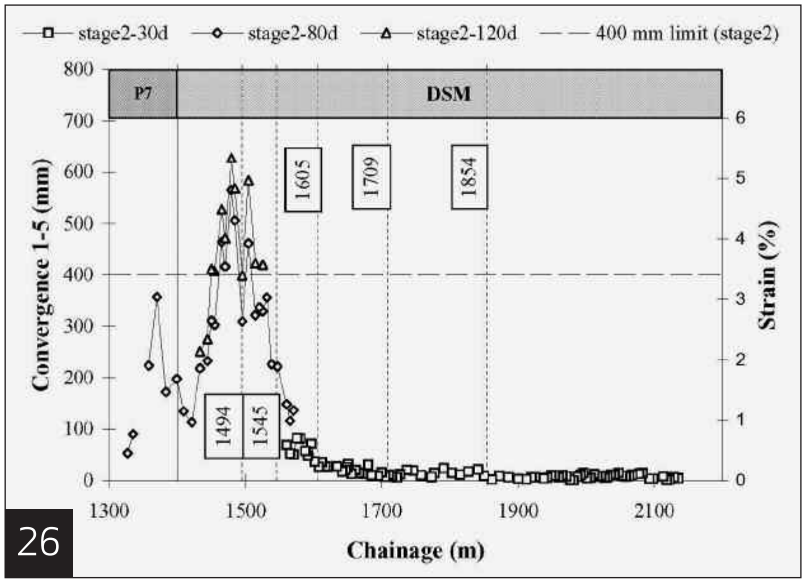
In addition, the strain/stress level in the primary and final linings is monitored. A few representative monitoring data will be described in the following.



25. CONVERGENCES MEASURED IN STAGE 1, 15 AND 30 M BEHIND THE FACE, BETWEEN CHAINAGE 1200 AND 2100 M APPROXIMATELY

Convergence monitoring

In order to gain in the understanding of the tunnel response, it is of interest to consider the diagram of Figure 25, which shows the convergences measured in stage 1, along array 1-5, between chainage 1100 m and 2200 m approximately, with the tunnel face being 15 m and 30 m ahead of the monitoring section. Also illustrated in Figure 25 is the tunnel "strain" that has occurred (i.e. the convergence divided by the length of the array 1-5 measured at the time of installation of the optical targets).

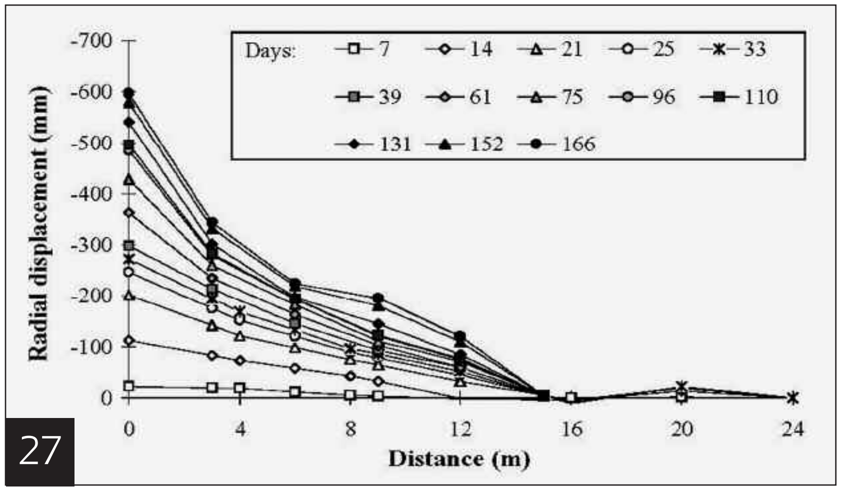


26. *CONVERGENCES MEASURED IN STAGE 2, 30, 80, AND 120 DAYS FROM CONVERGENCE TARGETS INSTALLATION, BETWEEN CHAINAGES 1200 AND 2100 M APPROXIMATELY*

The following observations can be made (Figure 25):
▸ the most significant convergences in the tunnel at a distance of 15 m behind the face occur between chainages 1230 and 1550 m approximately, with a maximum strain of 4-8 %;
▸ large deformations (up to 12 % strain measured 30 m behind the face) are associated with the horseshoe cross section (i.e. the "old" support installed, P7) between chainages 1230 and 1400 m;
▸ with the "new" cross section (DSM) installed systematically starting with chainage 1400 m, the mean tunnel strain measured 15 m behind the face is 4 % and locally never in excess of 6-7 %;
▸ the 1230-1550 m length is characterized in general by higher percentages of "weak" rocks and lower rock mass strength (2 MPa); following chainage 1550 m the ground conditions improved substantially (Figures 18 and 19);
▸ the 600 mm allowed convergence with the "new" cross section (DSM) has been exceeded locally which required re-profiling of the cross section before installing the yielding support in stage 2.
Of interest, in order to appreciate the tunnel performance and compare the "old" (P7) and "new" (DSM) excavation/support methods, is the diagram of Figure 26, which illustrates the convergences 30, 80 and 120 days following the opening of the full cross section in Stage 2.
The following remarks can be made (Figure 26):
▸ the convergences developed in the tunnel after 80 days of additional monitoring with the "old" (P7) support installed, attain a maximum of 370 mm (i.e. 3 % tunnel strain), which takes the total tunnel closure near to 2000 mm (2 m) around chainage 1370 m;
▸ comparatively, with the "new" (DSM) support installed in stage 2, the maximum convergence is 600 mm (i.e. 5 % tunnel strain) around chainage 1480 m, in excess with respect to the target value of 400 mm (i.e. 3.5 % tunnel strain).

Deformations in the rock surround

A number of special monitoring sections were installed along the adit. In these sections, in addition to convergence measurements also multi-position borehole extensometers were used in order to observe the rock mass response in the ground around the tunnel.
For the purpose of further illustration, the monitoring results for the section at chainage 1444 m are plotted in Figures 27 and 28. This section has exhibited during excavation a significant non symmetric closure, with severe over-stressing of the primary lining and of some of the deformable elements, mainly on the right sidewall.
A total of 6 multi-position borehole extensometers, each 24 m long, were installed in Stage 1, one at the invert and crown, and two on each sidewall, left and right. These were replaced by 15 m long borehole extensometers at the beginning of stage 2, 33 days after installation, approximately 30 m behind the tunnel face.



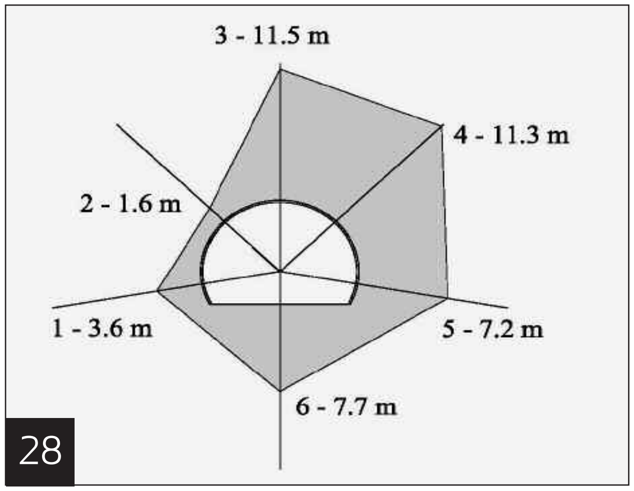
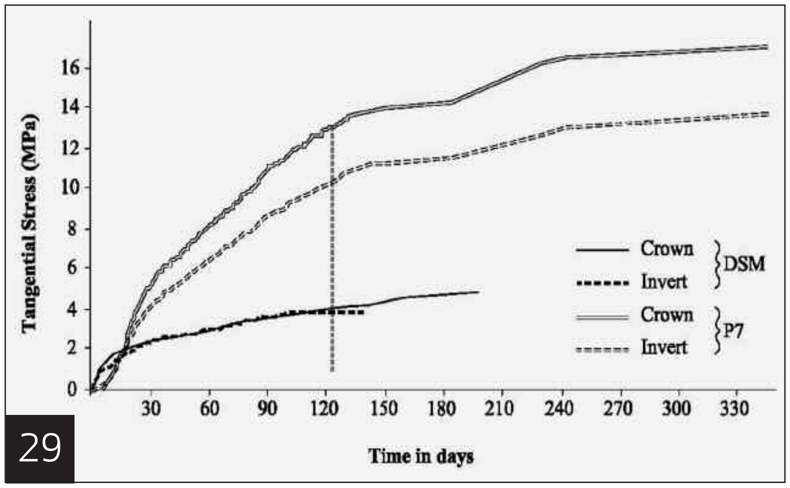


Figure 27 shows the typical displacement distribution versus time on the right side, whereas Figure 28 illustrates the zone around the tunnel where the radial strain is greater than 1%, this strain limit being taken as the onset of squeezing behavior.

It is clear that, where a more significant extent of the mobilised zone around the tunnel occurs in Stage 1 (Figures 27 and 28), the support system in Stage 2 undergoes greater deformations. The non symmetric response of the tunnel is due to the essentially anisotropic features of the rock mass, the presence of "strong" (sandstones and schists) and "weak" (coal and clay-like shales) "layers" which dip from the left to the right of the tunnel cross section.

Tangential stress in the lining

Also monitored in a number of special sections was the tangential stress in the final lining for both the "old" (P7) and the "new" (DSM) cross sections. Typical results available are plotted in Figure 29 where the measured tangential stress is given versus time both at the crown and invert. It is clearly shown that indeed the DSM cross section is undergoing a level of stress which is much smaller than that of the P7 cross section.



6.4 Laboratory tests

Table 1 summarises the results of laboratory tests derived from unconfined and triaxial compression tests performed on

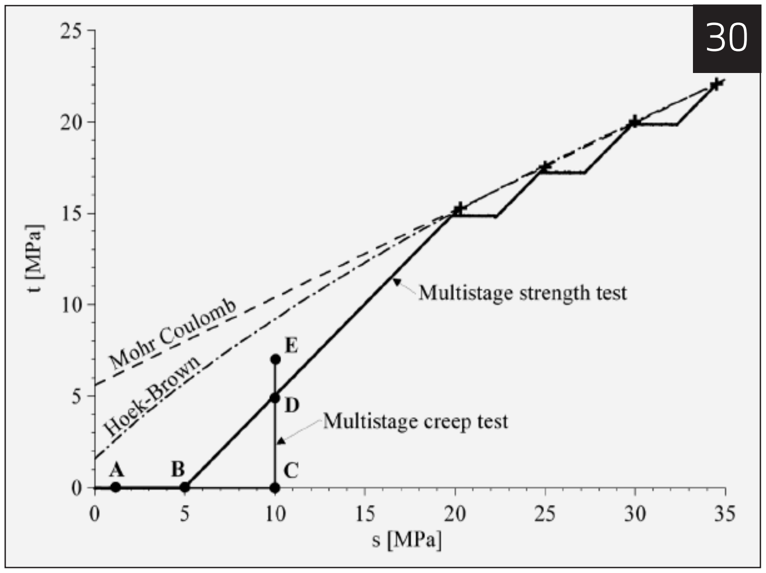
Parameter	Sandstones	Schists
Unit weight (γ, kN/m^3)	27.1	26.1
Unconfined compressive strength (σ_{ci}, MPa)	20.3	16.1
Hoek-Brown constant (m_i)	6.4	4.3
Tangent elastic modulus (E_t, GPa)	21.8	19.5

27. *SPECIAL MONITORING SECTION AT CHAINAGE 1444 M RADIAL DISPLACEMENT VERSUS TIME AROUND THE TUNNEL*

28. *SPECIAL MONITORING SECTION AT CHAINAGE 1444 M ZONE AROUND THE TUNNEL WHERE THE RADIAL STRAIN IS GREATER THAN 1%*

29. *TANGENTIAL STRESSES AT CROWN AND INVERT VERSUS TIME IN FINAL LINING CROSS SECTIONS P7 AND DSM*

TABLE 1. *PARAMETERS DERIVED FROM LABORATORY TESTS ON SANDSTONES AND SCHISTS*



the "strong" rock components (sandstones and schists). As far as the "weak rock" components (coal, clay-like shales and cataclastic rock) most of the attention in testing has focused on the mechanical properties of coal and its time dependent behaviour in triaxial conditions. A detailed description of the laboratory tests performed and of the results obtained is given by Debernardi (2008) in his PhD thesis.

In particular, multi-stage triaxial tests were carried out on coal with the intent to determine its deformability and strength properties. Four compression stages were performed with different confining pressures of 5, 7.5, 10 and 12.5 MPa, by applying a constant axial strain rate of 0.01 %/min, until a peak was reached in each stage on the stress-strain curve.

At peak, the confining pressure was increased in order to allow for the determination of a new peak strength. This process was repeated four times to determine four points on the failure line as shown in Figure 30. The deformability and strength parameters are given in Table 2.

Also performed were a series of creep and relaxation tests on coal under different laboratory conditions (Barla et al. 2007; Debernardi, 2008; Barla et al., 2008). In particular, according to the stress paths shown in Figure 30, creep phases were performed in isotropic conditions (points A, B and C) and under a deviatoric stress (points D and E).

The creep curves for the stress state D and E are shown in Figure 31. It is observed that a secondary creep deformation is soon attained with a moderate strain rate which is strictly related to the limited level of the mobilised strength and of the mean stress being applied.

30. *STRESS PATH OF THE MULTI-STAGE TRIAXIAL TEST AND STRENGTH ENVELOPES OF COAL STRESS PATH OF THE MULTI-STAGE TRIAXIAL CREEP TESTS*

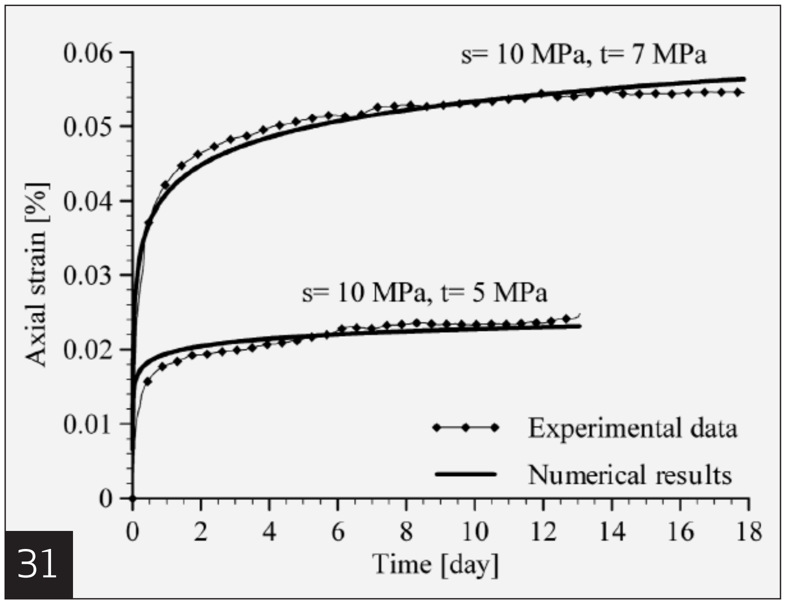
TABLE 2. *DEFORMABILITY AND STRENGTH PARAMETERS DERIVED FROM LABORATORY TESTS ON COAL*

31. *RESULTS OF MULTISTAGE TRIAXIAL CREEP TESTS ON COAL*
CALIBRATION WITH SHELVIP MODEL

6.5 Numerical modelling

In squeezing conditions time dependent deformations are observed whenever face advance is stopped and are likely to

Parameter	
Tangent elastic modulus (E_t, GPa)	6.9
Mohr-Coulomb parameters (c, MPa) (φ, °)	6.4 28.5
Hoek-Brown parameters Unconfined compressive strength (σ_{ci}, MPa)	15.3
Hoek-Brown constant (m_i)	8.97



take place during excavation, when it is difficult to distinguish the "face effect" from the "time effect". Therefore, in such conditions, an appropriate representation of the tunnel response is obtained only by using constitutive models which account for time dependent behavior (Barla et al., 2007).
Many constitutive models have been proposed to describe such a behavior for weak rock and soil. Nevertheless, only few models can reproduce satisfactorily all the time-dependent features involved in tunnel excavation, with a reasonably simple mathematical formulation to be used in design practice. With the phenomena observed in the Saint Martin La Porte access adit in mind, the SHELVIP (Stress Hardening ELasto VIscous Plastic) model was formulated (Debernardi, 2008; Debernardi and Barla, 2009).

SHELVIP, a new constitutive model
The SHELVIP model is derived from the Perzyna's overstress theory, by adding a time independent plastic component. Therefore it is possible to split the strain rate tensor $\dot{\varepsilon}_{ij}$ into elastic $\dot{\varepsilon}_{ij}^{e}$, plastic $\dot{\varepsilon}_{ij}^{p}$, and viscoplastic $\dot{\varepsilon}_{ij}^{vp}$ components, to give:

$$\dot{\varepsilon}_{ij} = \dot{\varepsilon}_{ij}^{e} + \dot{\varepsilon}_{ij}^{p} + \dot{\varepsilon}_{ij}^{vp}$$

According to the classical theory of elastoplasticity, the time-independent plastic strains ε_{ij}^{p} develop only when the stress point reaches the plastic yield surface $f_p = 0$ (Figure 32), defined by the Drucker-Prager criterion:

$$f_p = q - \alpha_p \cdot p - k_p$$

The plastic strains ε_{ij}^{p} can be evaluated by using the classical flow rule of elastoplasticity:

$$\varepsilon_{ij}^{p} = \lambda \cdot \frac{\partial g_p}{\partial \sigma_{ij}}$$

where g_p is the plastic potential function, $g_p = q - \omega_p \cdot p$, that defines the direction of ε_{ij}^{p}, is the plastic dilatancy and λ is the plastic multiplier, that can be determined using the consistency condition $df_p = 0$, $f_p \leq 0$.
The viscoplastic strain rates $\dot{\varepsilon}_{ij}^{vp}$ develop only if the effective stress state exceeds a viscoplastic yield surface $f_{vp} = 0$ (Figure 32) which is also defined by the Drucker-Prager criterion. This surface is internal to the plastic yield surface and intersects the p -axis at the same point as the plastic yield surface. Thus, it is possible to write:

$$f_{vp} = q - \alpha_{vp} \cdot \left(p + \frac{k_p}{\alpha_p} \right)$$

where α_{vp} is a visco-hardening parameter that defines the internal viscous state of the material.
The viscoplastic strain rates $\dot{\varepsilon}_{ij}^{vp}$ strain rate can be determined using the flow rule of

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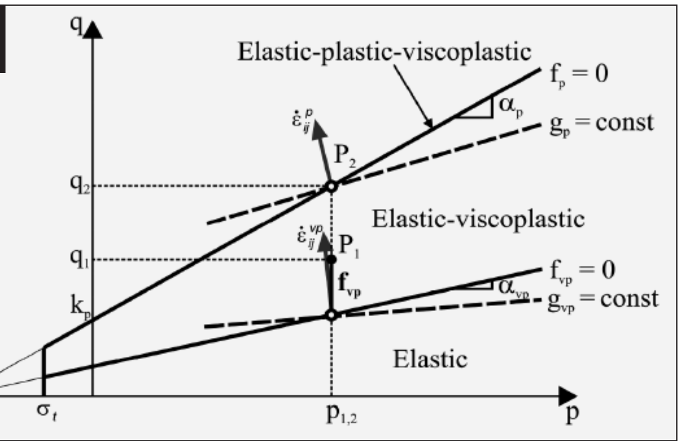


32. SHELVIP CONSTITUTIVE MODEL
VISCOPLASTIC AND PLASTIC YIELD SURFACES

Perzyna's overstress theory:

$$\dot{\varepsilon}_{ij}^{vp} = \gamma \cdot \Phi\left(\langle F \rangle\right) \cdot \frac{\partial g_{vp}}{\partial \sigma_{ij}}$$

The overstress function F is assumed to be equal to the viscoplastic yield function f_{vp} and the viscous nucleus Φ to be a power law:

$$\Phi = \langle F \rangle^n = \langle f_{vp} \rangle^n$$

where n is a constitutive parameter. The viscoplastic potential function g_{vp} is assumed to be $g_{vp} = q - \omega_{vp} \cdot p$, where ω_{vp} is the viscoplastic dilatancy.
The hardening of the viscoplastic yield surface is governed by the differential equation:

$$\dot{\alpha}_{vp} = \frac{l}{m \cdot n} \cdot \frac{f_{vp}}{p + k_p/\alpha_p} \cdot \left(\frac{f_{vp}}{q}\right)^{n \cdot m}$$

where m and l are constitutive parameters.
The introduction in SHELVIP of a stress based hardening law is associated with some advantages.
It allows one to evaluate the viscoplastic hardening level from the stress level which defines the threshold for development of viscoplastic deformations, which can be done by appropriate tests.
In addition, as demonstrated in detail in Debernardi and Barla (2009), it permits a clear definition of each time-dependent behavioural feature by means of a single constitutive parameter. One is to note that the overall number of parameters in the model are 11 as summarized in Table 3: 2 elastic parameters, 4 plastic parameters, and 5 viscoplastic parameters.
The SHELVIP model is shown to be effective in describing the time dependent behaviour of the "weak" rock components as demonstrated with the calibration of several triaxial creep tests performed on coal samples. As an example, Figure 31 (see above) shows a comparison between the experimental and nu-

TABLE 3. *CONSTITUTIVE PARAMETERS OF THE SHELVIP MODEL*

Elastic Behaviour	E	Young's modulus
	ν	Poisson's ratio
Plastic Behaviour	α_p	Slope of the Drucker-Prager's plastic yield criterion
	k_p	Intercept of the Drucker-Prager's plastic yield criterion
	σ_t	Volumetric tension cut-off
	ω_p	Plastic dilatancy
Viscoplastic Behaviour	γ	Fluidity parameter
	m	Shape factor
	n	Load dependency factor
	l	Time stretching factor
	ω_v	Viscoplastic dilatancy

TABLE 4. *PARAMETERS FOR THE SHELVIP MODEL LABORATORY CONDITIONS FOR COAL*
*(*TIME IN DAYS AND PRESSURE IN KPA)*

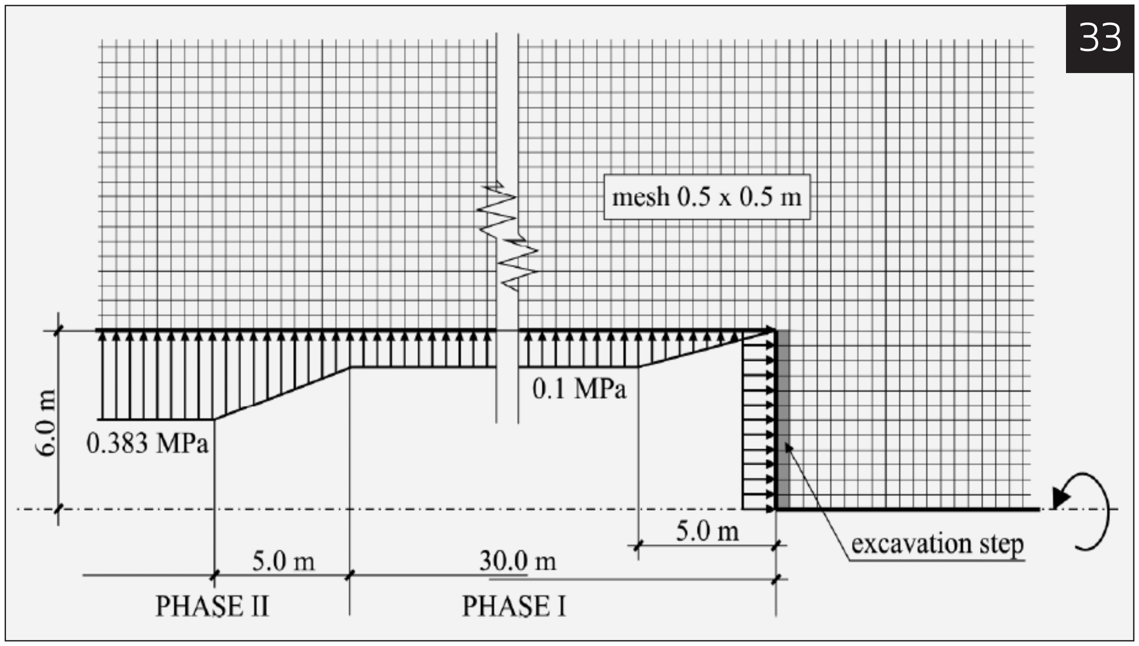
Parameter	
E (GPa)	5.00
ν (-)	0.30
(φ, °)	34.29
c (MPa)	3.52
α_p	1.39
k_p (MPa)	7.16
σ_t (MPa)	0.10
ω_p	0.00
γ (*)	1.2 E-15
m (*)	1.006
n (*)	3.884
l (*)	2.30 E+05
ω_{vp} (*)	0.735

merical results obtained from the multistage creep tests on coal described above. As noted, the agreement results to be satisfactory with the material parameters of the constitutive model shown in Table 4 below.

Modelling of the tunnel excavation

The SHELVIP constitutive model has been used to analyse the tunnel response in terms of convergence monitored during excavation (Barla et al., 2009).
For the purpose of numerical modeling the DSM cross sections between chainage 1394 m and 1527 m have been chosen where the full face with reinforcement coupled with the yield-control support system was used.
The overburden is approximately 363 m and the initial stress state is assumed to be isotropic and equal to 9.8 MPa.
The analyses have been performed with the Finite Difference Method and FLAC code. Axisymmetric conditions have been adopted in order to reproduce the three-dimensional influence of the tunnel face, which is known to play a significant role in squeezing conditions.
The tunnel cross section is assumed to be circular, with an equivalent radius of 6 m. The total size of the model (96 m - 280 m) is very large in order to minimize the boundary effects that are very significant in the case of large deformations.
Particular attention has been paid to the chronological sequence of excavation. The ground reinforcement ahead of the face has been described by using an equivalent pressure of 0.1 MPa applied to the face (Figure 33).
IThe radial reinforcement and the first stage lining (10 cm thickness) have been simulated by using an equivalent radial pressure, which has been assumed to reach the constant value of 0.1 MPa 5 m behind the tunnel face, as shown in Figure 33. The influence of the second stage lining has been introduced by way of an additional inter-

33. SKETCH OF THE AXI-SYMMETRIC NUMERICAL MODEL

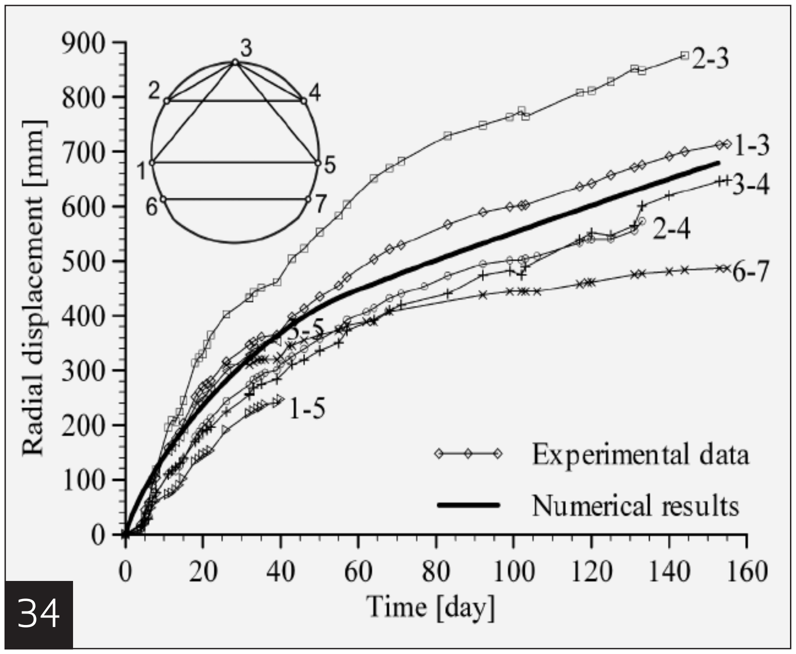


nal pressure 30 m behind the face, which reaches the constant value of 0.383 MPa in a 5 m span. This value has been calculated on the basis of the yielding stress of the hiDCon elements, as determined at the laboratory scale. A constant excavation rate of 0.54 m/day has been considered.

Back-analysis has led to the values of the constitutive parameters listed in Table 5.

Figure 34 shows the comparison of computed and measured values in terms of radial displacement for the section at chainage 1444 m.

The agreement of the numerical results with the mean curve is rather satisfactory, notwithstanding the scattering of the monitoring data due to the high heterogeneity and anisotropy of the rock mass. Also the displacements around the tunnel monitored with the multi-position borehole extensometers are satisfactorily reproduced, as shown in Figure 35.



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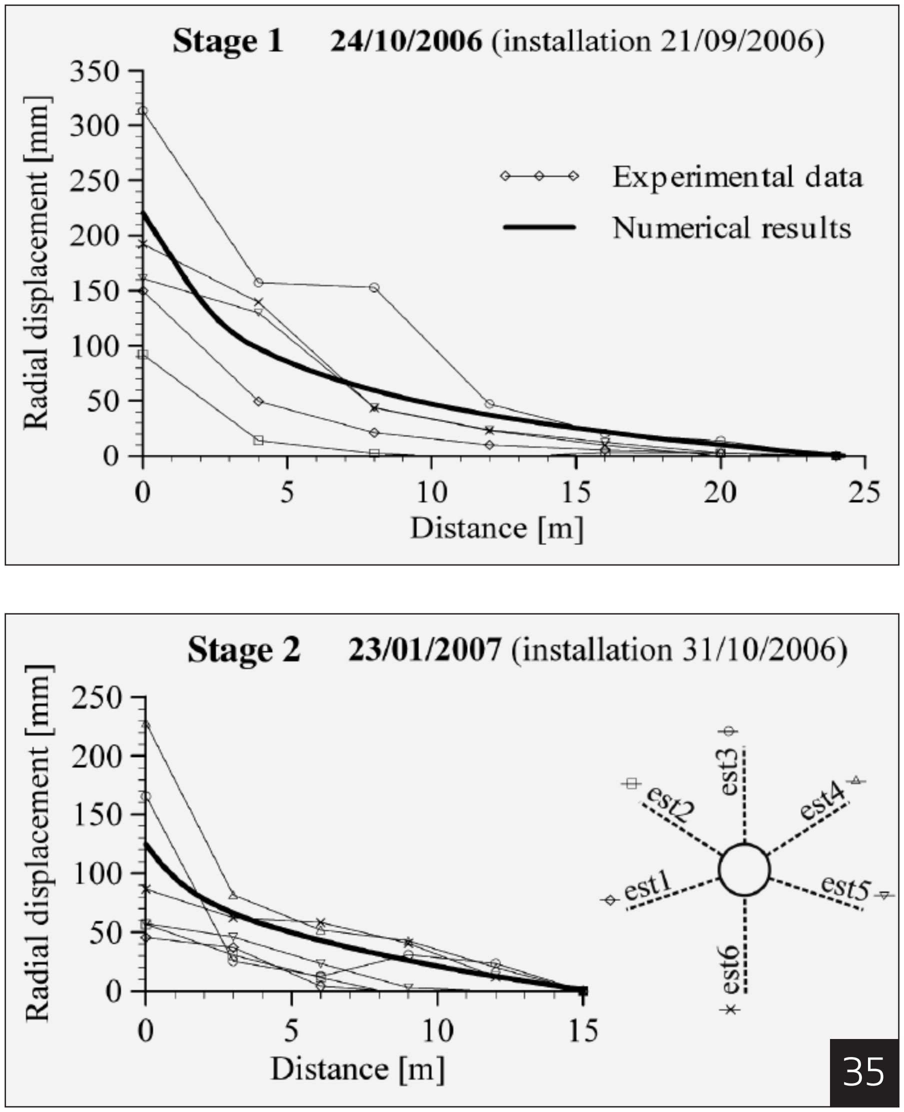

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34. *COMPUTED VERSUS MONITORED CONVERGENCES AT CHAINAGE 1444 M*

35. *COMPUTED VERSUS MONITORED RADIAL DISPLACEMENTS AROUND THE TUNNEL AT CHAINAGE 1444 M*

TABELLA 5. *PARAMETERS FOR THE SHELVIP MODEL IN SITU CONDITIONS*

(TIME IN YEARS AND PRESSURE IN KPA)*

Parameter	
E (GPa)	0.64
ν (-)	0.30
(φ, °)	26
c (MPa)	0.56
α_p	0.10
k_p (MPa)	7.16
σ_t (MPa)	0.10
ω_p	0.00
γ (*)	5.1 E-5
m (*)	2.20
n (*)	0.18
l (*)	0.01
ω_{vp} (*)	0.735

7. CONCLUSIONS

This lecture deals with the studies performed during excavation of the Saint Martin La Porte access adit (Lyon-Turin Base Tunnel), which experienced squeezing conditions during excavation in the Carboniferous Formation. An innovative tunnel excavation and construction method was introduced which couples face reinforcement by means of fibre glass dowels with a yield-control support. In situ performance monitoring, laboratory tests, and numerical modelling are described.

It is to point out that the results obtained so far through the studies performed and briefly reported in this lecture are thought to be essential in taking the decision if, where, to what extent, and by what measures can mechanized tunnelling be adopted for excavation of the Base Tunnel in conditions similar to those experienced in the Saint Martin La Porte access adit. It is thought that with the level of information available and the design tools provided a thorough analysis can be performed in order to assist in taking such a decision.

ACKNOWLEDGEMENTS

The opportunity to prepare this Invited Lecture which adds to the content of a Keynote Lecture recently prepared for Eurock 2009, has made it possible to summarize some of the research work on "squeezing rock", with the Saint Martin La Porte access adit as a case study, carried out on behalf of LTF (Lyon Turin Ferroviaire SAS) through a research contract (in the years from 2006 to 2009) between the Engineering Group "Egis, Alpina, Antea" and Politecnico di Torino, Department of Structural and Geotechnical Engineering. The author wishes to acknowledge the work of Dr Marco Barla, Mariacristina Bonini, and Daniele Debernardi who have contributed and are contributing very significantly to this research effort.

BIBLIOGRAPHY

ALAGNAT, D. 2005. La méthode observationnelle pour le dimensionnement interactif des ouvrages. Presses de l'école nationale des Ponts et chaussées. 326 pp.

BARLA, G. 1995. Squeezing rocks in tunnels. ISRM News Journal, no. 3/4: 44-49.

BARLA, G. 2005. Sviluppi nell'analisi progettuale delle opere in sotterraneo. Quinta Conferenza "Arrigo Croce". Rivista Italiana di Geotecnica, vol. XXXIX, no. 3: 11-67.

BARLA, G., BONINI, M. AND DEBERNARDI, D. 2007. Modelling of tunnels in squeezing rock. EURO:TUN 2007. 1st Conference on Computational Methods in Tunnelling, Vienna.

BARLA, G., BONINI, M., AND DEBERNARDI, D. 2008. Time dependent deformations in squeezing tunnels. Proc. of the 12th International Conference of IACMAG, Goa.

BARLA, G., DEBERNARDI, D., AND STERPI D. 2009. Numerical analysis of tunnel response during excavation in squeezing rock by using two constitutive models. EURO:TUN 2009. 2nd Conference on Computational Methods in Tunnelling, Bochum.

BARLA, G. 2009. Innovative tunnelling construction method to cope with squeeezing at Saint Martin La Porte access adit (Lyon-Torino Base Tunnel). Rock engineering in difficult conditions in soft rocks and karst. Proc. of Eurock 2009. ISRM Regional Symposium. Dubrovnik, Croatia, (to be published).

CANTIENI, L. AND ANAGNOSTOU, G. 2009. The Interaction between yielding supports and squeezing ground. Tunnelling and Underground Space Technology, vol. 24: 309-322.

DEBERNARDI, D. 2008. Viscoplastic behaviour and design of tunnels. PhD Thesis. Doctoral Degree in Geotechnical Engineering. Politecnico di Torino: 302 pp.

DEBERNARDI, D. AND BARLA, G. 2009. New viscoplastic model for design analysis of tunnels in squeezing conditions. Rock Mech Rock Engng, vol. 42, no. 2: 259-288.

GEOMECHANICS AND TUNNELLING. 2009. Site Reports. vol. 2, no. 1: 117-118.

HOEK, E., 2001. Big tunnels in bad rock. Terzaghi Lecture. ASCE Journal of Geotechnical and Geoenvironmental Engineering, vol. 127, no. 9, September 2001: 762-740.

HOEK, E. AND MARINOS, P. 2000. Predicting tunnel squeezing problems in heterogeneous rock masses. Tunnels and Tunnelling International, Part 1, 32 (11): 45-51; Part 2, 32 (12): 33-36.

HOEK, E. 2006. Practical rock engineering. (available on: www.rocscience.com)

HOEK, E. AND GUEVARA, R. 2009. Overcoming squeezing in the Yacambú-Quibor Tunnel, Venezuela. Rock Mech Rock Engng, vol. 42, no. 2: 389-418.

KOVARI, K. 1998. Tunnelling in squeezing rock. Tunnel no. 5:12-31.

KOVARI, K. 2005. Method and device for stabilizing a cavity excavated in underground construction. US Patent Appl. 20050191138.

LUNARDI, P. 2000. Design and constructing tunnels-ADECO-RS approach. Tunnels and Tunnelling International (Special supplement in conjunction with Rocksoil spa).

SCHUBERT, W. 1996. Dealing with squeezing conditions in alpine tunnels. Rock Mech Rock Engng, vol. 29, no. 3: 145-153.

SCHUBERT, W., MORITZ, D., SELLNER, P., AND BLÜMEL, M., 1999. Tunnelling in squeezing ground recent improvements. Felsbau 17 (1): 56-58.

THUT, A., NATEROP, D., STEINER, P., & STOLZ, M. 2006. Tunnelling in squeezing rock-yielding elements and face control. 8th International Symposium on Tunnel Construction and Underground Structures, Lubljana, Slovenia.

Giovanna Cassani

30 YEARS OF ROCKSOIL

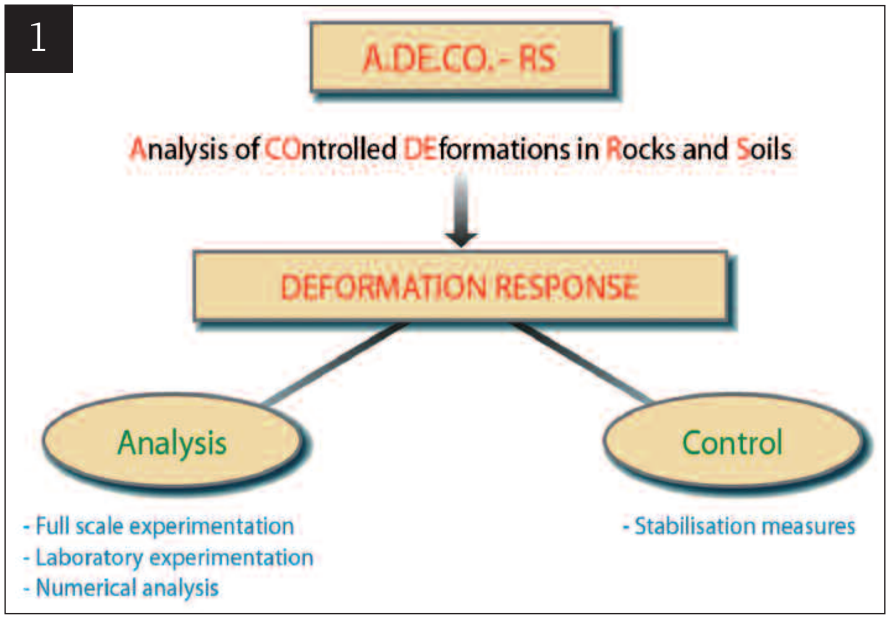
DOTT. ING. GIOVANNA CASSANI,
ROCKSOIL S.P.A., TECHNICAL DIRECTOR

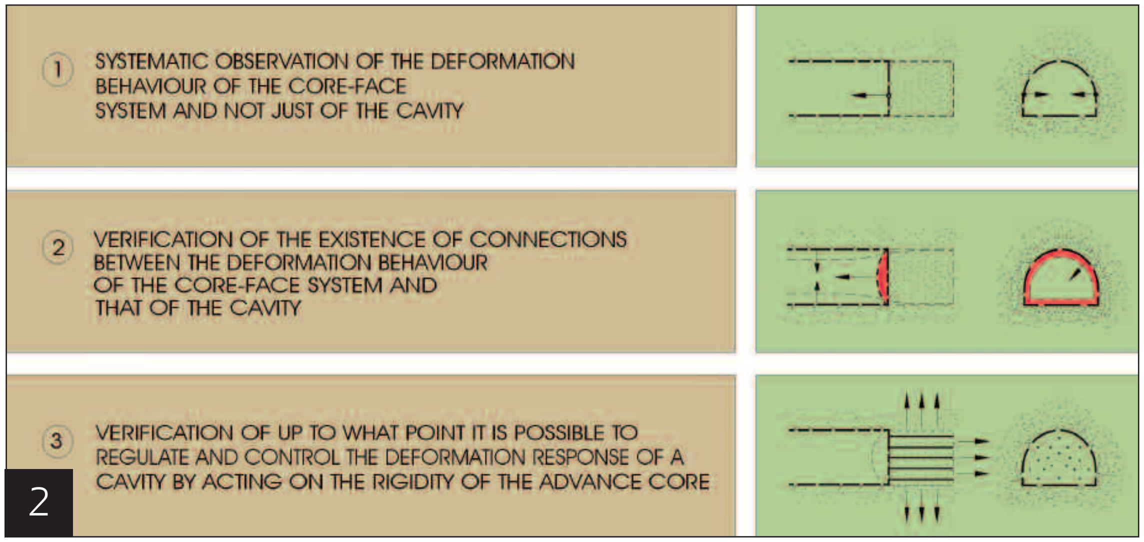
Good afternoon everybody. Please allow me too to steal a few minutes of your time to look back together with you over the work that Rocksoil has done over the last thirty years and to tell you of a story which began years ago and which with the help of all my colleagues, is still continuing each day with equal passion and commitment.

As Prof. Lunardi told you this morning, a long path of research was triggered by his first insights during the work on the Frejus Tunnel, which led step by step to the formulation of the ADECO–RS approach (Figure 1), as it is known today and widely used as an approach to the design and construction of tunnels. I do not want to go into the details of the approach, which everybody knows and which was also briefly illustrated this morning by Fulvio Tonon, but I wish on this occasion to look back over some the basic steps taken in its formulation.

As Prof. Lunardi already mentioned this morning, ADECO is the result of years of research and observation in tunnels, which allowed the following to be achieved (Figure 2):

- firstly to examine and catalogue the deformation phenomena that manifest during excavation both at the face and in the tunnel;
- then to find a correlation between the stress-strain behaviour of the core and the face and that of the cavity, dividing the deformation response of the medium to excavation into extrusion, preconvergence and convergence;
- finally, to understand how the excavation core can and must be used as a means to control the deformation response of the medium and therefore as a cavity stabilisation tool.





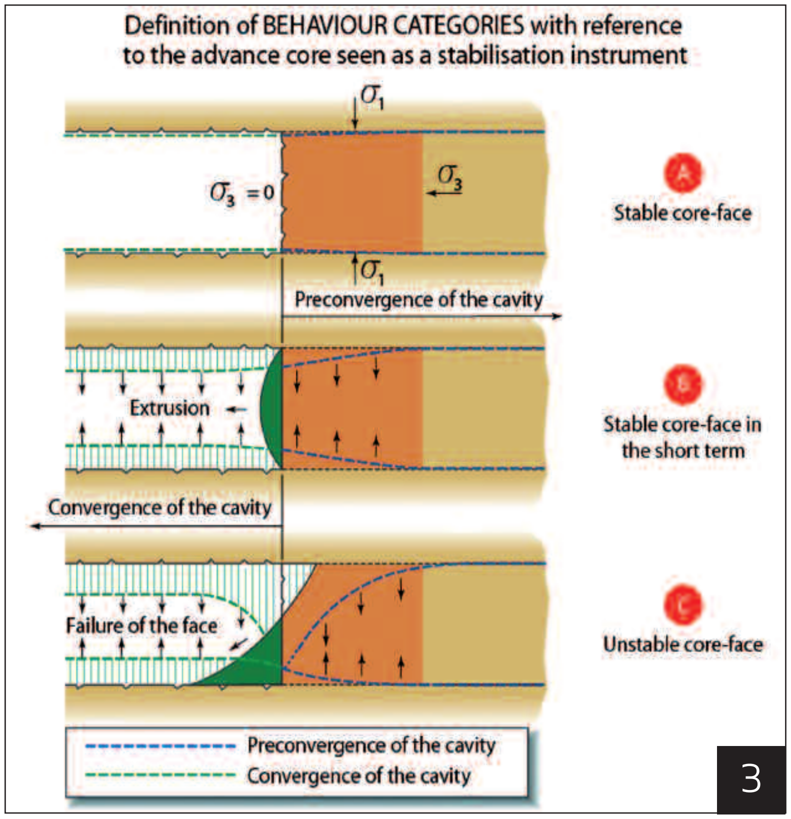
***1.** A LONG RESEARCH PROJECT WAS TRIGGERED DURING THE CONSTRUCTION OF THE FREJUS TUNNEL, WHICH LED STEP BY STEP TO THE FORMULATION OF THE ADECO–RS APPROACH*

***2.** THE ADECO-RS APPROACH INVOLVES THE CONTINUOUS COLLATION AND INTERPRETATION OF DATA ON THE DEFORMATION OF THE CORE AND THE CAVITY DURING TUNNEL ADVANCE*

This theoretical and experimental approach, which involves the continuous collation and interpretation of data on the deformation of the core-face and the cavity during tunnel advance, still today constitutes the driving force behind the company and it is one of its most distinctive features. It is made possible by the continuous presence of our engineers in tunnels.

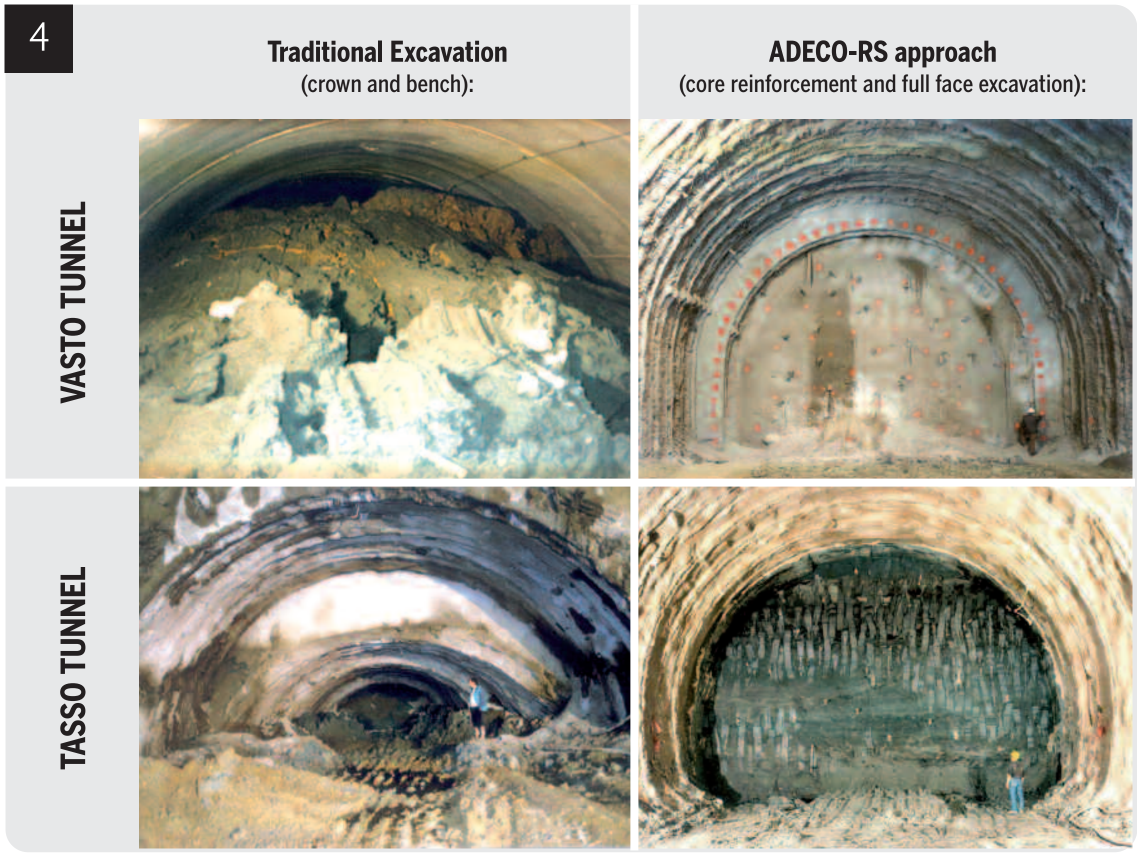
The following behaviour categories for the response to excavation were defined by working on this first experimental data (Figure 3):

- category A, core-face stable;
- category B, core-face stable in the short term;
- category C, core-face unstable.



And all the technologies required to act on the rigidity of the core-face and to control the effectiveness of the operations performed (conservative intervention and extrusion measurements) were then conceived of and developed.

Briefly, various projects allowed as to test and experiment with the approach in the field between the end of the 1980s and the middle of the 1990s. I am speaking for example of the Tasso, Vasto and San Vitale tunnels, where Rocksoil was brought in because it was impossible to continue excavation using what were then normal methods. The main innovations introduced were full face excavation and ground improvement of the core, together with casting the tunnel invert at a short distance from the face. In fact advance in all these tunnels was successfully resumed and completed on schedule by reducing the extrusion surface at the face and acting decisively on the rigidity of the core (Figure 4).

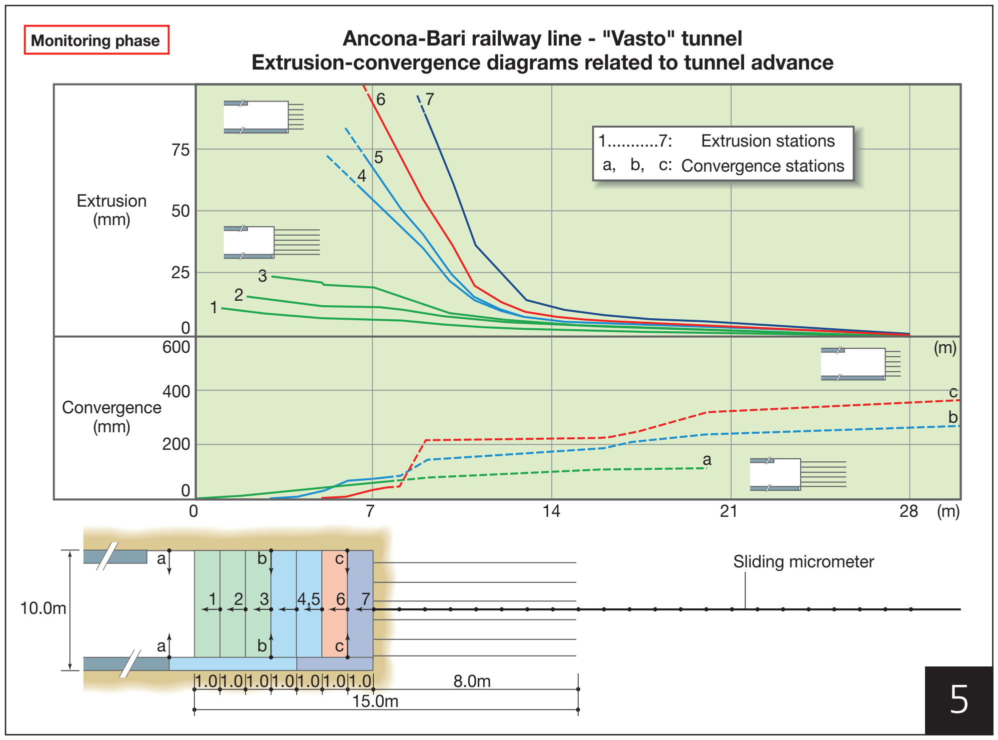


3. *THE CATEGORIES OF EXCAVATION BEHAVIOUR*

4. *THE APPLICATION OF THE ADECO-RS APPROACH MADE IT POSSIBLE TO COMPLETE TUNNELS WHICH SEEMED IMPOSSIBLE WITH OTHER SYSTEMS*

Extrusion measurements were introduced for the first time on these tunnels together with the more conventional convergence measurements. Together they were used to verify the basic principles of the approach and to control the deformation response of the ground by regulating the number of reinforcement elements in the face, their length and overlap and the distance from the face at which linings were cast. (Figure 5). This stage of the research, which as already said was conducted in the middle of the 1990s, produced a fundamental result. It confirmed that by taking action to control deformation, the stresses on linings were not excessive, as feared by many, but extremely limited and in any case well below the strength of the materials.

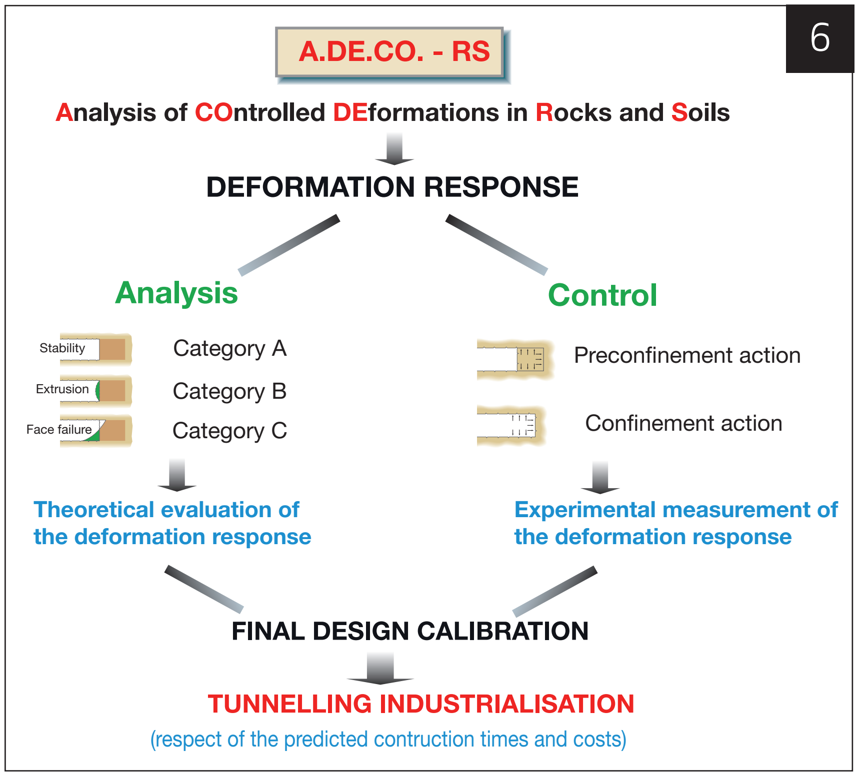
This was in the middle of the 1990s and the formulation of ADECO as an approach to the design and construction of tunnels was in fact completed. The analysis and control phases were quite distinct and the concept of variability in the preconfinement and confinement operations and in the fine tuning of them during construction was introduced. This was done by formulating a construction system that was as simple and linear as possible with no redundancy, fully able to ensure that construction time and cost objectives were met, to the point where underground construction could be defined as an industrial process (Figure 6).



5. EXTRUSION-CONVERGENCE DIAGRAMS AS A FUNCTION OF TUNNEL ADVANCE

It is now an established fact, both experimentally and numerically, that three dimensional analysis of the stress-strain state of the rock mass during tunnel excavation leads to assessments of both deformation and stresses that are very different from those obtained using classic plane models. Therefore it is absolutely indispensible to tackle the subject in three dimensions for the proper design of a tunnel.

Simple plane models can nevertheless be used to illustrate the concept of deformation control which underlies the ADECO method. The characteristic line of the cavity expresses the relationship between the radial pressure p of the ground and the radial movement u along the profile of the excavation. Together with the characteristic line of the lining, it can be used to assess the pressure acting on the lining and the level of deformation reached, which varies according to the type of behaviour of the ground (A – elastic, B – elastic–plastic, C – plastic) and the type of lining used (Figure 7).
The choice of the optimal point of intersection between the characteristic line of the cav-



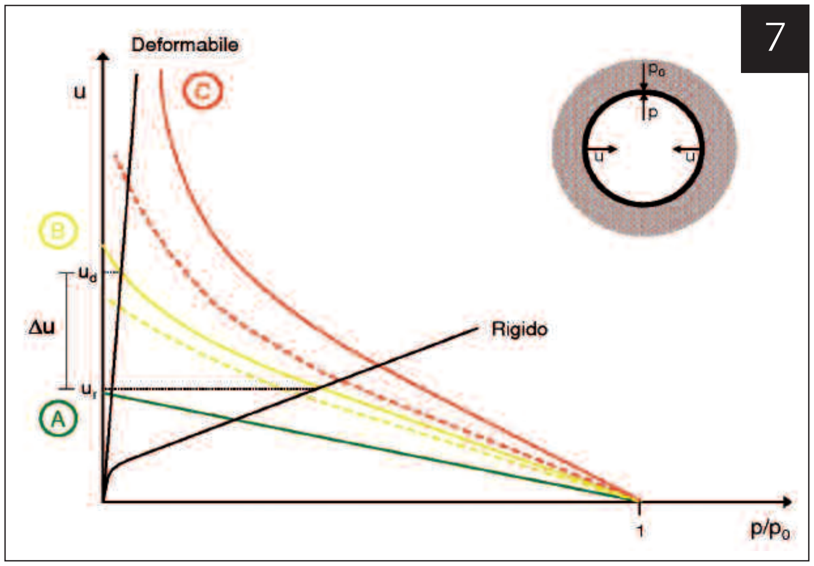
ity and that of the lining is the most delicate point in the design of a tunnel. It is in fact clear that in materials with behaviour type C (unstable), the use of a preliminary lining that is too deformable may result in not finding a point of equilibrium and the collapse of the tunnel. Consideration of the three dimensional effects, as described by Prof. Anagnostou this morning, leads to a more realistic estimate of both the pressures and the convergence and it has been demonstrated that the rigidity of the preliminary lining and the distance from the face at which it is installed modify the characteristic line of the cavity. This is more significant, the more the behaviour of the ground is of type C, unstable.

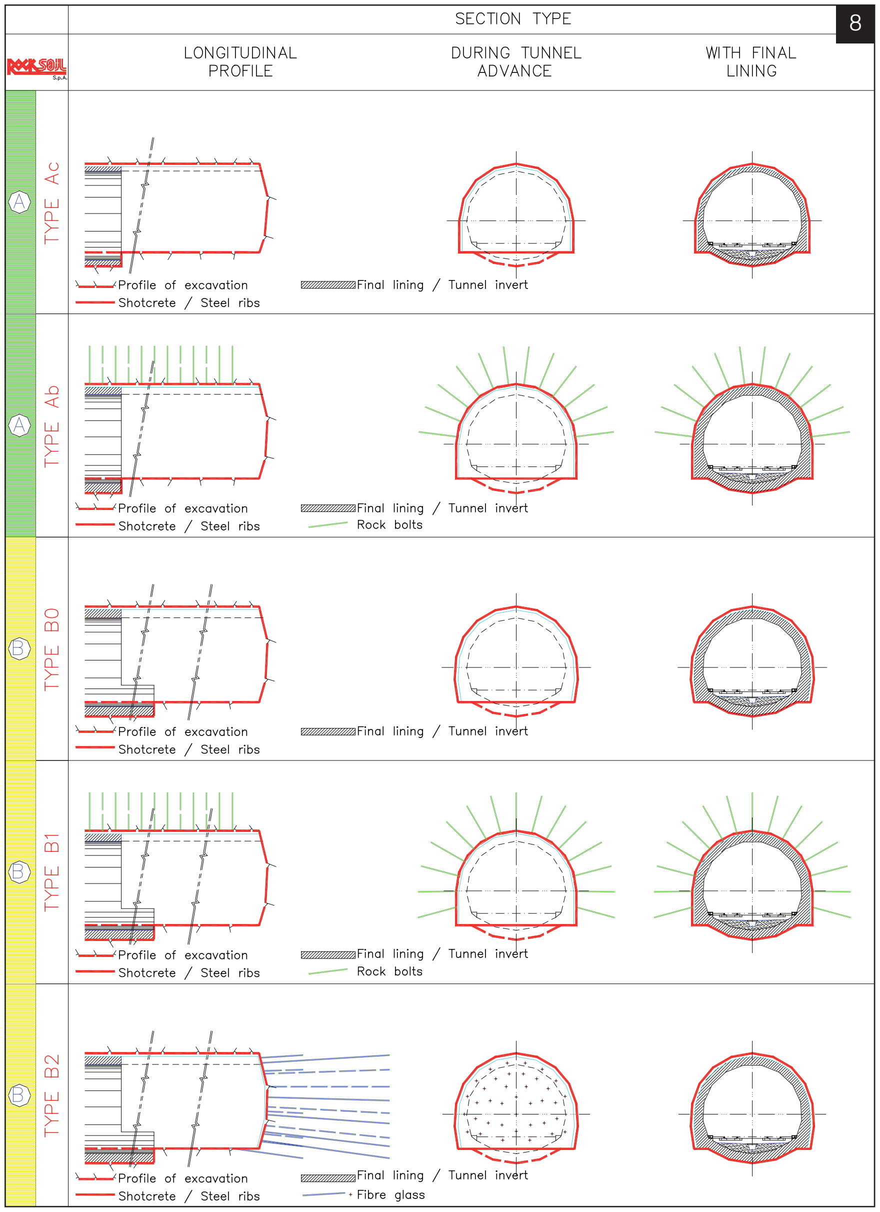
Similarly the behaviour of the ground, especially when the geomechanical conditions are complex and therefore, for example in, clays and also in soft rocks with high stress levels, and consequently in grounds subject to consolidation and creep, depends significantly on time. Also the ability to maintain good tunnel advance speeds has its effect on the control and development of stress states. A good construction method, which succeeds in guaranteeing rapid execution, is therefore an essential element for the success of excavation under these conditions.

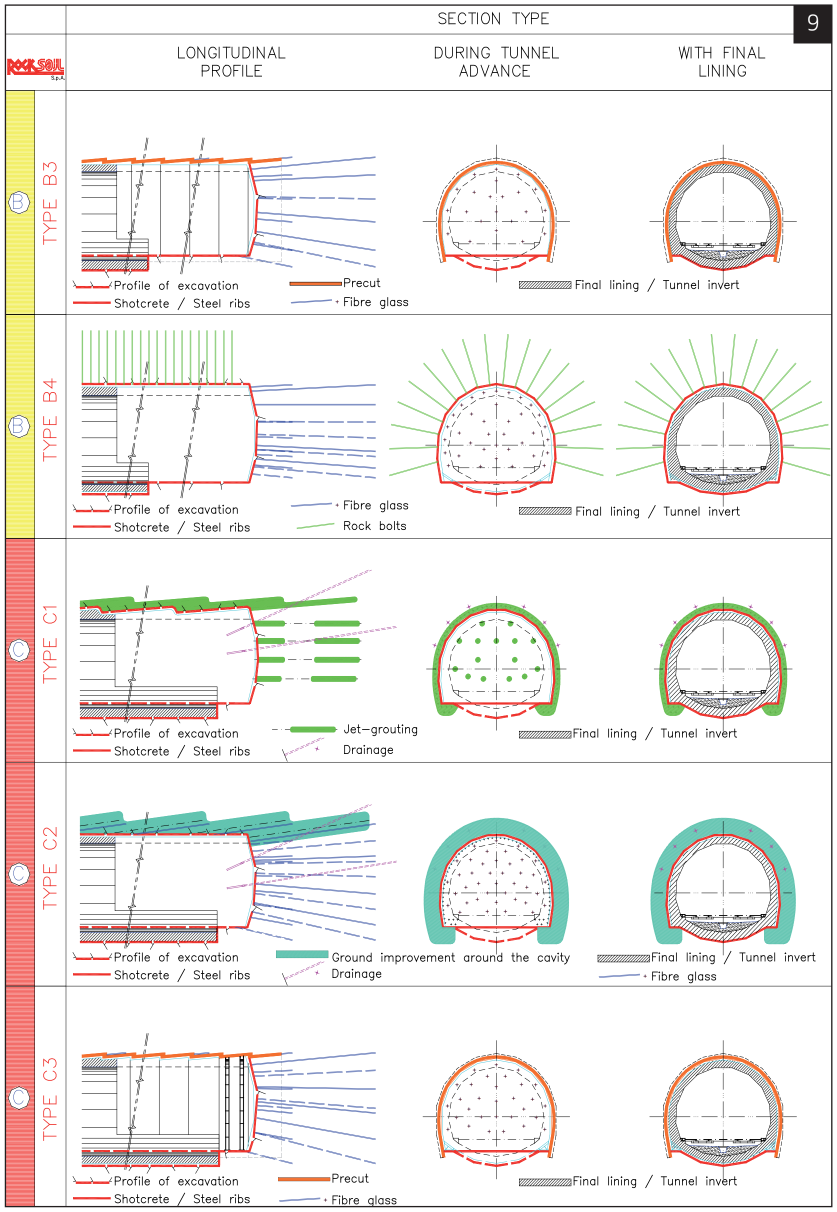
6. THE INDUSTRIALISATION OF TUNNELLING

7. THE EFFECT OF THE RIGIDITY OF THE LINING ON THE FINAL EQUILIBRIUM PRESSURE

The ADECO approach has introduced a whole series of advance section types with which tunnels can be tackled in any geomechanical context and under a huge variety of stress conditions, while maintaining constant advance speeds, by using a limited number of specially conceived and developed technologies. I think I can say that the "cleanness" and the "simplicity" of the design approach remain one of Prof. Lunardi's fundamental lessons. And I must say that having shared a large part of Rocksoil's history, I have a clear image in my head of tunnel construction sites in the early 1990s, confused and untidy, which gave the impression of advance a bit like groping in the dark, with all sorts of equipment crowded into narrow spaces, vaguely reminiscent of the underworld.







It was precisely in the middle of the 1990s that Rocksoil had the opportunity to apply the ADECO approach, which had by then been fully formulated, to the design and construction of tunnels on the Apennine section of the high speed rail line between Bologna and Florence. It involved more than 100 km of large diameter (excavation area of 140 sq. m.) tunnels in the most complex ground in Italy from a geomechanical viewpoint.
Over 30% of excavation was performed in the presence of consolidation for a total therefore of 30 km. of tunnel driven under conditions of stability in the short term or instability. For Rocksoil it was a chance to apply the method on a large scale in all geomechanical contexts and under stress conditions varying from low overburdens in loose soils or landslide deposits in portal zones to up to 500 metres of the Raticosa Tunnel in the scaly clays of the chaotic complex (Figures 8 and 9). Convergence and extrusion measurements were taken continuously in type C contexts.

__8, 9.__ SOME OF THE SECTION TYPES USED FOR THE TUNNELS ON THE NEW HIGH SPEED/CAPACITY RAILWAY LINE BETWEEN BOLOGNA AND FLORENCE

Type of monitoring station	Application range	Number of stations implemented
Geological-structural survey of the excavation face	Section types A, B, C	2978
Topographic monitoring around the cavity	Section types A, B, C	2460
Core-face extrusion	Section types B, C	460
Stress-strain state of the first stage lining	Section types A, B, C	342
Stress-strain state of the final lining	Section types A, B, C	156
Pore water pressure	Section types A, B, C in the presence of groundwater	61
Radial extensometric station	Section types B, C	21

Approximately 2,500 topographic measurement stations to monitor cavities and 460 extrusion metre stations, of the 30 metre type, were installed to give a total of approximately 30,000 convergence readings and 2,500 extrusion readings.

The numerous strain measurements taken in the final linings confirmed the low stress levels even under the very poorest of geomechanical conditions. Let us look here at the Pianoro Tunnel, excavated in the scaly clays of the chaotic complex under overburdens varying from between 30 m. and 100 m. The data collected is impressive. It demonstrated the validity of the principles of the method and constitutes an exceptional data bank for our firm. It gives concrete support to current designs and will also support future designs.

Also the tunnel advance rates reached in different geomechanical contexts and the constant advance rates give an idea of the result achieved in terms of the industrialisation of tunnel construction.

And now let us rapidly look at a few numbers which give a good idea of the work performed.

<table>
<tr><th>Tunnel</th><th>Tunnel length [m]</th><th>Overburden [m – min/max]</th><th colspan="2">Main geological formations [bored length in m]</th><th>Section types applied</th><th>Average advance rates [m/month]</th></tr>
<tr><td rowspan="13">PIANORO</td><td rowspan="13">10,710.00</td><td rowspan="13">5/167</td><td>EmS - SCHLIER MARLS
(Silty marls)</td><td>1,442.00</td><td>B0</td><td>130</td></tr>
<tr><td>ECO – COTIGNACO FORMATION
(Scaly argillites)</td><td>115.00</td><td>C4V</td><td>35</td></tr>
<tr><td>EmA (2) – ANTOGNOLA FORMATION
(Very dense fine sands and
low cemented fine sandstones)</td><td>45.00</td><td>C4V</td><td>40</td></tr>
<tr><td>EmA (3) – ANTOGNOLA FORMATION
(Clayey marls and marly argillites)</td><td>75.00</td><td>C4V</td><td>45</td></tr>
<tr><td rowspan="3">EmA (4) – ANTOGNOLA FORMATION
(Silty marls and marly siltites)</td><td rowspan="3">325.00</td><td>B2</td><td>65</td></tr>
<tr><td>B2V</td><td>40</td></tr>
<tr><td>C4V</td><td>55</td></tr>
<tr><td>EmA (5) – ANTOGNOLA FORMATION
(Argillites with scaly structure)</td><td>150.00</td><td>C4T
C4V</td><td>25</td></tr>
<tr><td rowspan="2">CHAOTIC COMPLEX
(Scaly argillites)</td><td rowspan="2">1,427.00</td><td>B2p</td><td>45</td></tr>
<tr><td>C4T
C4V</td><td>50</td></tr>
<tr><td>Epi – PLIOCENE INTRAPPENNINICO INFERIORE
(Marly siltites)</td><td>4,181.00</td><td>B0</td><td>85</td></tr>
<tr><td>Eps – PLIOCENE INTRAPPENNINICO SUPERIORE
(Silts and siltites)</td><td>2,950.00</td><td>B0
B0p</td><td>65</td></tr>
</table>

Tunnel	Tunnel length [m]	Overburden [m – min/max]	Main geological formations [bored length in m]		Section types applied	Average advance rates [m/month]
SADURANO	3,764.00	0/250	Eps – PLIOCENE INTRAPPENNINICO SUPERIORE (Silts and siltites)	3,587.00	Ac Ap Ap-b	80
					B0	60
					B2 with A.G.O.	30
			Epi – PLIOCENE INTRAPPENNINICO INFERIORE (Marly siltites)	61.00	AL-c	80
			EmB – BISMANTOVA FORMATION (Marly siltites)	116.00	B0-L	50
MONTE BIBELE	9,101.00	6/275	EmB – BISMANTOVA FORMATION (Marly siltites)	1,927.53	B0-L	50
					AL-c Ac Ab	130
			EaB – BISMANTOVA FORMATION (Arenaceous facies)	1,413.00	Ac Ab	145
					B0	90
			EaL – LOIANO FORMATION (Sands and low cemented sandstones)	824.00	B0	80
					B2	50
					B2V	25
			Lam - MONGHIDORO FORMATION (Alternation of marls, sandstones and argillites)	4,937.00	B2c B2b B2r	35
RATICOSA	10,370.00	3/525	dfr – LANDSLIDE DEBRIS	420.15	C1R C1R/bis C4R C4R/PU	55
			CHAOTIC COMPLEX (Scaly argillites)	3,920.00	C4R/PU C4R/P2 C4R/P300 C4R/p C6-1Rbis C6-Rbis	55
			Ol – OLISTOSTROME (Siltites)	1,949.00	Ab Ac	55
					B0	45
					B2	50
					B2V	20
					C4V-0	55
			RMA – MARLY-ARENACEOUS FORMATION (Alternation of marls and sandstones)	3,927.00	Ac Ab	95
					B0 B0Rc B0Rb	40 / 50
					B2V	30
			DEBRIS	153.58	C2P/1 C2P/1P C2P/2P C2P/2T C2P/2 C2P/2T C2P/1T	150

Tunnel	Tunnel length [m]	Overburden [m – min/max]	Main geological formations [bored length in m]		Section types applied	Average advance rates [m/month]
SCHEGGIANICO	3,530.60	0/301	RMA – MARLY-ARENACEOUS FORMATION (Alternation of marls and sandstones)	3,530.60	Ac Ab	150
					B0	55
FIRENZUOLA	15,210.83	0/582	RMA – MARLY-ARENACEOUS FORMATION (Alternation of marls and sandstones)	9,975.83	Ac Ab Ap Ap+b Ac-M2 Ac-M1	110
					B0 B0m-b B0m-c BOF B0-LF B0V	90
			CHAOTIC COMPLEX (Scaly argillites)	340.00	C6F	25
					B2F	60
					B0F	70
			TMG – CASTEL GUERINO FORMATION (Alternation of silty marls and sandstones)	1,008.00	Ap Ac	95
					B0 B0-LF	80
					B2	40
					B2V	30
			TMV – MOTTLED MARLS	351.00	B2V	30
					B2	60
			TM – MACIGNO FORMATION (Sandstones)	589.00	B0	80
					B2	60
					B2V	30
					C1F	25
			sBM – BACINO DEL MUGELLO FORMATION (Prevailing sands)	83.00	C1F	25
			aBM – BACINO DEL MUGELLO FORMATION (Prevailing clays)	2,864.00	B2 B2-pr B2M	30
					C4V C1M C1F C2M	30
MORTICINE	565.00	2/12	aBM – BACINO DEL MUGELLO FORMATION (Prevailing clays)	565.00	B2+p B2pr+p B2M+p	70
					C2br+P	25

Tunnel	Tunnel length [m]	Overburden [m – min/max]	Main geological formations [bored length in m]		Section types applied	Average advance rates [m/month]
BORGO RINZELLI	529.00	2/13	aBM – BACINO DEL MUGELLO FORMATION (Prevailing clays)	529.00	B2 B2pr B2pt	30
					C2br C2br+p	20
VAGLIA	16,755.17	0 - 32	aBM – BACINO DEL MUGELLO FORMATION (Prevailing clays)	393.00	B2	50
					C1	30
		12 - 123	TMM – MACIGNO DEL MUGELLO FORMATION	2,881.00	Avc	110
					B01 BOV	50
					B2 B2Pt	40
					B2V	40
					C2	45
		30 - 70	SSi – SILLANO FORMATION (Scaly argillites)	893.00	B01	60
					B2	40
		25 - 576	ScM – MONTE MORELLO FORMATION (Limestones and marls)	12,405.00	Avc	115
					B0 B01 B0LV B0VP	130
					B2	35
		15 - 30	sBF – BACINO DEL MUGELLO FORMATION (Sands)	183.00	B2 B2v B2v+p	25

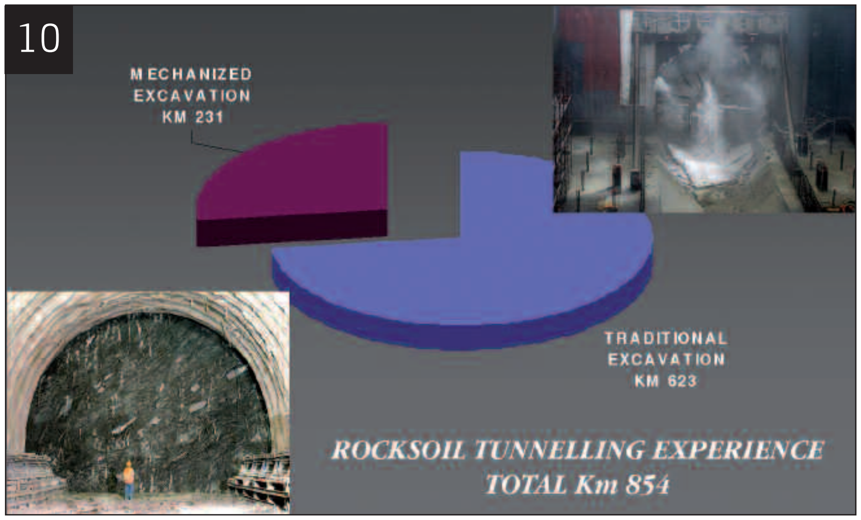
In the 30 years of its life Rocksoil has designed and supervised the construction of approximately 800 km of tunnels, of which 625 km. were driven using conventional means and 136 km using TBMs. Over 2,000 tunnel faces have been supervised.

I wish to make one point on the question of TBM excavation. Often Rocksoil is cited as a specialist in conventional excavation and I would certainly not wish to detract from this description which is high consideration.

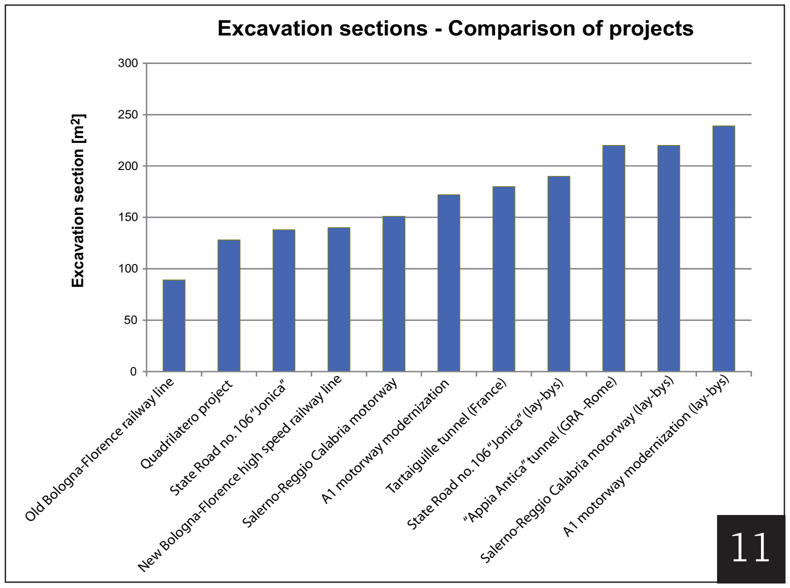
However, I would like to point out that 136 km of TBM tunnel design is really quite a lot and it has given us the opportunity to learn considerably in this field. Amongst other things, Rocksoil consulted on the first two EPB TBMs to work in Italy for the Genoa metro and for the Milan Urban Railway Link Line.

Today we are currently consulting on approximately 40 km of TBM driven tunnels with the same number under design.

10. ROCKSOIL HAS DESIGNED AND SUPERVISED OVER 800 KM OF TUNNELS SINCE IT WAS FOUNDED 30 YEARS AGO



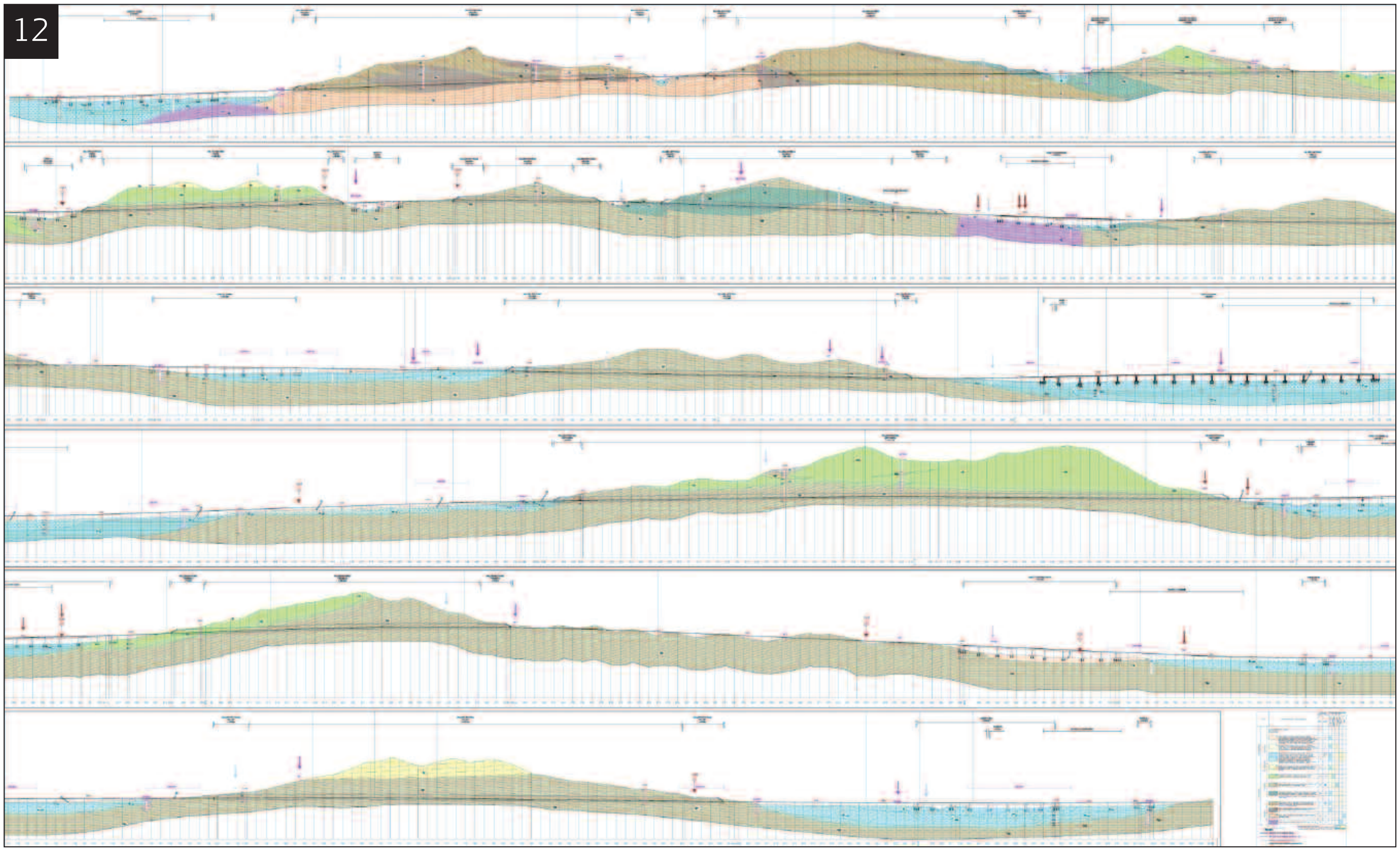
Tunnels	Excavation faces
Jonica	22
A1 Mororway modernization	20
Salerno-Reggio Calabria motorway	18
"Quadrilatero" project	10
Fortorina	2
Darfo Edolo	2
Brenner Base pilot tunnel	2
Pontremolese railway	1
Pieve di Teco	1



11. *EXCAVATION SECTIONS*

12. *JONICA STATE ROAD – TOTAL LENGTH OF UNDERGROUND TUNNEL: 13.265 KM*

THE MOST COMMON LITHOTYPE: SILTY PLIOCENE CLAYS ("AGS"). OVERBURDENS: RECENT HOLOCENIC



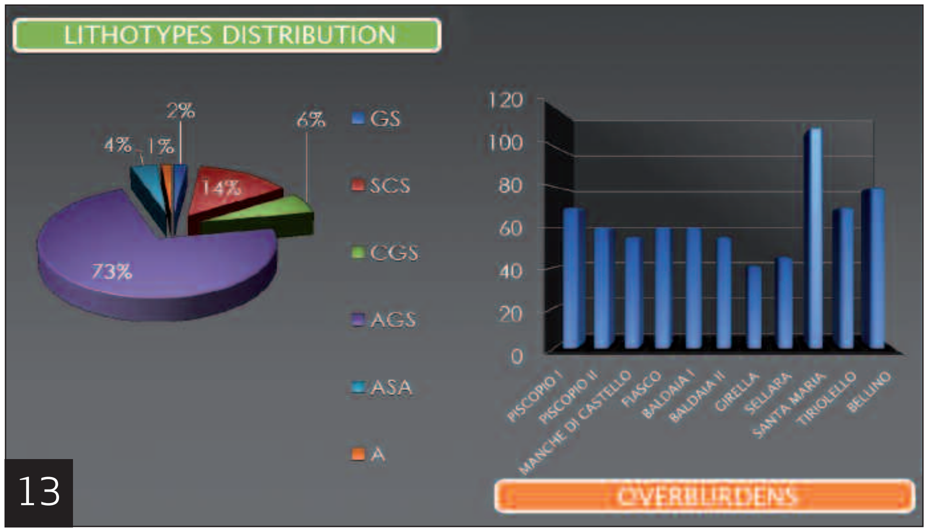
Finally with regard to conventional excavation, we are consulting on over 70 km of tunnels for a total of almost 80 tunnel faces.

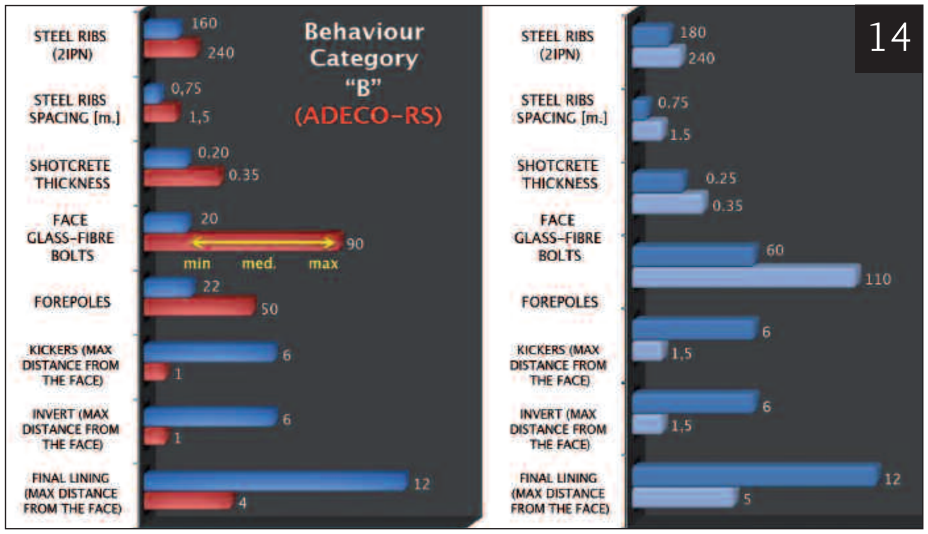
It is an impressive scenario in which to continue our research. It is evolving together with the technologies and the equipment available for tunnelling. With excavation cross sections becoming increasingly larger (Figure 11), this makes the challenge of constructing underground works ever more interesting and exciting.

I will give just a brief report on the underground tunnels for which we are consulting in Calabria for the SS 106 Jonica state road (Figure 12).

These are road tunnels with a cross section similar to those for the Bologna-Florence motorway, with 80% driven through sandy silts that vary greatly from a geomechanical viewpoint, with overburdens of between 50 m. and 100 m. The tunnel requires continuous calibration of tunnel advance types to match the conditions actually encountered at the face (Figure 13).

The problem was solved by defining the variability of the different types of preconfinement and confinement at the design stage and by adapting the tunnel section types seeking to maintain deformation levels within a given range, which would ensure tunnel advance under stable conditions with good production levels (Figure 14).





13. LITHOTYPES DISTRIBUTION

14. EXAMPLE FOR THE SECTIONS IN AGS: VARIABILITY OF THE SECTION TYPES AND DEFINITION OF THE BEST MIX

I will end with a phrase that a colleague found on the internet and which we found perfect to describe our working philosophy.

"...Little observation and much reasoning lead to error.
Much observation and little reasoning lead to truth..."
(Alexis Carrel , Nobel prize for medicine 1912)

... But our work and research continues.

Thank you everyone for your attention.

Hiromichi Shiroma

THE EVOLUTION OF SPRAYED CONCRETE METHODS AND AUXILIARY METHODS IN JAPAN

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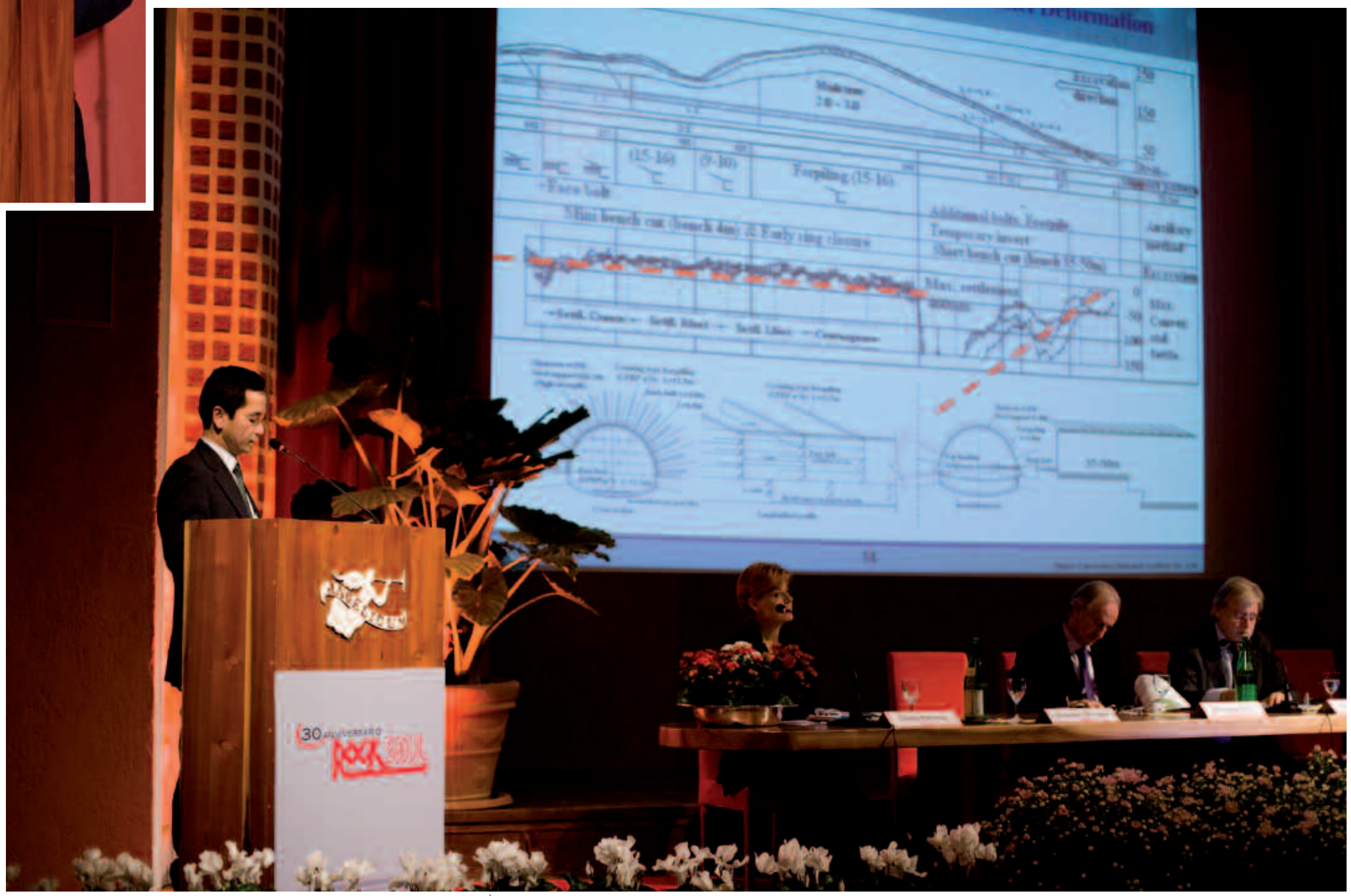
Around 1970, sprayed concrete method (including concepts of NATM) was introduced into Japan. It has developed to an original tunneling method that is able to suit the Japanese geological conditions, with the help of development of new auxiliary methods and excavation method. Recently, there are cases that also the tunnel excavation to consider three-dimensionally as well as ADECO-RS is done in squeezing ground.

1.INTRODUCTION

Japan is an archipelago located off the eastern coast of Eurasia. The Japanese geology is composed of mountainous areas that have developed on the accretionary wedge as the substratum mainly of sediments, under various phases of active diastrophism due to forces of subduction



by the Pacific Plate and Philippines Plate. So the archipelago presents very complex features of topography and geology, which has a great impact on the performance of tunneling projects.
Construction of tunnels for roads and railways was modernized in Japan around 1870. At first, in order to introduce tunneling technologies from overseas, foreign engineers were invited from abroad.
The prevailing practice of tunneling at that time was to support the tunnel with timbers, and the excavated wall was lined with bricks and masonry. Since then, innovative materials, including steel supports, shotcrete/rock bolts have been introduced, and depending upon the newly introduced concepts of NATM (New Austrian Tunneling Method) and many other improvements in construction like many kinds of auxiliary methods, the tunneling technology in Japan has progressed tremendously, overcoming the complex and varying requirements of the geology, and has reached an established level as the mainstream technique. Here, the NATM is considered as one aspect of the sprayed concrete method.
This paper is intended to discuss, using examples of actual tunneling projects, the various aspects of tunneling technologies in Japan in chronological sequence, starting from introduction of sprayed concrete method up to the newest technologies.

2. INTRODUCTION OF THE SPRAYED CONCRETE METHOD IN JAPAN

2.1 Before the introduction of the sprayed concrete method

In Japan, until the introduction of a sprayed concrete method, the technique of rib and lagging method (conventionally used) had been widely used, which consists of laggings and steel spiles to prevent small collapses during excavation, with wooden supports and steel arch supports to bear the load of the ground, and ultimately, the lining concrete takes over their role.
Around 1960, the sprayed concrete method was initially used as a countermeasure for ground prone to swelling, to prevent the ground from loosening and deteriorating. The application of sprayed concrete was made immediately after excavation. At that time, the supports were designed mainly depending on their rigidity to bear ground pressure. Even though there were cases of measurements of earth behavior, their results were used only to confirm the stability of supports.

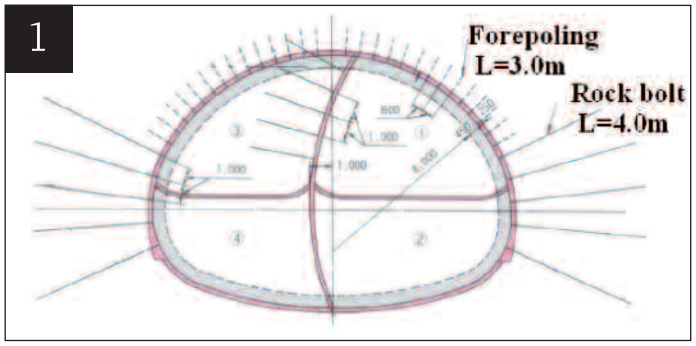
2.2 Full introduction of the sprayed concrete method

In 1970s the demand for the construction of tunnels intensified along with development of expressway and bullet train networks. In the midst of this rapid growth era, Japan introduced a sprayed concrete method (including the concepts) which consists in using shotcrete and rock bolts as the main supports.
This practice was already widely used in Europe. This method and the concepts revolutionized the concepts and construction techniques of tunnels to that point. This sprayed concrete method for tunneling no longer depends upon using the supports and lining to bear the load resulting from ground loosening, rather, upon effectively using shotcrete and rock bolts as support. By controlling ground deformation, this technique exhibits the capability to follow ground deformation, and supplement the inherent ground support.

According to reports, at a tunneling site for which the rib and lagging method was used, but was suffering significant deformations, the deformation was successfully controlled by resorting to the sprayed concrete method.
The sprayed concrete method in Japan, through development of new auxiliary methods and improvements in excavation, has evolved to a level that is able to accommodate the conditions of the geology in our country.
The sprayed concrete method, consisting of support members including shotcrete, rock bolts and steel supports, was given status as a standard method in 1986 in the "Standard Specifications for Tunneling" by the Japan Society of Civil Engineering Association of Japan.

3. DEVELOPMENT OF THE SPRAYED CONCRETE METHOD AND AUXILIARY METHODS

3.1 Difficulties overcome by the sprayed concrete method



1. EXCAVATION BY DIVIDING SMALL SECTION

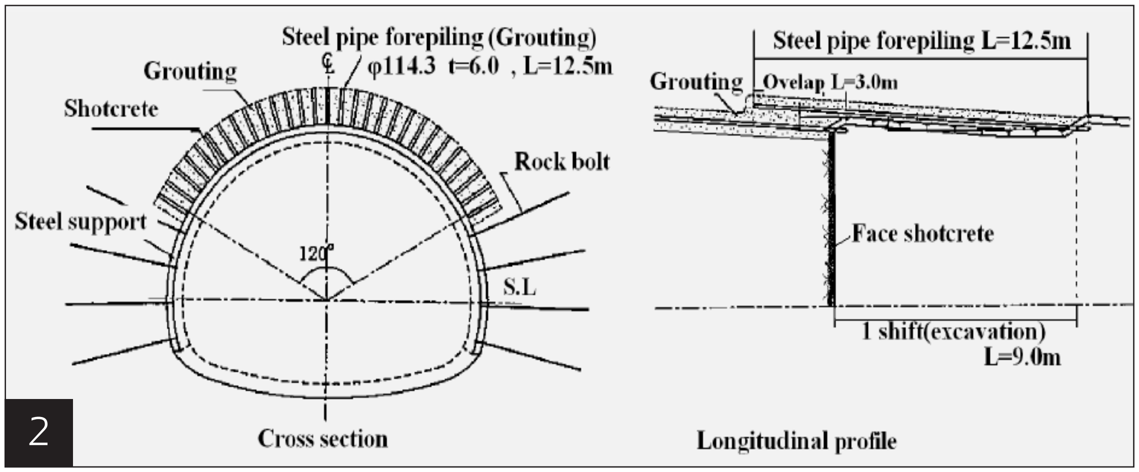
The sprayed concrete method is basically a method applicable to the ground that is able to form a ground arch, and able to keep the face stable. However, the tunnel is a longitudinal structure along which a variety of geological conditions are found. In the unconsolidated ground (tunnel portal) with shallow overburden where the formation of a ground arch is difficult, loosened loads and ground surface settlement may cause significant deformation.
In the event the face is unstable and not able to support itself, the structure of a tunnel will not be stabilized, and accidents occur due to face collapses and loosened loads. In such cases, prior to introduction of auxiliary methods, the head was divided into small sections (excavation section) that in the process of excavation were closed one by one to stabilize the tunnel structure and the ground (Figure 1). With this solution however, it was impossible to introduce a large tunneling machine and the efficiency in excavation remained restricted. In addition, the stress is redistributed each time the width is enlarged, disturbing the ground, and in some cases, leading to a large deformation.

3.2 Development of auxiliary methods

It is known that the sprayed concrete method is economical and applicable effectively as far as the ground is stable, however in cases having no stable face, the excavation becomes difficult, and significant deformations need to be controlled. To widen the applicability of the sprayed concrete method to cope with a broader range of geological conditions, auxiliary methods were developed to stabilize the crown and face, and to control deformation.
Currently auxiliary methods are used actively in tunneling projects. In many cases, the head is kept as a large section, so as to introduce a large tunneling machine for larger excavation. Table 1 shows the classification of auxiliary methods and Figure 2 typical examples of auxiliary methods.

TABLE 1. *EXAMPLE OF AUXILIARY METHODS*

Example of auxiliary methods						
Method		Construction safety			Environmental preservation	
		Face stabilization				
		Crown stabilization	Face stabilization	Footing stabilization	Ground surface settlement control	Neighboring structure protection
Presupport	Forepoling	○				
	Steel pipe forepiling	○			○	○
	Pipe roof	○			○	○
	Horizontal jet grouting	○	○	○	○	○
	Slite concrete method	○			○	○
Face, footing reinforcement	Face shotcrete		○			
	Face bolt (Long)		○			
	Temporary invert			○		
	Footing reinforcement bolt/pipe		○			
○ : relatively common method						



2. *EXAMPLE OF AUXILIARY METHOD (STEEL PIPE FOREPILING)*

4. REINFORCEMENT IN THE AREA OF THE FACE AND EARLY RING CLOSURE

4.1 Reinforcement in the area ahead of the face

4.1.1 Steel pipe forepiling (presupport)

The technique of forepiling is aimed at supporting the crown of the area ahead of the face by forming an arch structure of a comparatively high rigidity to control in advance displacement ahead of the face in the event of a surface settlement at the site of a small overburden. In Italy, this technology is called the "Umbrella Method".
There are two types of presupport, steel pipe driving and jet grouting. In Japan, the ground conditions vary significantly, and in order to cope with this problem, a newly developed universal presupport system consisting in driving long steel pipes into the ground by means of a drill jambo, called AGF (All Ground Fastening), is widely used.

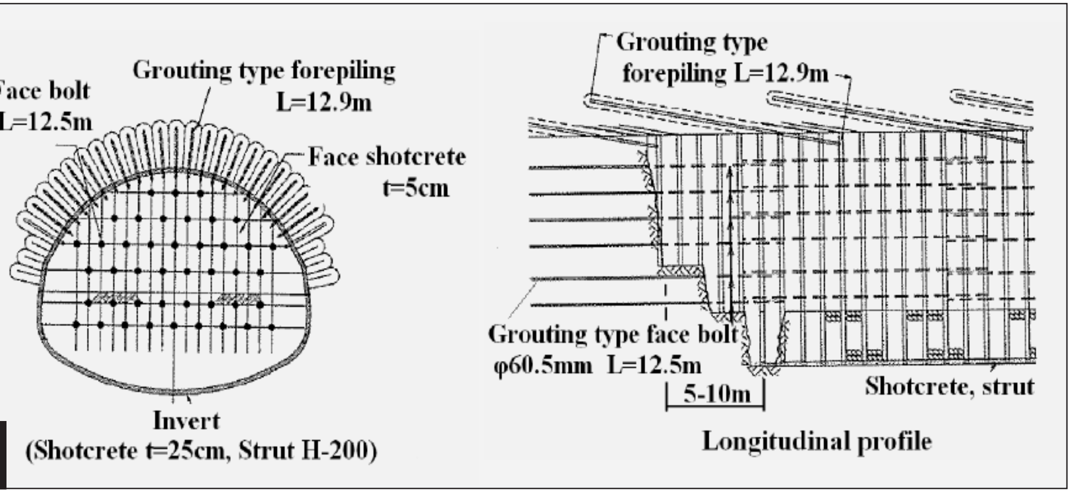
The jet grouting system consists of a machine dedicated to applying grout horizontally to form an arch in the ground in the area ahead of the face, and exhibits an excellent performance in controlling advance displacements. This method finds many uses in urbanized areas where forcibly controlling deformations in the unconsolidated ground is needed.
At a tunneling site applying presupport to the ground, the foot of the upper half in some cases needs reinforcement because it is prone to settlement by concentration of loads after the passage of the face.

4.1.2 Face bolts (face reinforcement)

There are two types of face bolts, use of short bolts up to a length of about 12 meters and use of long bolts longer than 12 meters.
The former is aimed at preventing small face collapses and falling of rock masses resulting from ground loosening and fissuring.
The latter, in addition to these, is intended to control displacement ahead of the face.
Although in Japan, the effectiveness of face bolting is understood through various study cases, there was no systematic approach to reinforce the area ahead of the face.
Meanwhile, Professor Lunardi, in May 2000, released in Tunnels and Tunnelling International a dissertation titled "The design and construction of tunnels using the approach based on analysis of controlled deformation in rocks and soils".
This release had a sensational impact on Japanese tunneling engineers. They were greatly impressed by this innovative understanding, paying attention to the construction of the core section.
This idea was integrated into a design for the tunneling system. Since then, the Japanese industry has attached importance to controlling of advance displacements by means of long face bolts, especially for squeezing ground susceptible to deformation.
Of course, it goes without saying that the 3D analysis technology and development of face bolts materials, including GFRP (Grass Fiber Reinforced Pipe), also have made a great contribution.

4.2 Early ring closure

It is widely known that a tunnel structure may develop stability most effectively when being closed in the form of a circular ring. In squeezing ground, early ring closure after passing of the face is a key approach.
In years past, when no large auxiliary method had been introduced, the head was divided into small sections as mentioned above, and their respective ring sections were closed to control deformation. However, in recent years, a large scale auxiliary method was employed at the site, making it possible to provide large sections of the head for excavation; so, the recommended approach at present is to close, the tunnel section as soon as possible. KONDA et al., using field measurement data and analysis, demonstrated that closing the full face in an early stage is effective in controlling deformation, and also that a spherical face when tunneling presents better stability for full face excavation.



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3. *EARLY RING CLOSURE*
BY STRUT AND SHOTCRETE INVERT

At the author's research institute, we are engaged in research and development to verify the effectiveness of the approach of early ring closure, especially at sites with squeezing ground significantly susceptible to deformation.

The main purpose of early ring closure is to control deformation. As an auxiliary method used in conjunction with the early ring closure, long face bolts and long steel pipe forepiling are frequently resorted to. Rings are closed at a distance of 4 to 10 meters from the face.

In Japan, the use of a mini bench with 3 meters long is usually seen as a step for excavation. Figure 3 shows a typical example of early ring closure.

The following are observations from engineers who have experience in early ring closure at the field.

1. Early ring closure is effective in controlling deformation.
2. With an early ring closure, the stress of supports increases, so it is necessary to increase support rigidity.
3. The early ring closure takes time in implementation of auxiliary methods and excavation (it needs a longer cycle time).

As shown above, in Japan there are an increasing number of cases using auxiliary methods to stabilize the area ahead of the face and crown in squeezing ground, and stabilizing a tunnel by the early ring closure.

In other words, the trend to understand the behavior of ground is shifting from two-dimensional to three-dimensional. This way of thinking, as seen in ADECO-RS, consists in controlling advance displacement of the area located ahead of the face (advance core), and in using the early ring closure to stabilize the whole of a tunnel and its surroundings.

As a reference, Table 2 shows the ground classification of road tunnels in Japan and corresponding ADECO categories. The grounds for which we recommend early ring closure for tunneling are class DII or lower classes, and correspond to B and C in ADECO-RS category.

TABLE 2. COMPARISON BETWEEN NEXCO GROUND CLASSIFICATIONS AND ADECO-RS BEHAVIOUR CATEGORIES

Comparison between NEXCO Ground Classifications and Categories ADECO-RS							
NEXCO Ground Classification	A	B	C		D		E
			C I	C II	D I	D II	
Competence factor	-	-	4 or more		4 - 2	2-1	1 or less
Amount of convergence (mm)	Minute	15 or less	20 or less	30 or less	60 or less	200 or less	200 over
Elastic coefficient (Mpa)		5,000	2,000	1,000	500	150	
Categories of ADECO-RS	A					B	C

4. *PRE-REINFORCEMENT BY JET GROUTING AND FACE BOLT*

5. *EFFECT OF THE FOOTING REINFORCEMENT BY JET GROUTING*

5. EXAMPLES OF TUNNELING BY THE REINFORCEMENT OF AREA OF THE FACE

5.1 The case of reinforcement of the area ahead of the face with jet-grouting (1997)

The tunnel was a two-lane road tunnel (12 meter wide) that was located 20 meters from another tunnel in service.

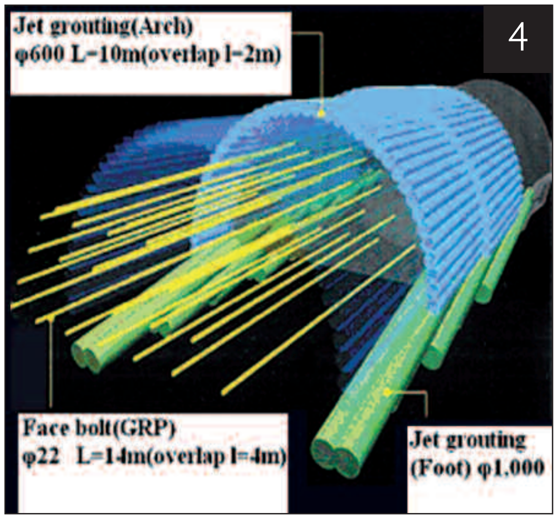
The site is located in geology composed of an alternation of strongly weathered, fractured sandstone and mudstone that is in a landsliding zone with a 0.5 competence factor.

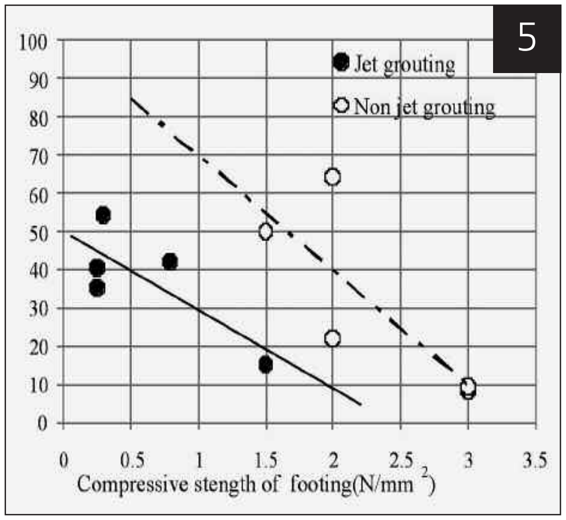
This site needed to be given a capability not only of controlling tunnel deformation, but also of preventing a land slide that may result from tunnel excavation, as well as avoiding adverse impact on the adjacent tunnel in service.

As a reinforcement measure, the project used a rodin jet forepiling method (RJFP) able to form a highly rigid advance arch (Figure 4).

In addition, footing and face reinforcement bolts were implemented during tunnel excavation, since there was fear that the solution of supporting the load ahead of the face by forming an arch might produce loads which act on the feet and face of the tunnel.

Figure 5 shows unconfined compressive strength of the footing ground as well as the settlement amount at the time of 0.5 D in face advance.





From this figure, we learn that the settlement is in controlled state even though the ground strength decreases due to previous reinforcement of tunnel footings. However, the project was continued using the top heading method, and it took a long time to completely close the ring of the whole section, so the settlement was larger than expected.

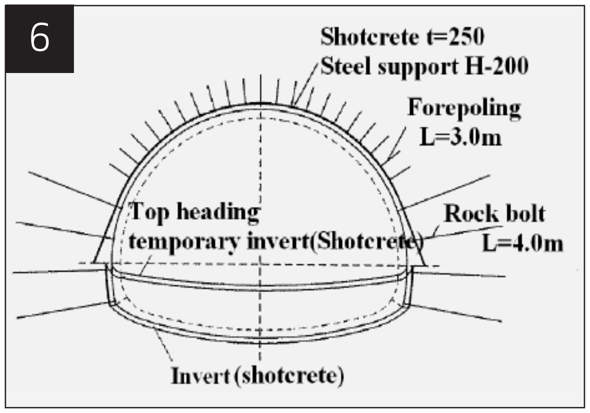
5.2 Example 1 for the case of reinforcement of the area ahead of the face and early closure (2006)

This tunnel was constructed in geology of weak mudstone subject to fracture and alteration, and the unconfined compressive strength varied greatly in a range of 0.1 to 7 N/mm^2.

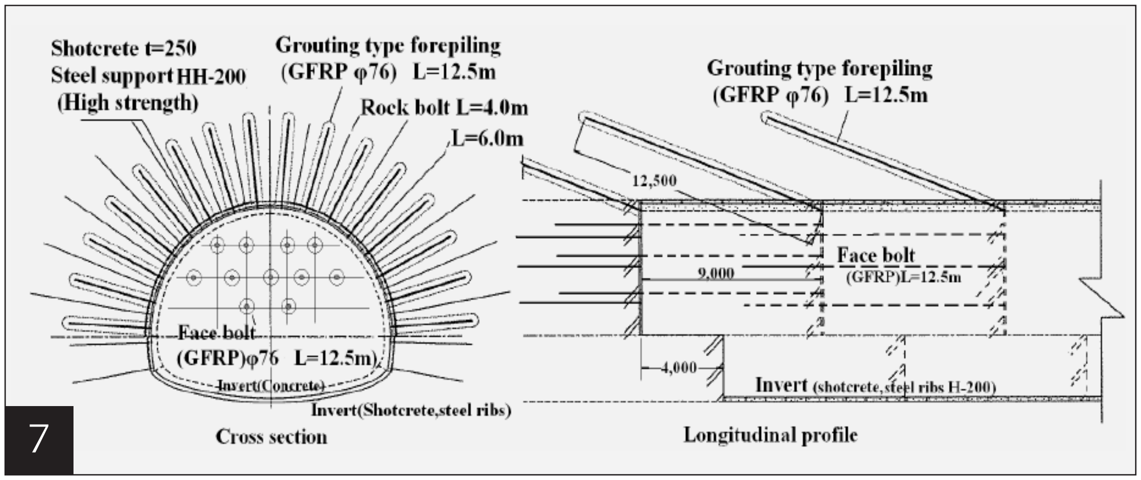
In the earlier stage of the tunneling process, short bench cutting method was used, but because of an increase in the settlement, was shifted to the temporary top inverting method to control deformation (Figure 6).

However, since as the overburden increased in depth, the top invert method could not sufficiently control deformation, we adopted an early closure of the ring by a mini-bench cut method shown in Figure 7.

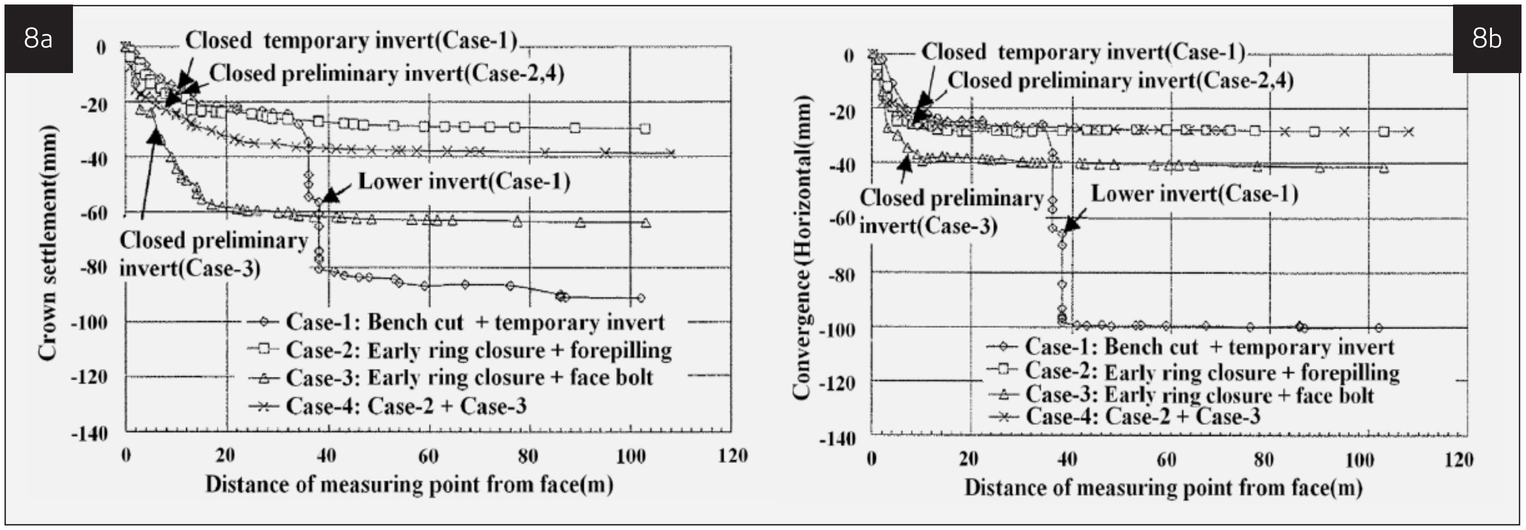
When closing the ring early, it was thought that shortening the benches would lead to instability of the face. To cope with this problem, the means used were early reinforcement of the outer circumference and reinforcement of the face by driving face bolts in the direction of advance of the tunnel. Figure 8 shows the amount of convergence at the ultimate stage and the behavior of deformation per excavation method and auxiliary method. We can see clearly that the early ring closure contributes to controlling of deformation.



6. CROSS SECTION OF THE SHORT BENCH METHOD WITH TEMPORARY INVERT



7. EARLY RING CLOSURE BY STRUT AND SHOTCRETE USING AUXILIARY METHOD



8. COMPARISON OF THE DEFORMATION BETWEEN BENCH CUT AND EARLY RING CLOSURE

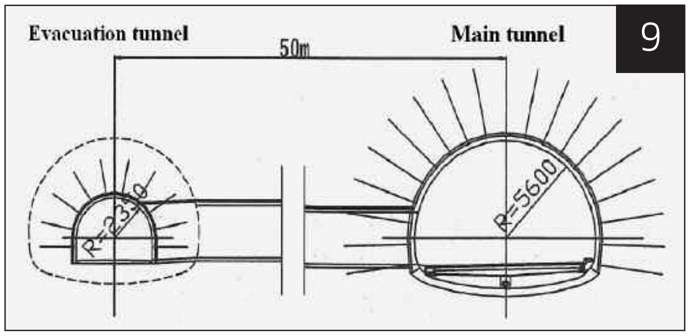
A) CROWN SETTLEMENT
B) CONVERGENCE (HORIZONTAL)

9. CROSS SECTION

Furthermore, the reinforcement of the area ahead of the face can be achieved, not solely with face bolt driving, but more effectively by using at the same time the previous reinforcement of the circumference that consists of pre-reinforcement at the crown.

5.3 Example 2 for the case of reinforcement of the area ahead of the face and early closure (a project under construction)

5.3.1 Geological conditions



The tunnel has a total length of 4,323m with an evacuation tunnel (Figure 9).
The geology of the site is a structure of accretionary wedge that is composed of various rock bodies (mudstone, green rock and serpentinite); in particular in the serpentinite section, there was significant deformation in the excavation of the evacuation tunnel. In addition, the serpentinite at this site was anticipated to squeeze, from the past data on construction of road and railway tunnels in the surroundings.

5.3.2 Construction of an evacuation tunnel

The evacuation tunnel, assuming the role of an investigation drift, was constructed prior to driving the main tunnel. As anticipated, the serpentinite section demonstrated significant deformation, and parts of the section suffered from heaving, leading to breakage of the invert. Measurement data on the serpentinite region are shown in Figure 10.

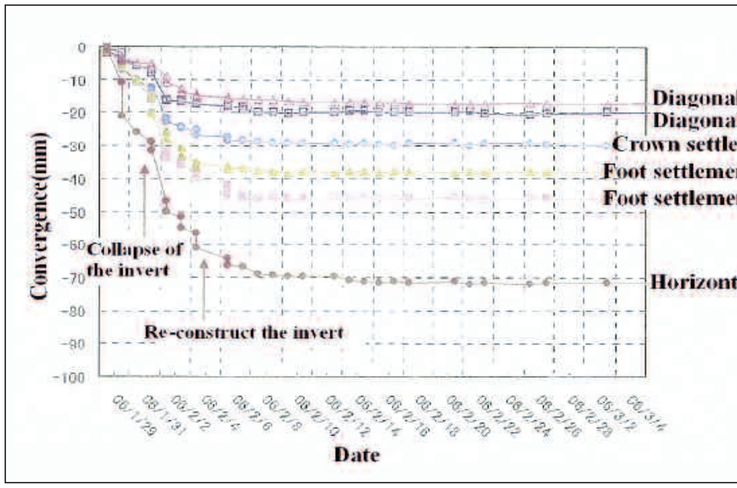
We identified the geomechanical parameters of the serpentinite section with the back analysis by the use of measurement data. The identified parameters are equivalent to ground class DII or E by NEXCO Standard.

5.3.3 Designing of the main tunnel

The support system of the main tunnel was designed using an analytical method based on the physical values that were identified at the evacuation tunnel.

Having identified a large ground pressure working at the site, the original design of the main tunnel was judged inadequate as support. The policy for design and construction was modified as follows:

- The geometrical profile of the section is designed as close to a ring as possible.
- Use the auxiliary method designed to control advance displacements and to stabilize the face.



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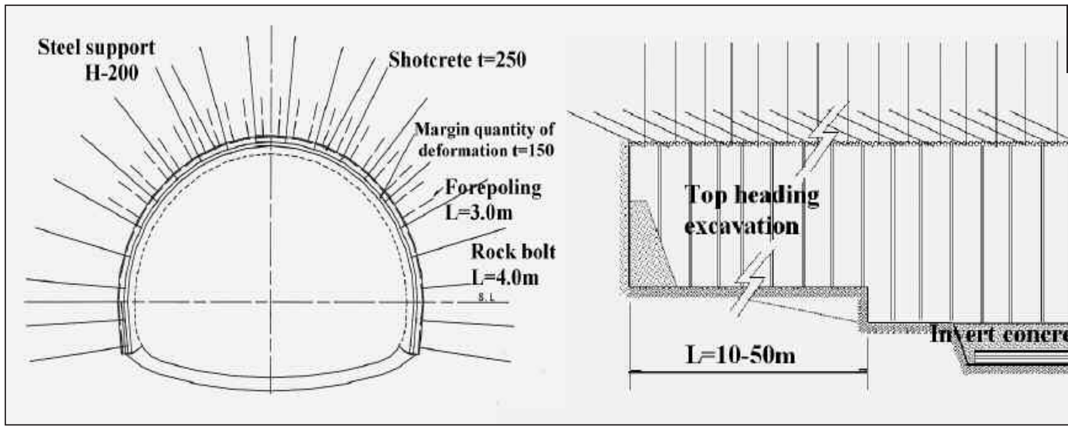
10. CONVERGENCE OF EVACUATION TUNNEL FOR THE SERPENTINITE AREA



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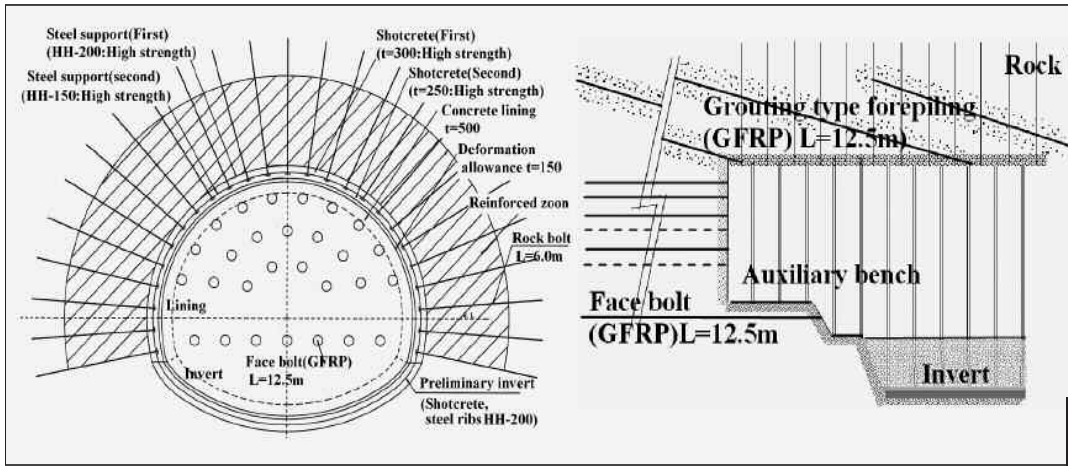


11b

11. MODIFICATION FROM ORIGINAL DESIGN

A) ORIGINAL DESIGN (SHORT BENCH CUT METHOD)
B) MODIFIED DESIGN (EARLY RING CLOSURE)

- Instead of the top heading method, the full face tunneling is used with auxiliary benches to close the ring early.
- The strength of the support is increased, and designed as a double support structure.
- The secondary support is constructed with a cavity strain of 0.5 % or less, in order not to increase the plastic band of behavior of the ground.

Figure 11 shows the section of a tunnel in the original design and after modification.

5.3.4 Construction

As a first step, crown pre-reinforcement and long face bolts were placed to control advance displacements that might occur ahead of the face. Full face excavation method was used with auxiliary bench cut. By constructing a 3-5m long mini-bench, the upper half and lower half were excavated at the same time or alternately, and then, the primary supports were placed.

The secondary supports were placed carefully, referring to measurements and watching that the inner cavity of the primary support did not induce significant strains leading to ground plasticization.

The reason why such a solution was used is that the ground, while integrated (in other words, not deteriorated), may bear a certain degree of load. A 0.5 % limit of convergence was established based upon past experience.

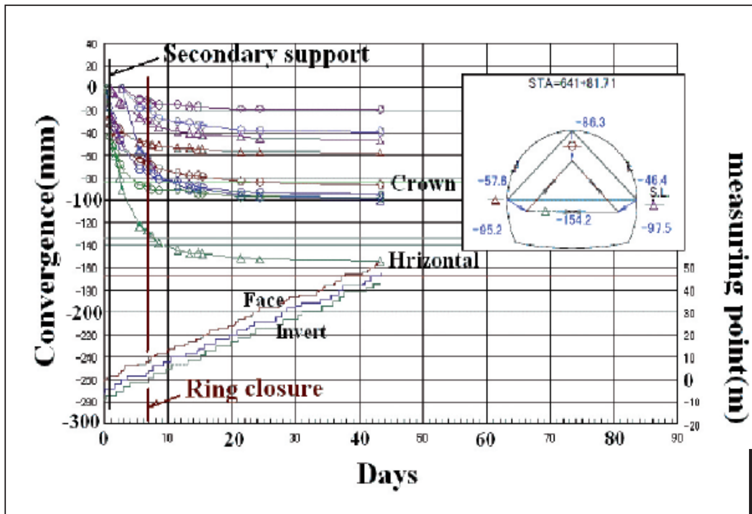
The next step was a process of closing the invert with struts and sprayed concrete. The ring should be closed, in principle, with 10 meters in the longest case and 5 meters or less in the shortest case.

Lining concrete was placed after confirming by measurement that the convergence reached the ultimate stage.

5.3.5 Measurement results and effects

Figure 12 shows convergence measurements for the serpentinite section. Radial displacement doesn't converge even after the placement of double support, but following the ring closure by invert, reaches the ultimate stage.

The ring closure by invert increased the stress of the support, and some parts of the steel support recorded the yielding stress (440 N/mm2). However, the support itself was in a compressive state as a whole, and the stress of the shotcrete was within an allowable range. Considering the displacement had already converged, the tunnel was deemed to be structurally stabilized.



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12. CONVERGENCE OF MAIN TUNNEL FOR THE SERPENTINITE AREA

5.3.6 Challenges in reinforcement of the area ahead of the face

This tunneling project is currently under construction, and the following are problems to be solved for the reinforcement of the area ahead of the face and early ring closure:

- Establishing reinforcement of the area ahead of the face and supports took a long time, the advance of month recording 35 m to 40 m/month.
- The large load is acting on the support in the current 250 m overburden. The support design will be problematical; the overburden will increase to 350m in future.
- The effect the placement of rock bolts may demonstrate should be studied for the case of an early ring closure and of increase in support rigidity.

6. CONCLUSIONS

It has been thought that the sprayed concrete method (NATM) is an approach to optimize the support taking advantage of the supporting capability of the ground. However, there have been cases where a squeezing ground allows significant deformation, inducing huge deformation. Considering such cases, the yielding supports, which were used initially in Japan, currently were excluded from the recommendation. It is thought that it is better for deformation of a tunnel to be confined within the deformation capacity of the ground. If the ground of squeezing or weak ground has no deformation capacity, it is necessary to use pre–reinforcement (exp. long forepiling and long face bolts) to control the deformation at the area ahead of the face, and to close the ring early.

Recently, the sprayed concrete method has been used for projects in urban areas. Reinforcement of the area ahead of the face and early ring closure are adopted in order to control deformation of the tunnel, considering influence for increase ground settlement and acting on structures in the proximity.

As discussed above, in Japan, the excavation method depending upon the reinforcement of the area ahead of the face, even though not employed systematically as an established tunneling method as seen in the case of ADECO-RS, is applied as an auxiliary method for sprayed concrete method (NATM).

When the ground is squeezing or weak, the early ring closure is used in conjunction with reinforcement of the area ahead of the face.

These two approaches are expected to become key solutions to control deformation of tunnels in the future. In other words, the ground behavior by tunneling should be understood three dimensionally, not from two dimensional approaches which prevail so far. We think the ADECO-RS that was discussed in TTI has largely contributed to the development of tunneling technologies. We would like to express our thanks to Professor P. Lunardi and ROCKSOIL Company for their contribution to the tunneling technologies, and at the same time, we are hopeful that field measurement and the engineering of rock/soil mechanics will develop further.

BIBLIOGRAPHY

LUNARDI P., 2000, Design & Constructing Tunnels- ADECO-RS approach, Tunnels and Tunnelling International, May 2000 (supplement)

STANDARD SPECIFIC FOR TUNNELING, 2006, Mountain tunnels, Society of Civil Engineers, August 2007, Japan

MITARASHI Y., MORISAKI Y, KONDA T., 2007, Method of Early Closure of All Tunnel Cross-sections and its Case Studies, Tunnels and Underground (Japanese), Vol. 40, No. 7, September 2007,

TERAYAMA T., TUNO K., UTUMI T., HIRUKAWA A., 2008, Excavate earth tunnel using full face cutting with short bench method and early invert closure – Kishiya-Namamugi Line tunnel, Tunnels and Underground (Japanese), Vol.39, No.5, May 2008

TOKUDOME O., OOTSU T., HIROSE M., SAWADA K., 2007, Countermeasure to prevent Displacement in Weak Mudstone Ground influenced by Faults and Folds, Proceedings of Tunnel Engineering, JSCE(Japanese), Vol.17, November 2007

Bruno Mattle

TUNNEL DESIGN FOLLOWING THE AUSTRIAN GUIDELINE FOR THE GEOMECHANICAL DESIGN OF UNDERGROUND STRUCTURES

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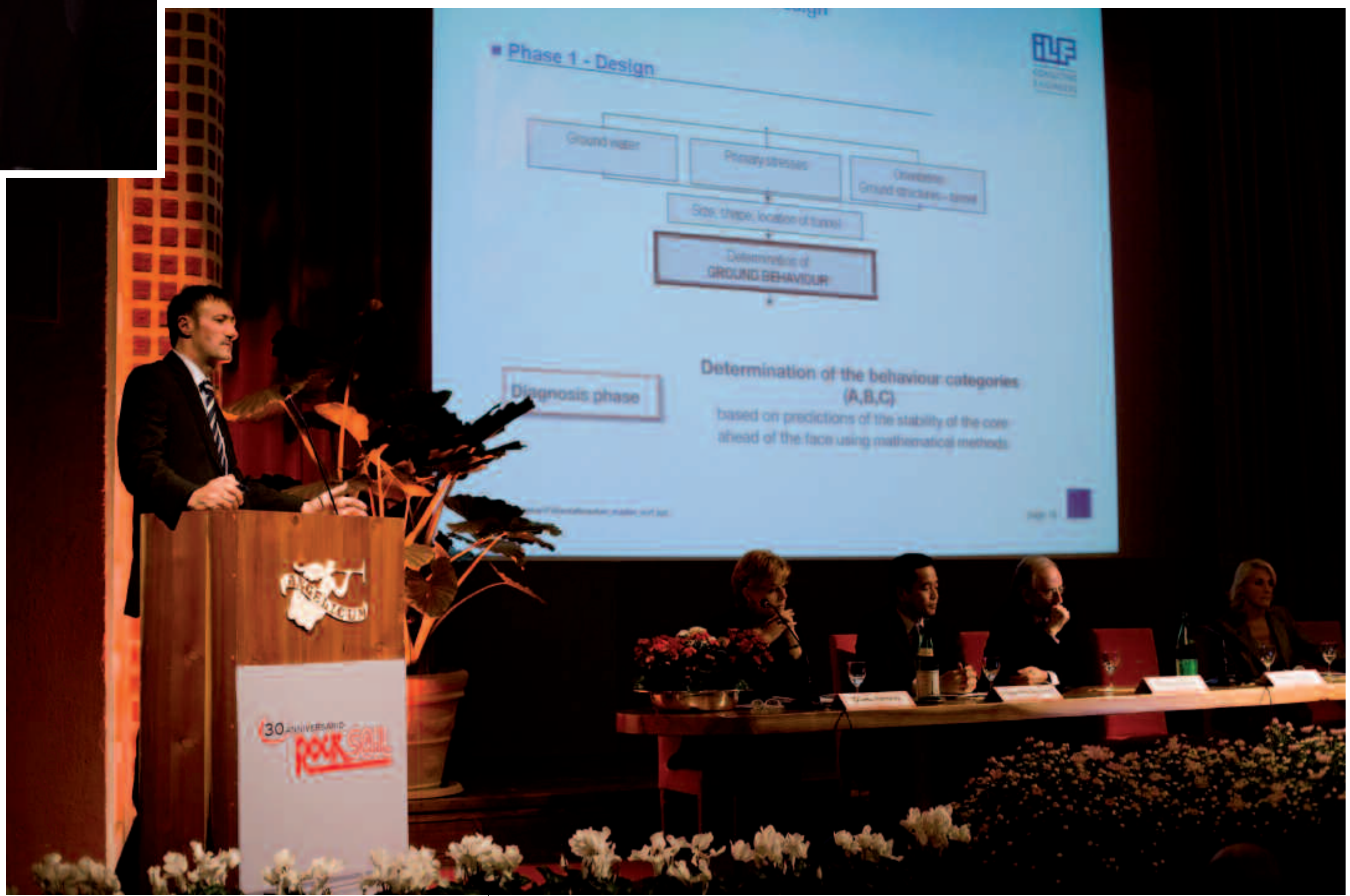
The common approach for designing tunnels in a specific country is usually governed by the historical background. As parts of Austria are located within the Alps tunnels have been an important issue for more than hundred years now. Within this long period not only construction methods but also design approaches have changed significantly. The current Austrian design approach which is summarised in this paper has been influenced by the historical background but also by remarkable ideas from foreign countries.

1. INTRODUCTION

The construction of tunnels and the development of tunnelling methods within a particular nation are influenced by factors such as:



- Geographical situation, topography, existing infrastructure and buildings, environment, natural resources, geological and hydrogeological conditions
- Demand for underground works
- Technical standards
- Legal status and competence of owners
- Availability of qualified workforce
- Mining tradition
- Contractual views.

Situated in central Europe, Austria is and has always been a vital corridor for the transportation of both people and goods. The country is mainly mountainous and partly hilly, with very few large plains or wide valleys.

In terms of tunnelling, the geological conditions in Austria are generally difficult and tend to change rapidly along a tunnel route. Tunnelling in the Alps means facing high overburden and, in places, heavily squeezing rock.

Since 1950 new materials, namely shotcrete and rock anchors, replaced the old timber support and a permanent, in-situ concrete lining was provided in lieu of the traditional masonry lining. In addition, the standardised use of synthetic membranes and fabrics significantly improved the quality of tunnels in terms of watertightness.

Owners experienced in tunnel design and construction together with specialised contractors developed an on-site decision making procedure based on an observational approach, which became common practice in Austrian tunnelling. This approach often has been criticised as it did not follow the procedures generally used for civil works which include a more or less detailed design and do hardly require decisions on site. In addition the criteria for the on site decisions have often not been described clearly in advance and were not a result of a rational and detailed design but have strongly been dependent on the experience of the people on site.

As many Austrian engineers and construction companies have been working outside of Austria in the last decades where a lot of different requirements regarding the tunnel design had to be fulfilled it became evident that the tunnel design process in Austria should also follow clearly defined rules. Such rules have been defined in the "Guideline for the Geomechanical Design of Tunnels" which now forms the basis of the tunnel design in Austria.

2. GENERAL APPROACH

Modern tunnel design in general is based on the concept that the ground around the tunnel not only acts as a load, but also as a load-bearing element in case its strength is sufficient. Typically excavation and support are continuously adjusted to the ground conditions based on criteria defined during the design process. An important issue is the observation of the ground reaction (deformation) and the comparison with the prognosis. In case the deformation limits of a certain type of support which have been defined during the design phase are reached, the type and / or amount of support and / or excavation sequence have to be changed.

Depending on the project conditions (e.g. shallow soft ground tunnel, deep rock tunnel) and the results of the geotechnical measurements, the requirements for a spe-

cific support are determined. Contractual arrangements have to be flexible to ensure that the most economical type and amount of support is used.
Typical support elements are shotcrete and rock dowels. Steel ribs or lattice girders provide limited early support before the shotcrete hardens and ensure correct profile geometry. Face dowels, shotcrete, spiles or pipe canopies are in-stalled, if ground conditions require support at or ahead of the excavation face.
The subdivision of the excavation cross-section in top heading, bench and invert depends on both ground conditions, as well as logistical requirements to facilitate the use of standard plant and machinery.

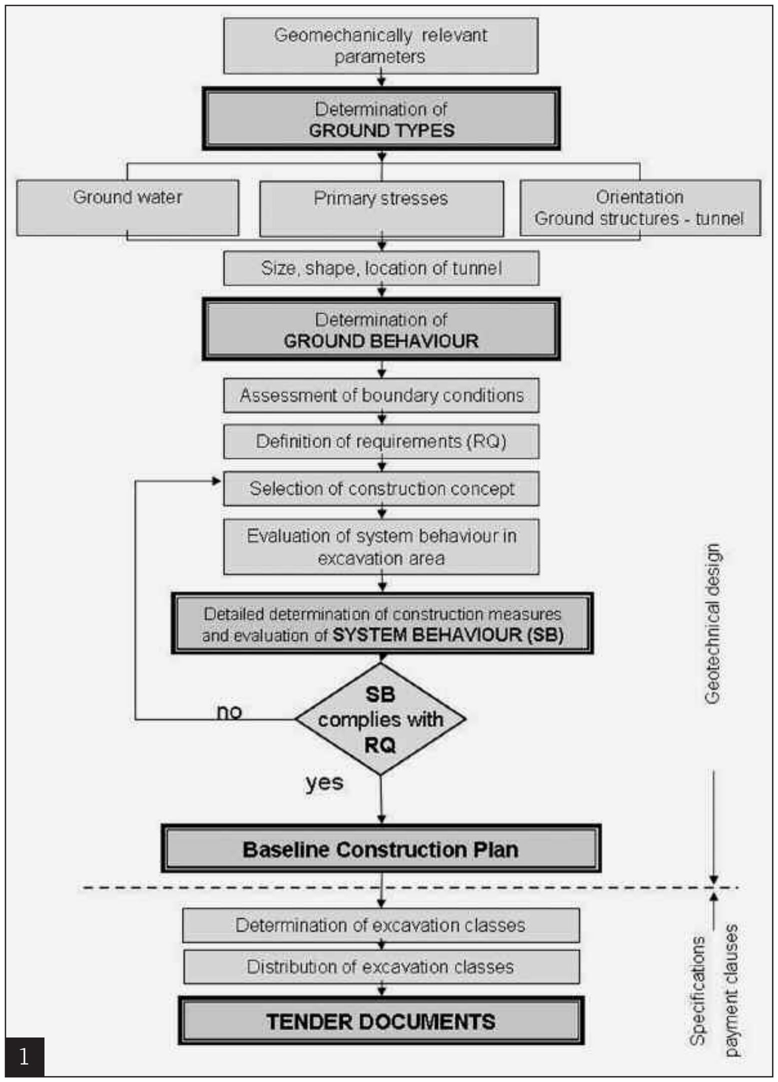
3. DESIGN PHASE

3.1. Geotechnical Design Procedure

The main task of the geotechnical design is the economic optimization of the construction considering the ground conditions as well as safety, long term stability, and environmental requirements. Despite of all uncertainties in the description of

1. DESIGN STEPS

2. TYPICAL GROUND TYPE CHART



1

2

Kriterien / Criterion	Gebirgsart GA IQP-4QP-3c / Type of rockmass GA IQP-4QP-3c		
Lithologie	Quarzphyllit, Chloritphyllit, Quarzitschiefer, Quarzit, Einschaltungen von Schwarzschiefer, Eisendolomit, Kalkmarmor.		
Lithology	Quartz phillite		
Schieferung: Orientierung/Abstand	s 160 - 210/20 - 60 oder 310 - 355/25 - 60		6 - 20 cm
Schistosity: Orientation/Distance	RTF 7s: 160 - 180/65 - 10 oder 320 - 350/65 - 90		0 - 6 cm
Trennflächenorientierung	RTF 2e: 085 - 110/45 - 80	RTF 1b: 135 - 150/50 - 90	RTF 2w,sb: 240 - 280 /45 - 70
Discontinuity orientation	RTF 2w: 230 - 290/40 - 70	RTF 6: 020 - 040/60 - 90	
Trennflächenabstände	RTF 2e: 0 - 0,5 m	RTF 1b: 0,5 - 2,0 m	RTF 2w,sb: 6 - 20 cm
Discontinuity distance	RTF 2w: 0,5 - 5 m	RTF 6: 0,5 - 2,0 m	
Trennflächenlänge	RTF 2e: 0 - 0,5 m	RTF 1b: 0,5 - 2 m	RTF 2w,sb: 0,5 - 2 m
Discontinuity length	RTF 2w: 2 - 5 m	RTF 6: 0,5 - 2 m	
Trennflächenöffnung	RTF 2e: 0	RTF 1b: 0	RTF 2w,sb: 1 mm
Discontunuity opening	RTF 2w: 0	RTF 6: 0	
Trennflächenbeschaffenheit	RTF 2e: 2	RTF 1b: 3 - 4	RTF 2w,sb: 2
Discontinuity characterization	RTF 2w: 1 - 2	RTF 6: 3 - 4	
Gesteinskennwerte	Mittelwert	Standardabweichung	Versuchsanzahl
Parameters of the rock	average value	standard deviation	number of tests
UCS [Mpa]	30		15
mi [-] (Hoek&Brown)	13		16
E [Gpa]	40		14
v [−]	0,18		9
CAI [-]	4,4		4
Quellpotential	keines		
Swelling potential	nil		
Quelldaten (Labor) [MPa]/%			
Swelling data			
Trennflächenkennwerte		Bandbreite	
Discontinuity parameters		Range	
Reibungswinkel [°] / Friction angle [°]	30 - 40°		
Kohäsion [Mpa] / Cohesion [Mpa]	0,10 - 0,50		
Gebirgskennwerte	Mittelwert	Bandbreite	
Rockmass parameters	average values	Range	
ROD (ISRM)	70	60 - 90	
GSI [-] (Hoek)	40	35 - 45	
RMR (Bieniawski, 1999)	65		
UCS [MPa] (Hoek&Brown)	1,0		
c [MPa] (Mohr - Coulomb)	1,8 (H=1000m)		
φ [°] (Mohr - Coulomb)	27° (H=1000m)		
E [MPa] (Hoek 2005 / 2002)	4.000 / 3.100		
E [MPa] (Serafim / Boyd)	*23.700/22.000*		
E [MPa] BLA	15.000	9.000 - 35.000	
E [MPa] empfohlen/recommended	3.000		
c [Mpa] (Mohr - Coulomb) empf./rec.	1,4		
φ [°] (Mohr - Coulomb) empf./rec.	30		
Hinweise	keine Bohrung, kein Aufschluß, grenznah zu Strukurbereich 3b, daher kein ausgeprägter Unterschied		
Notes	no boreholes, no outcrops close to the edge of the structural domain 3b, therefore no evident difference		

the ground conditions, underground engineering needs a strategy, allowing a consistent and coherent design procedure that is traceable throughout the entire project, and allows an adjustment of the construction to the actual ground conditions encountered on site.
The design phase includes determining the expected ground properties, the classification into Ground Types (GT) – which corresponds to the survey phase according to the ADECO-RS approach, the assessment of the Ground Behaviour, its categorization into Ground Behaviour Types (BT) corresponding to the diagnosis phase, as well as the determination of construction and support measures derived from the ground behaviour which corresponds to the therapy phase of the ADECO-RS approach. On this basis the expected System Behaviour is predicted.
Statistic and/or probabilistic methods should be used to account for the variability and uncertainty in the key parameters and influencing factors.
Excavation classes are then determined according to the rules stipulated in the Austrian stan-dard ONORM B2203-1.
The results of all phases of the geotechnical design are summarized in a geotechnical report including a baseline construction plan which clearly has to show, on which ground conditions, boundary conditions, and other assumptions the design is based. In addition it has to contain clear application criteria for the support.

***3.** BASIC CATEGORIES OF BEHAVIOUR TYPES*

3.2. Design steps

Determination of Ground Types (GT)

The design process starts with the definition of ground types, meaning ground sections with similar geotechnical properties. These properties are derived from ground investigations, literature, comparable projects and in addition estimated with engineering and geological judgement. The required quality and accuracy of the properties depends on the design phase (conceptual design, preliminary design, tender design and detail design).

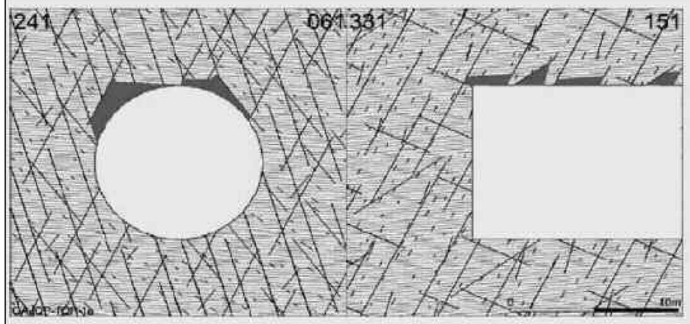
Determination of Ground Behaviour

Within this step the potential ground behaviour is derived for each ground type considering the size of the excavation, the relative orientation of discontinuities, ground water conditions, primary stress situation, etc. The ground behaviour is evaluated for the full cross section without any support or other auxiliary measures. The evaluated project specific ground behaviours are assigned to basic Ground Behaviour Types (refer to table below). Project specific conditions may require a further subdivision of the Ground Behaviour Types.

Basic categories of Behaviour Types (BT)		Description of potential failure modes/mechanisms during excavation of the unsupported ground
1	Stable	Stable ground with the potential for localised gravity induced falling or sliding of small blocks
2	Potential of discontinuity controlled block fall	Discontinuity controlled, gravity induced falling and sliding of blocks in large volumes, occasional local shear failure on discontinuities
3	Shallow failure	Shallow stress induced failure in combination with discontinuity and gravity controlled failure
4	Voluminous stress induced failure	Stress induced failure involving large ground volumes and large deformation
5	Rock burst	Sudden and violent failure of the rock mass, caused by highly stressed brittle rocks and the rapid release of accumulated strain energy
6	Buckling	Buckling of rocks with a narrowly spaced discontinuity set, frequently associated with shear failure
7	Crown failure	Overbreak in the crown of large volumes with progressive shear failure
8	Ravelling ground	Flow of dry or moist, intensely fractured, poorly interlocked rocks or soil with low cohesion
9	Flowing ground	Flow of intensely fractured, poorly interlocked rocks or soil with high water content
10	Swelling ground	Time dependent volume increase of the ground caused by physical-chemical reaction of rock and water in combination with stress relief, leading to inward movement of the tunnel perimeter
11	Ground with frequently changing deformation characteristics	Combination of several behaviours with strong local variations of stresses and deformations over longer sections due to heterogeneous ground (i.e. in heterogeneous fault zones; block-in-matrix rock, tectonic melanges)

3

Gebirgsverhaltenstyp GVT 2: IQP-1QP-1a-nH



Gebirgsarten **Type of rockmass**	IQP-1QP-1a H < 300 m
Orientierung der Haupttrennflächen	überwiegend steilstehende Trennflächen
Orientation of the fractures	above all subvertical
Gebirgsbeanspruchung	0,40 nachbrüchig
Stress of the rockmass	0,40 brittle
Einfluss Bergwasser	kein Einfluss
Influence of the mountain water	no influence
Quelldruck	keiner
Swelling pressure	nil
Gebirgsverhalten	Oberflächliche Ablösungen, örtlich unterschiedliche Ausbrüche entlang Trennflächen, Ortsbrust stabil
Rockmass behaviour	Surface detachment, Diversified local detachments along the discontinuities (fractures), stable face
Tropie / Radialdeformation	isotrop < 5 cm
Tropia / Radial deformation	isotropic < 5 cm

4

4. TYPICAL CHART FOR A GROUND BEHAVIOUR TYPE

Derivation of System Behaviour (SB)

During this step a construction concept is chosen, consisting of excavation method, sequence of excavation, and support measures. With applying different analysis methods (analytical, numerical) the behaviour of ground and support (system behaviour) is analysed and compared with the defined requirements.

These requirements in general are structural stability and serviceability for the underground structure, limited surface settlements but also criteria like vibration and noise. In case the system behaviour does not fulfil the requirements the excavation procedure and support measures are modified until the system behaviour fits to the requirements.

Geotechnical Report – baseline construction plan

Based on the steps above the alignment is divided into sections with similar excavation and support requirements. The baseline construction plan indicates the excavation and support methods available for each section, and contains limits and criteria for possible variations or modifications on site.

Determination of excavation classes

In the final step of the design process excavation classes are defined, based on the evaluation of the excavation and support measures. The excavation classes form a basis for compensation clauses in the tender documents. In Austria the evaluation of excavation classes is based on the regulations in the Austrian code ONORM B2203-1.

4. CONSTRUCTION

4.1. General

During construction geotechnical relevant ground parameters have to be collected, recorded, and evaluated to determine the ground type and the System Behaviour according to the stipulations of the design. Excavation and support measures have to be chosen based on the criteria laid out in the geotechnical report and the safety management plan.

The geotechnical design and the baseline construction plan have to be continuously updated based on the findings on site. The improved quality of the geotechnical model allows an optimization of the construction while observing all safety and environmental requirements. The relevant data and assumptions made for all decisions during design and construction have to be recorded. Relevant information in connection with the ground properties, ground and system behaviour has to be collected in a way, that a review of the decision making process is possible.

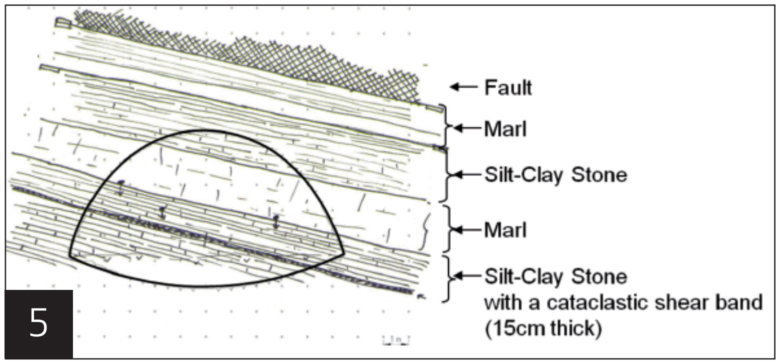
4.2. Geotechnical assessment during construction

Determination of the encountered Ground Type and Ground Behaviour

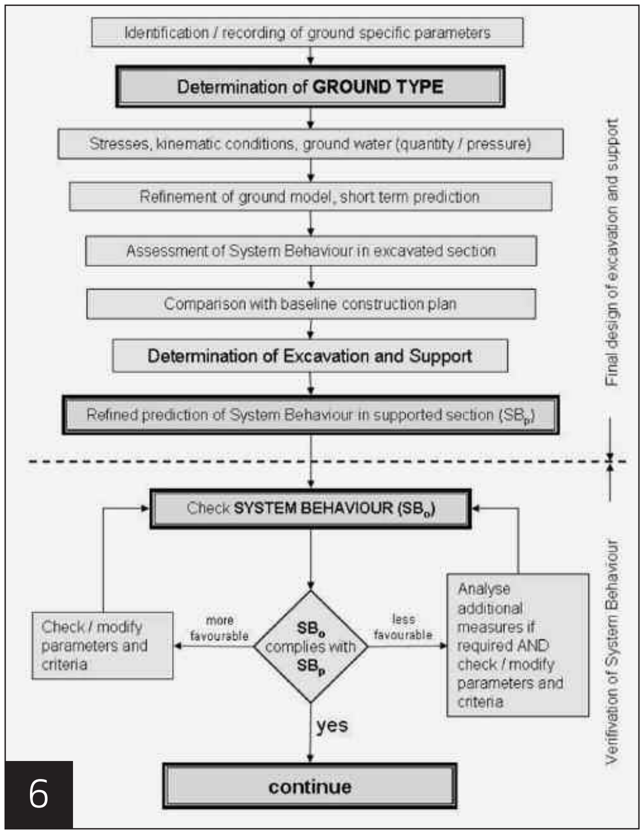
To be able to determine the encountered Ground Type, the geological documentation during construction is targeted to collect and record the relevant parameters specified in the design. Additional observations, like indications of over-stressing, deformation and failure mechanisms, as well as results from probing ahead and the evaluation of the geotechnical monitoring are used to determine the ground behaviour type.

Determination of excavation and support measures

To determine the appropriate excavation and support the criteria laid out in the baseline construction plan have to be followed. Consequently, it has to be checked if the actual ground conditions (ground type, ground behaviour type) comply with the prediction.
The goal is to achieve an economical and safe tunnel construction. The system behaviour has to be predicted for the next excavation section, considering ground conditions, and the chosen construction measures. Records have to be kept on this process.
Note: both, excavation and support, to a major extent, have to be determined prior to the excavation. After the initial excavation only minor modifications, like additional bolts, are possible. This fact stresses the importance of a continuous shortterm prediction.



5. EXAMPLE OF FACE MAPPING WITH EXTRAPOLATION SHOWING THE SURROUNDING GROUND



6. FLOW CHART OUTLINING THE PROCEDURE DURING CONSTRUCTION

(SBP=PREDICTED SYSTEM BEHAVIOUR; SBO=OBSERVED SYSTEM BEHAVIOUR)

Verification of System Behaviour

By monitoring the system behaviour the compliance with the requirements and criteria defined in the geotechnical safety management plan (please refer to item 7) is checked. When differences between the observed and predicted behaviour occur, the parameters and criteria used during excavation for the determination of the ground type and the excavation and support have to be reviewed. When the displacements or support utilization are higher than predicted, a detailed investigation into the reasons for the different system behaviour has to be conducted, and if required mitigation measures (like increase of support) ordered. In case the system behaviour is more favourable than expected, the reasons have to be analyzed as well, and the used parameters modified if appropriate. This allows for a continuous improvement and refinement of the method for assignment of excavation and support methods.

5. GEOTECHNICAL SAFETY MANAGEMENT

The stability of the underground structure is a key concern during design and construction. The measures employed to ensure stability will vary depending on the geotechnical and on other boundary conditions.
In the design of underground structures, the geotechnical conditions, the structural system and the load bearing capacity of ground and support can be quite variable. As such, the design of underground structures cannot be compared to the structural design of buildings, where the loads and the material properties are well known.
Uncertainties in the geotechnical model increase the risks associated with underground construction. To minimize these risks, a safety management system shall be implemented.
The safety management system must cover the following topics:

- Design requirements for excavation and support; criteria for the assessment of the stability during construction;
- Monitoring plan which includes all technical and organizational measures to allow continuous comparison between the expected and actual conditions;
- Management plan to deal with deviations from the predicted/acceptable System Behaviour.

The Safety Management Plan must include the following elements:

- Definition of the expected system behaviour during construction
- Definition of relevant parameters to be measured
- Definition of the expected measurement results and acceptable deviations (trigger values, alert levels)
- Determination of the evaluation methods of monitoring data
- Management of the data evaluation process (collection of data, evaluation, interpretation and communication between the parties)
- Contingency measures in case of deviations from the expected or acceptable System Behaviour
- Procedure to be followed in the event that trigger values / alert levels are exceeded (definition of lines of report and command)

6. MONITORING - DATA EVALUATION

6.1. General

Systematic monitoring is an important component of the approach. Monitoring is necessary to verify the appropriateness of the applied excavation and support to confirm tunnel stability. Monitoring is achieved using optical 3D-displacement measurements in conjunction with sophisticated methods of data processing and visualisation.
In addition to optical monitoring the following geotechnical instruments are used:

- extensometers
- sliding micrometers
- inclinometer
- load cells (anchor forces)
- strainmeters
- piezometers

6.2. Time - Displacement Diagrams

Time - displacement diagrams are used to present vertical, horizontal and longitudinal displacement components versus time.
Construction phases are also presented on the same plot so that construction activities can be correlated with displacement trends.

6.3. State Diagrams (Influence Curves)

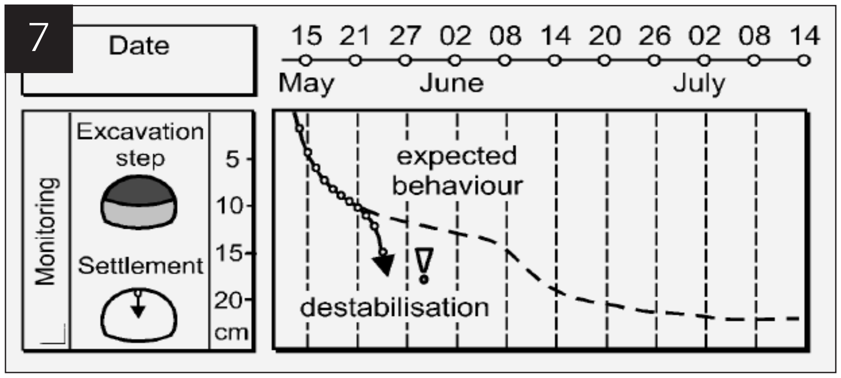
State diagrams are created by plotting a series of displacements measured at a particular target position, at a particular point in time. Results of a specified period are presented on one plot.
By showing several influence curves on the same plot, it is possible to compare displacements along the tunnel. Information on the longitudinal extent of tunnel deformation is provided. Trends of relative decreasing or increasing rock mass stiffness can be verified.

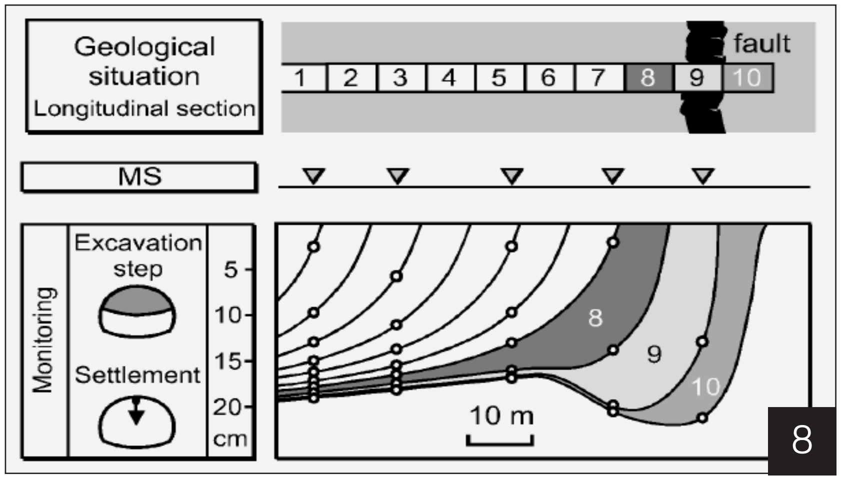
6.4. Vector Plots

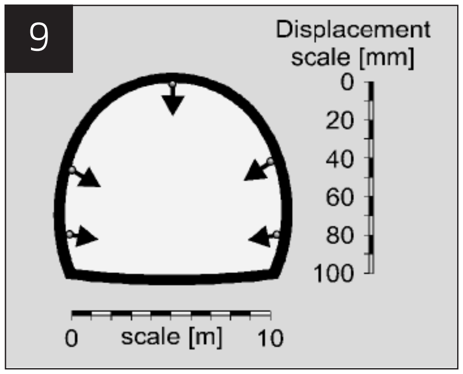
Vector plots evaluated for a monitoring cross section are used to show the orientation of displacements as well as the development of displacements with time.
The displacement vector orientations in a monitoring cross section provide additional information to aid in the identification of weak zones outside the tunnel excavation.

7. SUMMARY

The Austrian guideline for the geomechanical design of underground structures, originally published in 2001 has now been used for about 8 years mainly in Austria but also in foreign countries.
This guideline has brought the required rules into the Austrian approach of tunnel design and forces the designer, the construction supervision and the contractor to follow a defined path. Nevertheless each tunnel project will be unique and a challenging task for clients, designers and contractors.







7. *TIME - DISPLACEMENT DIAGRAM SHOWING EXPECTED BEHAVIOUR AND INDICATING DESTABILISATION*

8. *INFLUENCE CURVES WHEN EXCAVATION APPROACHES A FAULT*

9. *VECTOR PLOT*

BIBLIOGRAPHY

AUSTRIAN SOCIETY FOR GEOMECHANICS
Division "Tunnelling"
Working Group "Conventional Tunnelling"
The Austrian Practice of Conventional Tunnelling

PIETRO LUNARDI
Design and Construction of Tunnels - Analysis of Controlled Deformation in Rock and Soils (ADECO-RS)

Juan J. Schmitter

TUNNELING IN THE SOFT CLAYEY SUBSOIL OF MEXICO CITY

Five decades ago there was built the first tunnel, in the very soft clayey subsoil of Mexico City. Today, more than 100 km of tunnels have been built there and almost 70 km more are now under construction. Several shield machines have been developed in time, to dig in the area. There are two mayor geotechnical uncertainties for design caused by the "general subsidence phenomena" and by the "sudden appearance of tension cracks".

1. INTRODUCTION

The search and use of the underground space has always been present in the minds of all urban authorities, especially in crowded and fast growing, cities.

Mexico City is not an exception for that searching, but in this case, the very soft clayey subsoil where the mayor portion of the City is founded, delayed for a long time, the benefit of that strategic use.

PROF. ING. JUAN JACOBO SCHMITTER,
ICA, GEOTECHNICAL ADVISOR

In the early sixties of last century, when the main drainage waterways were solved with open channels, the need for deeper drainage conduits, forced the authorities to look for new construction schemes, and as a result, after a short period of searching, the shield concept for tunneling was "discovered", and several locally designed and manufactured, shields were successfully put in operation.
Probably the first tunneling experiences in the City started back in 1960, when the urban stretch of an open drainage channel, was replaced by a more environmentally friendly and safer, underground tunneled conduit.
Since then, several big tunneling projects, like the Deep Drainage System (DDS) and the first Subway lines, were successfully excavated in that soft and very soft, clayey environment, also strongly affected by the seismic activity of the zone.

2. CLAYEY SUBSOIL OF MEXICO CITY

As described by Mooser (1978) volcanic emissions deposited in a lacustrine environment, formed in time the unique very soft material, which is world wide known as Mexico City Clay.
During the decade of the forties, two outstanding geotechnical researchers, Leonardo Zeevaert and Raul J. Marsal, have explored systematically the subsoil of Mexico City, extracting undisturbed samples, and carrying out many laboratory tests, to find out static as well as dynamic properties of the clay.
Dr. Leonardo Zeevaert, has been involved mainly in the analysis and design of tall buildings of the City, including the very important input of the seismic action on the area. He has written many technical articles and Books, and one of the most well recognized is: Foundation Engineering for Difficult Subsoil Conditions (1972).
Dr. Raul J. Marsal (†) wrote also many technical articles, and a "Classic" very comprehensive book titled: "El Subsuelo de la Ciudad de México", that includes many statistical properties of the subsoil of Mexico City, as well as many interesting experiences on the behavior of buildings.

TABLE 1. *SIMPLIFIED STRATIGRAPHY OF MEXICO CITY SUBSOIL*

Strata	Short description	Depth	Natural water content	Unconfined compressive strength	Average unit weight	Deformation modulus
-	-	m	%	kPa	kN/m^3	MPa
Superficial crust	Silty soils, with some sand, and eventually clay	0 to 5	92	87	13,1	5,7
Upper clayey deposit	Soft, and very soft bentonitic clay, with high compressibility	5 to 30	281	72	11,5	3,0
First hard layer	Very stiff sandy silt, and dense or medium dense silty sand, of low compressibility	30 to 33	64	131	14,4	6,5
Lower clayey deposit	Soft bentonitic clay, with high compressibility	33 to 41	192	153	12,0	6,6
Deep deposits	Very stiff sandy silt, and dense silty sand	Below 41	63	176	13,6	9,1

2.1 Simplified stratigraphy of Mexico City

In accordance with Marsal and Mazari (1959), a simplified stratigraphy of the Mexico City subsoil, in the so called lake zone, can be expressed as shown in Table 1.
In the following paragraphs, three peculiar geotechnical characteristics of the Mexico City clayey subsoil are briefly described.
Those peculiarities play important roles in the analysis and design of tunnels, and also in their construction procedures.

2.2 Very high water content

As shown on Table 1, one of the most remarkable physical characteristics of Mexico City clayey soil is its very high natural water content, with an average value around 281 %, and with maximum values around 600 % for the Upper clayey deposit (almost three to six times the weight of the natural water contained in the soil for one time its solid portion.
Nor less surprisingly is the numerical value of its Liquid Limit that usually is practically equal to its natural water content. So, it follows that technically speaking, the Mexico City Clayey subsoil, is a "liquid".
Of course, thanks to its internal structure, which gives an average sensitivity ratio of 8, and a noticeable elastic behavior, the subsoil of México City does not behave as a liquid, even during the occurrence of strong motions caused by seismic action.
Nevertheless, any contractor on deep excavations or tunneling knows that if he does not apply a smooth and careful procedure, large deformations can be induced in the clayey soil mass, destroying its internal structure and thus transforming it into a real liquid, with the unfortunate consequences of that change.

2.3 General subsidence phenomena

As pointed out by Marsal and Mazari (1959), the intensive ground water extraction by pumping within the City limits, for supply purposes, is causing since the beginning of the past century, a general subsidence phenomena, that is reflected on the surface of the City, as a continuous settlement, that yearly amounts 10 to 40 or more centimeters.
This phenomenon also affects proportionally the vertical displacement of the intermediate layers of the compressible subsoil, thus affecting the behavior of tall buildings resting on deep foundations, and also tunnels excavated within the compressible strata.
It is well known that this very unfavorable phenomenon, have reversed historically the hydraulic gradient of the main open waterways for drainage, that usually work by gravity, and as a result, during the decade of the forties of past century, persistent flooding appeared during the rainy season in some down town areas of the City, that needed heavy pumping to be cleared.
To solve this problem, the authorities studied, planed, built and finally put in operation, an almost 143 km long, Deep Drainage System (DDS), which is a net of deep tunneled interceptors with a finished internal diameter, of 500 cm, located under the city and connected to a Central Outfall with a finished diameter of 650 cm, that disposes off the served and the rainy waters, out of the city.
The construction of that Deep System, started back in the decade of the sixties, and since then has been enlarged continuously, as needed.

2.4 The sudden appearance of tension cracks

Another important phenomena that usually affects the Upper Clayey Deposit, especially when the Superficial Crust is practically inexistent, is the sudden appearance of tension cracks.

As explained by Juarez Badillo (1959), this long and relatively deep, tension cracks, appear suddenly in non, or almost non urbanized areas, where there are not to much paved surfaces, and neither surface drainage system. Those cracks usually appear after a period of intense superficial evaporation of the water contained in the soil, followed by the beginning of the annual rainy season that forms thin water puddles.

Those cracks can be more than 100 m long, with openings at the surface, on the order of 0.1 to 1.0 m and can reach 10 to 30 m, below the surface, depending on the position of the ground water level, and the strength properties of the clayey subsoil.

3. EARLY TUNNELING EXPERIENCES IN MEXICO CITY

When in the decade of the Sixties, the authorities of the city decided to "conquer" the underground space, there were not any experience at all about tunneling in soft ground and neither the concept of the shield has been known.

So the first contractor assigned for that task, applied a conventional tunneling procedure, placing primary lining formed by steel ribs an lagging, within a portal tunnel section, with no special footing precautions at the base of the steel ribs. Surprisingly it was possible to excavate a few meters of the tunnel, with that inappropriate method, until the state of stresses reached a limit, and the incipient primary lining collapsed, probably starting at the bottom, and then progressing upwards to finally ending with the complete closure of the incipient opening. Fortunately no human injuries occurred because this accident happened during one weekend.

After this very illustrative failure, the construction practitioners of that time, looked upon the use of the shield concept, propelled by shove jacks, and placing small concrete segments, as initial lining.

As a result of that initiative, several shields were locally designed and built, using commercial dump truck jacks, that were successfully operated, to build the more than two kilometers long, and four meters of external diameter, tunnel, that replaced the southern extension of the Great Chanel, open waterway drainage of the City.

3.1 The grid shields

One of the most ingenuous shield designs, that have worked successfully in the clayey subsoil of Mexico City, is the Grid Shield (Fig. 1), designed by Cravioto and Villareal (1969).

While advancing this "cookie cutter" shield into the natural subsoil, there were formed prismatic blocks of the clayey soil that entered into the shield body. Then a mechanical mixer remolded and transformed those blocks into a liquid mud, and by adding extra amount of water, all the excavated soil and the additional water, were mucked out by centrifuge pumping.

1. GRID SHIELD
DESIGNED BY CRAVIOTO AND VILLAREAL (1969).

The pushing action of the shield stabilized the face, thanks to the frictional forces developed between the soil and the steel plates of the grid.
Behind the grid, there was a movable screen that in case of unexpected instabilities could be closed to protect the tunnel against intrusions.
The initial support for a 400 cm diameter shield of this concept included fourteen concrete blocks per ring, 40 cm wide, 20 cm thick and a wooden key.
Between the years 1961 and 1975, this type of shields, with diameters between 200 and 400 cm, have excavated almost 10 kilometers of tunnels, mainly for drainage. Its monthly rate reached almost 200 m.

3.2 Open face shields with front jacks

During the decade of the seventies, an American tunneling consultant, A. Chase (1973), designed two hand mined, open face shields, for the Mexico City subsoil, that included breast jacks to pressurize/stabilize the face, while shoving the machine.
The first shield design, with an external diameter of 915 cm, was used for the construction of an almost 1.2 km long, tunneled portion of Line No 1 of the Mexico City Subway, in a non clayey zone, placing three expandable concrete segments per ring, 80 cm wide an 25 cm thick, as initial an final lining (Fig. 2). The external diameter of the lining was 900 cm.

2

__2.__ MEXICO CITY METRO LINE N°1

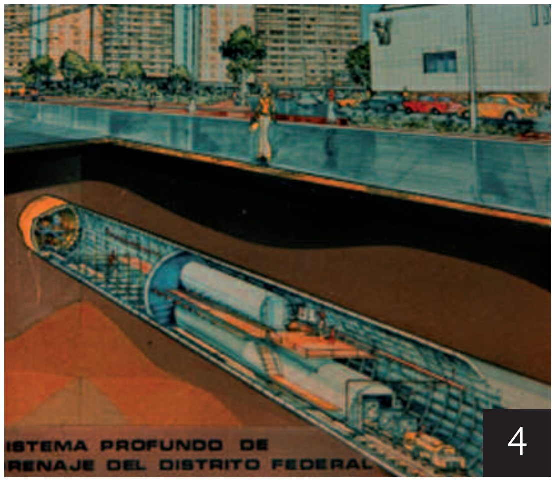
The second shield design, with an external diameter of 624 cm (Fig. 3) was used for the excavation of several stretches of the Central Interceptor drainage conduit, for Mexico City, totalizing almost 16 km.
For that task, half a dozen of those shields were locally manufactured, and put successfully in operation. The initial support for those tunnels had an external diameter of 610 cm, and included eleven pieces plus a key per ring, 75 cm wide and 25 cm thick. The final lining was formed by cast in place reinforced concrete, 30 cm thick, thus leaving an internal diameter of 500 cm. Those shields worked in a non clayey as well as in clayey, zones. To stabilize the face in the clayey zones, an auxiliary compressed air system (Fig. 4) was used.
The compressed air system applied maximum pressures around 127 kPa (considered a limiting value for Mexico City) and included two locks (for personnel and muck), mounted on a pressure bulkhead installed inside the tunnel, and a compressed breathable air, plant, placed at the surface, near the shaft entrance.

3

__3.__ OPEN FACE SHIELD (O.D. 624 CM) USED IN THE DEEP DRAINAGE SYSTEM OF MEXICO CITY

Those open face shields provided a very interesting way to see in detail the stratigraphic profile of the lacustrine clayey soils, where bones of extinct elephants, as well as pieces of wood from trees, were eventually found. One sample taken at 22 m below the surface, looking as a branch of a tree, was send to the Teledyne Isotopes Laboratory (1980), and by the radiocarbon technique an age of 25,280+/-750 years, was determined for it.

Between the years 1972 and 1986, this type of shields, with diameter of 624 cm, have excavated almost 15.7 km, of tunnels for the DDS, 55% of those length in soft clayey soils, using compressed air to stabilize the face. Its average monthly rate reached 116 m without compressed air, and 98 m, while using that auxiliary system.



4. COMPRESSED AIR SYSTEM USED AT MEXICO CITY

4. PRESENT EXPERIENCES

Since the unconfined compressive strength of the clayey subsoil, has rather low values in some zones of the City, for example less than 50 kPa, the value of the air pressure needed for stabilization, exceeds 127 kPa, which is considered a limiting value, from the point of view of human behavior and economics.
So, in order to built tunnels in those zones of very soft clay, there was a search for new tunneling procedures, and at the end of the decade of the seventies, a group of international designers integrated by American, Mexican and Japanese, companies, designed a 624 cm Slurry Shield, to work in the soft and very soft, clayey soils of México City.

4.1 The slurry shields

Following the very interesting experiences obtained during the before mentioned design, in 1984, a 400 cm external diameter slurry shield (Fig. 5), manufactured in Japan, by the company Okumura, carried out very successfully, its first tunneling task, a 5.3 km long sewer at an intermediate depth of 12 m.
The initial support for that tunnel was designed under the concept of normal and corrective, concrete segment rings, having an external diameter of 385 cm, including five pieces plus a key per ring, 100 cm wide and 17.5 cm thick. The final lining was formed by cast in place reinforced concrete, 15 cm thick, thus leaving a final internal diameter of 320 cm.
It is worth to say that the bentonitic natural clayey soil of Mexico City was used to form the slurry needed for the circulation system. For that purpose it was necessary to add water, at a rate by weight of one part of water for one part of excavated soil. The muck removal was achieved by centrifugal pumping. Following this very successful experience, two more slurry shields with a diameter of 624 cm, also manufactured in Japan by the company Okumura, were acquired in 1987 by the authorities of the City, to dig the rest of the tunneled Interceptors of the DDS of the City, in its softest clayey zones.

5. SLURRY SHIELD (O.D. 400 CM)

The initial support for those machines was designed also under the concept of normal and corrective, concrete segment rings, having an external diameter of 610 cm, and include five pieces plus a key per ring, 100 cm wide and 25 cm thick. The final lining was formed by cast in place reinforced concrete, 30 cm thick, thus leaving a final internal diameter of 500 cm.
Between the years 1984 and 2006, those slurry shields, with diameters of 400 and 624 cm have excavated almost 60 km of tunnels, mainly for the DDS. Its average monthly rate reached almost 360 m.

4.2 The earth pressure balance, shields

One unfavorable byproduct of the slurry shields is the liquid/muddy effluent that needs to be recycled in large treatment and separation plants, before sending it to a disposal site.

The earth pressure balance shields, produces a more "solid" effluent, that doesn't need large treatment plants, and is friendlier with the environment.

For this reason, when in 2007, another 6.7 km long tunneled drainage conduit, was required, as a replacement for an open surface channel, a 630 cm outside diameter, earth pressure balance, shield (Fig. 6), manufactured by the German company Herrenknecht, was selected for that task.

The initial support for that tunnel was designed under the concept of universal rings, having an external diameter of 610 cm, and includes five pieces plus a key per ring, 150 cm wide and 25 cm thick. The final lining was formed by cast in place reinforced concrete, 30 cm thick, thus leaving a final internal diameter of 500 cm.

The muck removal of the clayey material was provided by two piston pumps that expelled the excavated material directly to the ground surface, where it was loaded on trucks and sent to the disposal area.

The high water content of the clayey soil is not enough to make it "piston pumpable", so a small quantity of water, around one tenth of the soil weight, was added for that purpose.

Between the years 2007 and 2009 that shield have excavated 6.7 km of a drainage tunnel, in a very complicated geotechnical zone, that caused significant delays on the execution of the Project. Nevertheless, its maximum monthly rate in a favorable geotechnical zone, reached 685 m.

6

***6.** EARTH PRESSURE BALANCE, SHIELD (O.D. 630 CM)*

5. LEARNED LESSONS

During the almost 50 years of tunneling experience developed in the very soft clayey subsoil of Mexico City, and surrounding areas, many lessons have been learned, that progressively have improved the way of making tunnels in the area, safer and faster. Several uncertainties, coming from the geotechnical characteristics of the Mexico City Subsoil, and its peculiar behavior, have been present and those needed to be understood, analyzed and eventually solved, to obtain proper solutions, for future Projects.

There are also some advantages, like the bentonitic nature of the high water content clayey soil, that have been favorably used while operating slurry shields and the relatively easy conditioning of the natural clayey soil, by adding a small quantity of water to make it piston pumpable.

In the following paragraphs, there are presented the main uncertainties associated to the tunneling works in the soft clayey subsoil of Mexico City.

7. INCREASED VERTICAL PRESSURE ON LINING, CAUSED BY THE GENERAL SUBSIDENCE PHENOMENA

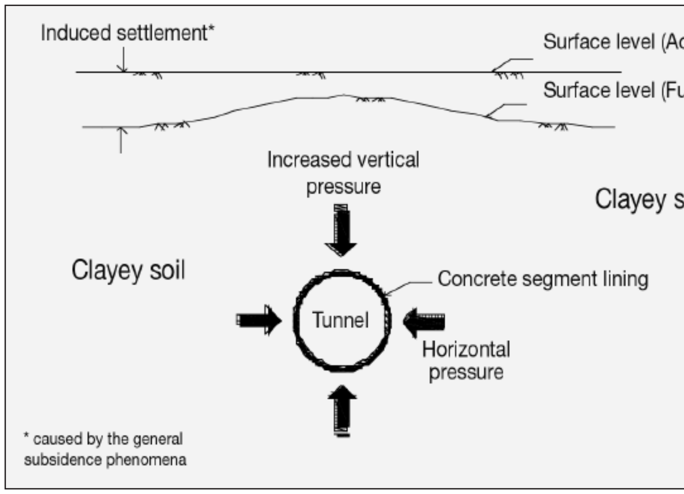
8. DECOMPRESSION OF LINING, CAUSED BY THE SUDDEN APPEARANCE OF TENSION CRACKS

5.1 General subsidence phenomena

This unfavorable phenomena that causes a progressive settlement on the surface of the City (Fig. 7), can also induce differential displacements between the intermediate strata which should be taken into account during the analysis and design of initial and final linings, of a tunnel, to be placed in those strata.

In some cases, the initial lining formed by concrete segments, will receive the favorable aid of a final lining, to resist the stresses induced by the mentioned differential displacements.

If the initial lining is also the final one, the strengthening problem is more difficult to solve. In both cases, the design should consider the total amount of differential displacements that can occur during the life time of the tunnel.



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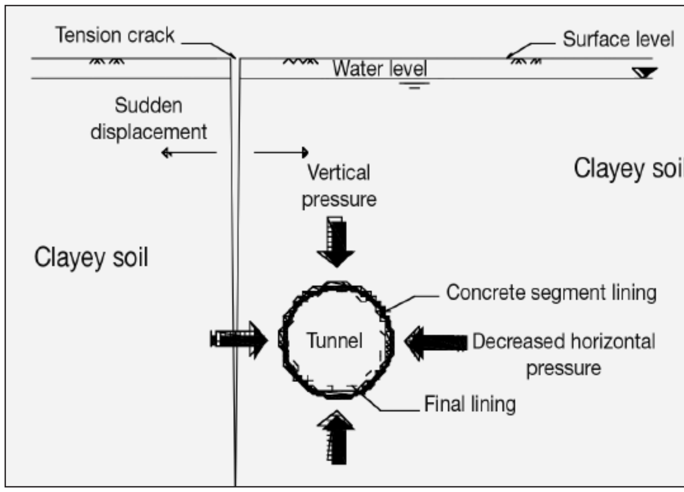
5.2 The sudden appearance of tension cracks

This phenomenon that occurs within the non urbanized zones that surrounds Mexico City (Fig. 8), can cause an undesirable progressive lateral decompression of the initial support, inducing large diametric deformations on the order of 3% or more, of the original diameter of the tunnel.

To solve this problem, it is necessary to increase the rigidity of the concrete segments, and their corresponding joints. The placement of a final lining can be designed to stop the undesirable progressive deformation of lining.



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6. TUNNELING PROJECTS, IN PROGRESS

Today, there are now in construction, two big shield tunneling projects in the area of Mexico City and vicinity, the Eastern Outfall an almost 62 km long tunnel with an internal finished diameter of 700 cm and the underground 7.7 km long stretch of the Line #12 of the Mexico City Metro, that would leave an internal diameter of 911 cm.

6.1. Eastern Outfall

Six earth pressure balance shields, three from the German manufacturer Herrenknecht and three from the American/Japanese manufacturer Robbins/Mitsubishi, will be used for the excavation of this long tunnel. Each TBM will excavate almost 10 km.
Twenty three shafts and one portal will be needed for the excavation of this tunnel. At the initial shaft, the tunnel depth at axis will be on the order of 19 m and at the exit portal almost 100 m more.
In the shallow zone of the outfall, the initial support for that long tunnel is designed under the concept of universal rings, and it will have an external diameter of 840 cm, including six pieces plus a key per ring, 150 cm wide and 35 cm thick.
The final lining will be cast in place reinforced concrete, 35 cm thick, thus leaving a final internal diameter of 700 cm.
In other, deeper zones of the tunnel the thickness of the concrete segments will be 40 cm.

6.2. Underground stretch of Line #12

One earth pressure balance shield with an external diameter of 1019 cm, from the German manufacturer Herrenknecht, will be used for the excavation of this 7.7 km long tunnel.
The average depth of this tunnel at axis will be on the order of 14 m.
The initial support for this tunnel is designed under the concept of universal rings. It will have an external diameter of 991 cm, including seven pieces plus a key per ring, 150 cm wide and 40 cm thick. In some zones the initial support will be also the final one.

BIBLIOGRAPHY

CHASE, A., 1973. Diverse comunicazioni orali e scritte.

CRAVIOTO, M. & VILLAREAL, E., 1969. Recent experience in the construction of tunnels and shafts in the city of Mexico. Contributo al 7° Congresso ICSMFE. México, D.F.

JUÁREZ BADILLO, E. 1959. Teoría de grietas en tensión. Primer Congreso Panamericano de Mecánica de Suelos e Ingeniería de Cimentaciones. México, D.F.

MARSAL, R.J. AND MAZARI, M. 1959. El subsuelo de la Ciudad de México. Contributo dell'Instituto de ingeniería de la UNAM, al Primer Congreso Panamericano de Mecánica de Suelos e Ingeniería de Cimentaciones. México., D.F.

MOOSER, F., 1978. Geología del relleno cuaternario de la cuenca de México. El subsuelo y la Ingeniería de Cimentaciones en el área urbana del Valle de México, p, 9-13. Sociedad Mexicana de Mecánica de Suelos. México.

TELEDYNE ISOTOPES, NEW JERSEY. 1980. Sample number 1-11, 788. Determination of age, by the Radiocarbon method.

SCHMITTER, J.J. 1992. Compressed air and slurry shield tunneling, at Mexico City. Proceedings of the International Congress, Towards New Worlds in Tunneling. A.A Balkema / Rotterdam / Brookfield.

ZEEVAERT, L. 1972. Foundation engineering for difficult subsoil conditions. Van Nostrand Reinhold Company.


A
C

It is a great pleasure to have been able to pass an entire day together with friends and colleagues to celebrate thirty years of Rocksoil. You will certainly have noticed that it was not a simple celebration to take note of the time that has passed. It was a day dedicated to engineering and scientific thinking on the real contribution made by our consulting engineering firm on the subject of the design approach to underground works over the last thirty years. To listen today to a series of contributions from outstanding representatives of the world of tunnelling has once again convinced me that the design approach we have developed over the years is genuinely cutting edge and that it will provide material for study for engineers all over the world. Our scientific and engineering experience is the result of 30 years of work with over 800 km of tunnel designed under all stress-strain conditions, thanks to the specialisation and work of all of us! Here I think it is important to underline that when at Rocksoil we speak of kilometres of tunnels designed, these kilometres are always of work successfully completed even under the most extreme stress-strain conditions and therefore we are speaking of concrete achievements and not just feasibility studies or preliminary designs which remain mere lines on paper. Our designs, which originate to satisfy construction site and client requirements, are the result of our scientific and engineering expertise which is based on study and respect for the equilibriums found in nature. This way of working distinguishes us from all other consulting engineering firms operating in the sector and it is why still today our clients prefer to call us for our support in the study of the most important engineering projects in Italy and the world.

I am convinced that it is this experience that we have acquired that will allow us to be here for another thirty years, in the hope of celebrating, with all of you and with many future colleagues, who could even be your children, our sixtieth birthday!

Giuseppe Lunardi

- JET-GROUTING
  sub horizontal and sub-vertical
- MECHANICAL PRECUTTING
  full face
- “RS METHOD”
  of a pilot tunnel
- REINFORCEMENT OF THE ADVANCE CORE
  with fibre glass structures
- CELLULAR ARCH
- ARTIFICIAL GROUND
  for low-overburden tunnels
- “NAZZANO METHOD”
  to widen tunnels without interrupting traffic

PART TWO
THE TECHNOLOGIES

INTRODUCTION

The new technologies developed by Rocksoil S.p.A. as part of its design and construction approach entitled the Analysis of Controlled Deformation in Rocks and Soils (ADECO-RS)

PART OF THE REINFORCED FACE WITH FIBRE GLASS REINFORCEMENT

TARTAIGUILLE TUNNEL – FRANCE

∅ = ~ 15.30 M.
GROUND: CLAY
MAX. OVERBURDEN: ~ 110 M.

As Prof. Fulvio Tonon observed in his paper published in the first part of this book, the new technologies conceived and designed by Prof. Ing. Pietro Lunardi, and introduced by Rocksoil S.p.A. into the world of tunnelling, have made Rabcewicz's dream of driving tunnels full face even and above all under difficult stress-strain conditions become a reality.
But what are these technologies?
They are technologies termed "cavity preconfinement", conceived and developed in Italy, in the early 1980s as a natural consequence of in-depth theoretical and experimental research conducted by Rocksoil S.p.A. under the leadership of Prof. Ing. Pietro Lunardi. As opposed to the exclusive way in which it had been considered until that time, research had focused for the first time on the deformation response that developed in tunnels in all its components (extrusion, preconvergence and convergence) and not just in the plane transverse to the direction of tunnel advance (convergence).
By employing these operational technologies in a consistent theoretical and experimental framework, ADECO-RS demonstrated that it was able to overcome the limitations of previous approaches, finally making it possible to industri-

alise excavations (predictability of construction times and costs), regardless of the type of ground and the stress-strain conditions to be addressed. This produced a scientific and technological leap forward in the field of tunnelling, similar to that which occurred at the beginning of the 1900s with the introduction of simple cavity confinement techniques, such as shotcrete, steel ribs and steel rock bolts, which made it possible to tunnel under conditions which previously would have been very difficult to tackle.

The experience acquired by Rocksoil S.p.A. in the construction on time and to budget of major works including the project to cross the Apennines with a new high speed/capacity railway line, which was exceptional in terms of its size and the heterogeneity and difficulty of the ground conditions, has demonstrated that the objective of industrialising tunnelling is finally possible even under difficult stress-strain conditions and even when it is not possible to employ TBMs. However, it is necessary for the new technologies of which Rabcewicz dreamed to be employed properly, within the framework of design approaches like that of ADECO-RS and consistently with the principles on which they are based.

APPIA ANTICA TUNNEL
ROME MOTORWAY RING ROAD
∅ = 20.65 M.
GROUND: GRANULAR PYROCLASTITES
MAX. OVERBURDEN: ~ 4 M.

APPIA ANTICA TUNNEL
ROME MOTORWAY RING ROAD
∅ = 20.65 M.
GROUND: GRANULAR PYROCLASTITES
MAX. OVERBURDEN: ~ 4 M.

This is the only way in which a programmed approach can be adopted to tunnelling under conditions that are impossible or extremely difficult to address using conventional methods, because of the huge deformation that would be triggered at the face and around the cavity.
On the basis of these considerations, it was felt appropriate to dedicate the second part of this book to the new preconfinement technologies developed over the years by Prof. Ing. Pietro Lunardi, underlining the importance of setting them correctly within the framework of the ADECO-RS approach.
Preconfinement of a cavity can in fact be achieved using various types of intervention depending on the type of ground (natural consistency), the stress states in play and the presence of water.

GENOA METRO

TUNNEL PORTAL BRIGNOLE SIDE AND FACE OF THE "CORVETTO" STATION TUNNEL

$\varnothing_{LINE} = \sim 9$ M.
$\varnothing_{STATION} = \sim 17$ M.
GROUND: MARLS
OVERBURDEN: ~ 10 M.

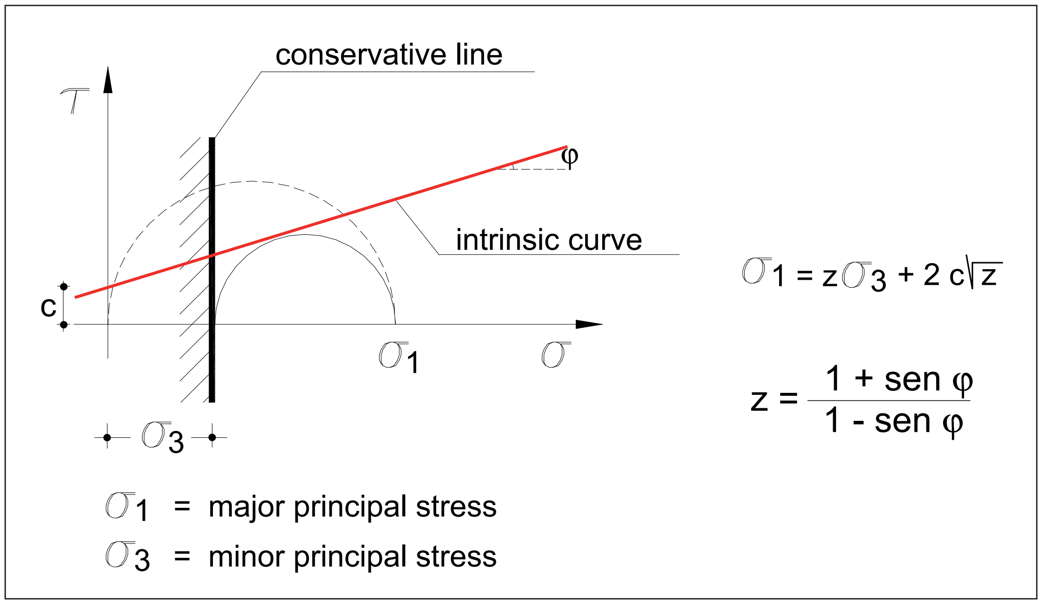
Because of the action that these types of intervention perform (designed to prevent the rock mass from relaxing and to conserve the principal minor stress σ_3 at levels higher than zero), they are termed "conservative", and include the following:

- conservative techniques to protect the core-face, when it channels stresses around the outside of the core ahead of the face, actually performing a protective action which conserves its natural strength and deformability properties (e.g. shells of ground improved by sub-horizontal jet-grouting, shells of fibre reinforced shotcrete or concrete created in advance by mechanical precutting, pretunnelling, or artificial tunnelling techniques and so on);
- conservative techniques to reinforce the face-core, when it acts directly on the consistency of the advance core itself, improving its natural shear strength and deformability by means of appropriate reinforcement techniques (e.g. reinforcement of the core using fibre glass structural elements, horizontal jet-grouting in core-face, etc.).

It is important to consider that this intervention is to be seen as complementary to ordinary conventional confinement of the face and the cavity, insofar as its effectiveness is dependent on the continuity and regularity of the passage from action to preconfine the cavity (ahead of the face) and that to confine the cavity (after the passage of the face). More specifically, it is always accompanied by the casting of the tunnel invert at the face and by concave shaping of the face itself. After describing the characteristics of each technology and how it is performed, a report is given of the main projects on which it has been used, with photographs and diagrams of the design and tunnel section types adopted. A bibliography of references is then given for readers who wish to study the subject in greater detail.

CONSERVATIVE LINE

THE ACTION EXERTED ON THE MEDIUM BY THE "CONSERVATIVE" OPERATIONS IS REPRESENTED ON THE MOHR PLANE BY THE "CONSERVATIVE LINE"



1982 JET-GROUTING

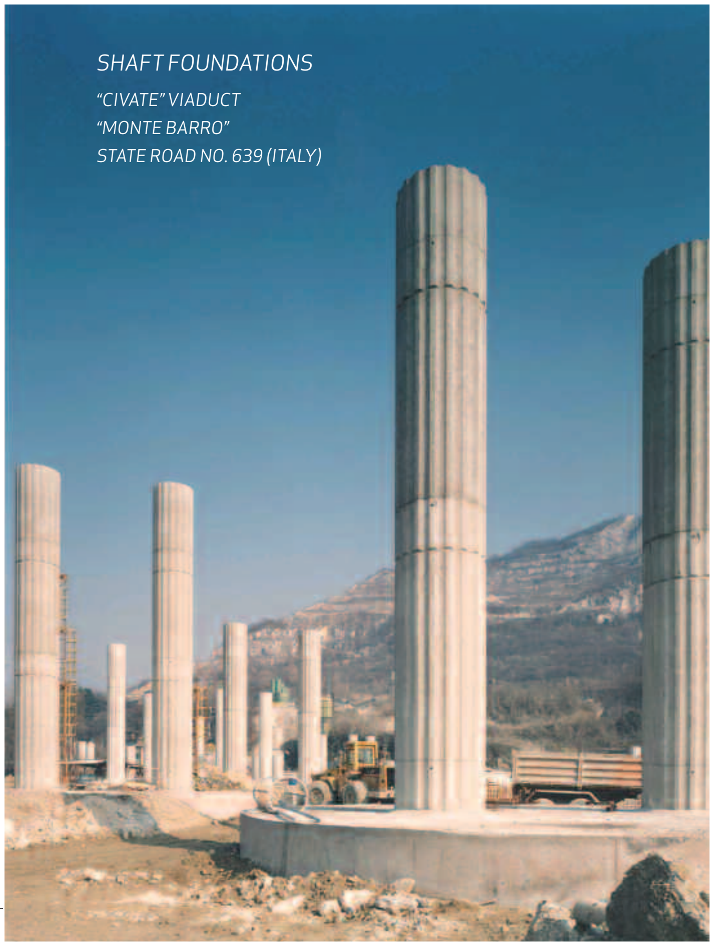
Jet-grouting technology interpreted and employed by Rocksoil

Applied for the first time in civil engineering in Pakistan by Cementation Co in 1950 circa and subsequently developed by the Japanese, jet-grouting started to rapidly enjoy great success in the 1980s when it was used widely in many fields of civil engineering.

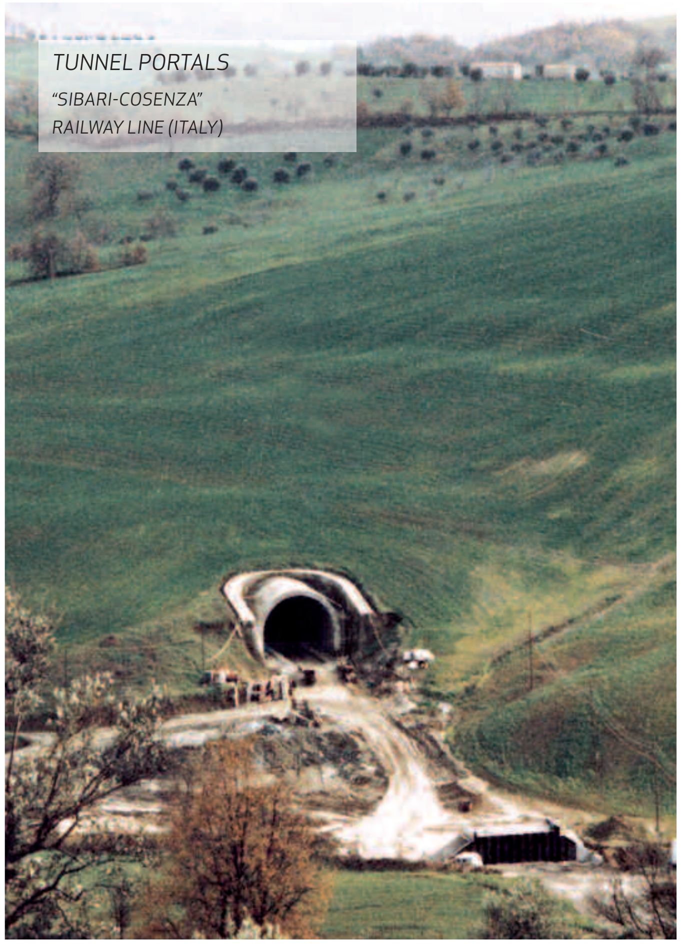
Large part of the merit for this overwhelming success is attributable without doubt to the intense research activity undertaken and developed since 1982 by Prof. Ing. Pietro Lunardi. He had seen the huge potential for the technology and introduced it in Italy through Rocksoil, working intensely to study and develop designs to exploit its particular characteristics, applicable to various types of civil engineering works.

More specifically, work was performed both to develop equipment and injection techniques, by fostering profitable collaboration with small specialist companies, which were then emerging, and also to develop specific designs for foundation works, slope confinement and stabilisation works, underground works, etc.

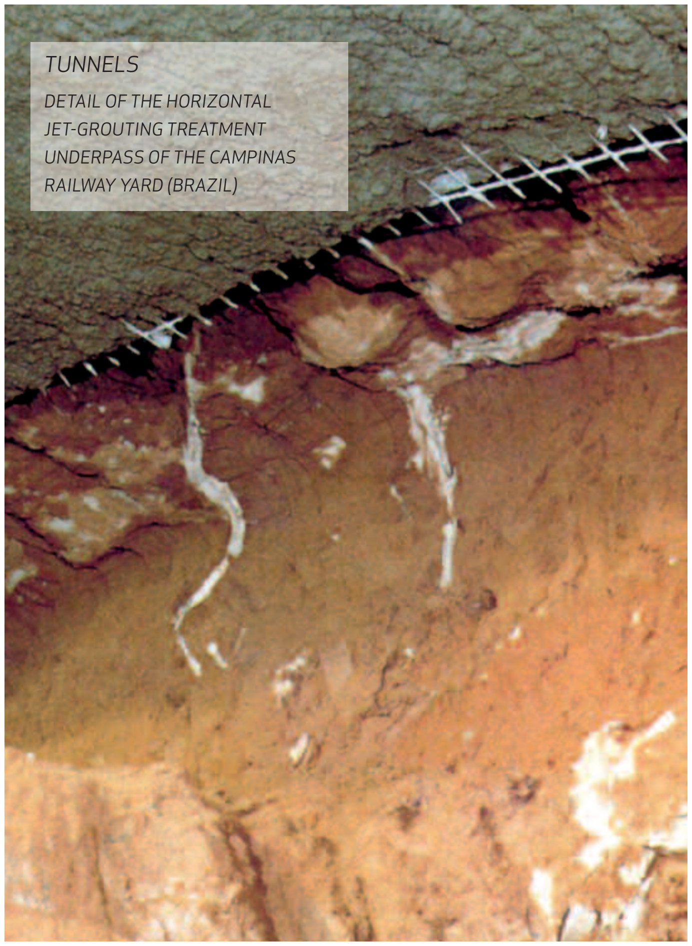
JET-GROUTING APPLICATIONS



SHAFT FOUNDATIONS
"CIVATE" VIADUCT
"MONTE BARRO"
STATE ROAD NO. 639 (ITALY)



TUNNEL PORTALS
"SIBARI-COSENZA"
RAILWAY LINE (ITALY)



TUNNELS
DETAIL OF THE HORIZONTAL
JET-GROUTING TREATMENT
UNDERPASS OF THE CAMPINAS
RAILWAY YARD (BRAZIL)

A particular focus was also placed on the development of calculation methodologies to dimension and monitor ground improvement structures created using the new technology and also on the development of monitoring techniques in order to effectively verify compliance of the actual results with the design assumptions.

Jet-grouting consists of injecting controlled volumes of cement mixture into the ground to be treated at very high pressure (from 300 to 600 bar) through nozzles of appropriate diameter. It can be performed using three different methods, the first two of which perfected in Italy and the third of Japanese origin:

- injection of mixture only (monofluid system);
- injection of air and mixture (2-fluid system);
- injection of air, water and mixture (3-fluid system).

JET-GROUTING FIELD TESTS

THE RESULTS OF GROUND IMPROVEMENT

Ordinary grouting methods are based mainly on the permeation and impregnation of fluids and are therefore limited by the absorption capacity of the ground and are consequently difficult to control in practice. Jet-grouting on the other hand is based mainly on hydrofracturing (claquage) which breaks up the ground by the mechanical action of a jet of fluid under very high pressure and velocity, which mixes, compacts and improves it within a well defined area.

The operations required to perform monofluid and 2-fluid jet-grouting, which are the most widely used systems, consist basically of two stages (Figure 1):

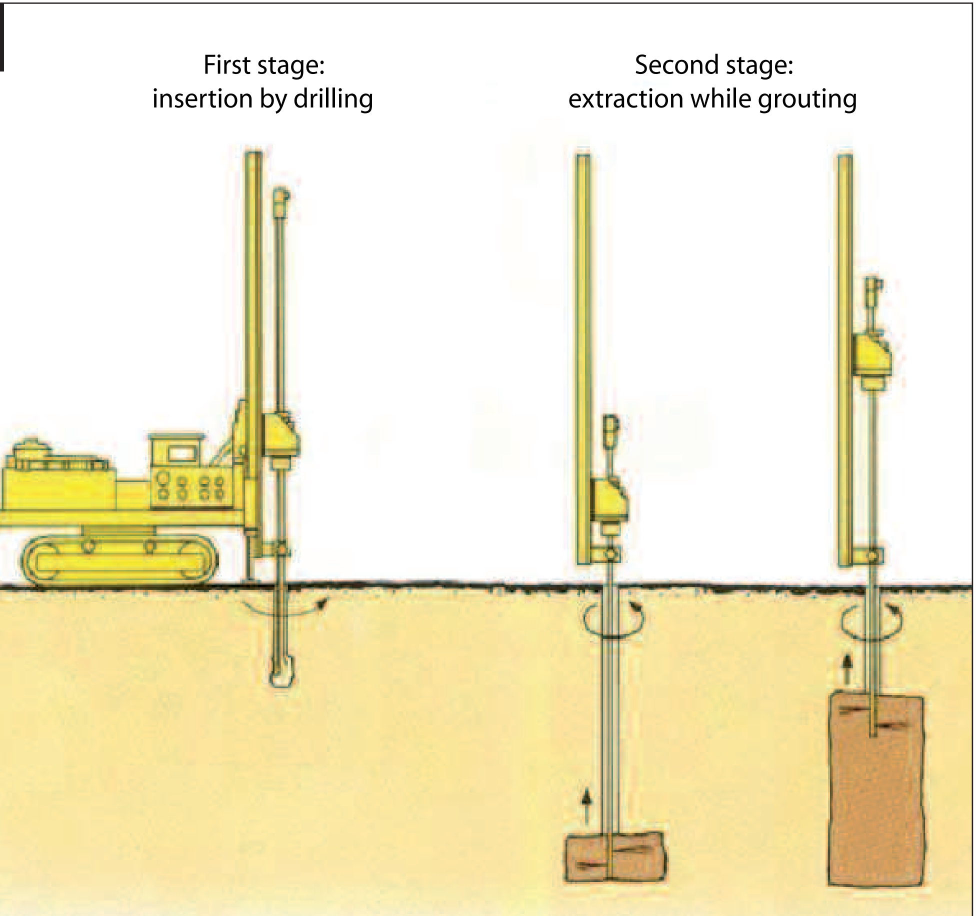
▸ the insertion or drilling stage in which a drill string is inserted into the ground, fitted with a valve and nozzles, to the depth required by the design;

▸ the withdrawal or extraction stage, in which the string of rods is extracted at the programmed withdrawal velocity and angle, while the mixture is injected through the nozzles at the same time.

Volumes of improved ground of the desired shape and dimension can be obtained by varying the withdrawal and rotation velocity of the drill rods and the number and diameter of the nozzles.

The mechanical characteristics of the ground are increased by the treatment to the point where its permeability and strength properties are comparable with those of concrete.

1



1. *THE STAGES OF JET-GROUTING OPERATIONS*

RESEARCH AND EXPERIMENTATION

When Rocksoil introduced it in Italy, the real potential of jet-grouting technology was not clear. The uncertainties regarded above all the following:

▸ the types of ground in which it could really be applied with good results;

▸ the geometrical and geomechanical characteristics of the volumes of ground treated that can be obtained as a function of the different operating parameters;

▸ the reliability of the treatment and the ability to verify its homogeneity.

In addition to this there was the total lack of experience, designs, parameters and calculation models to refer to when designing works to be performed using it.

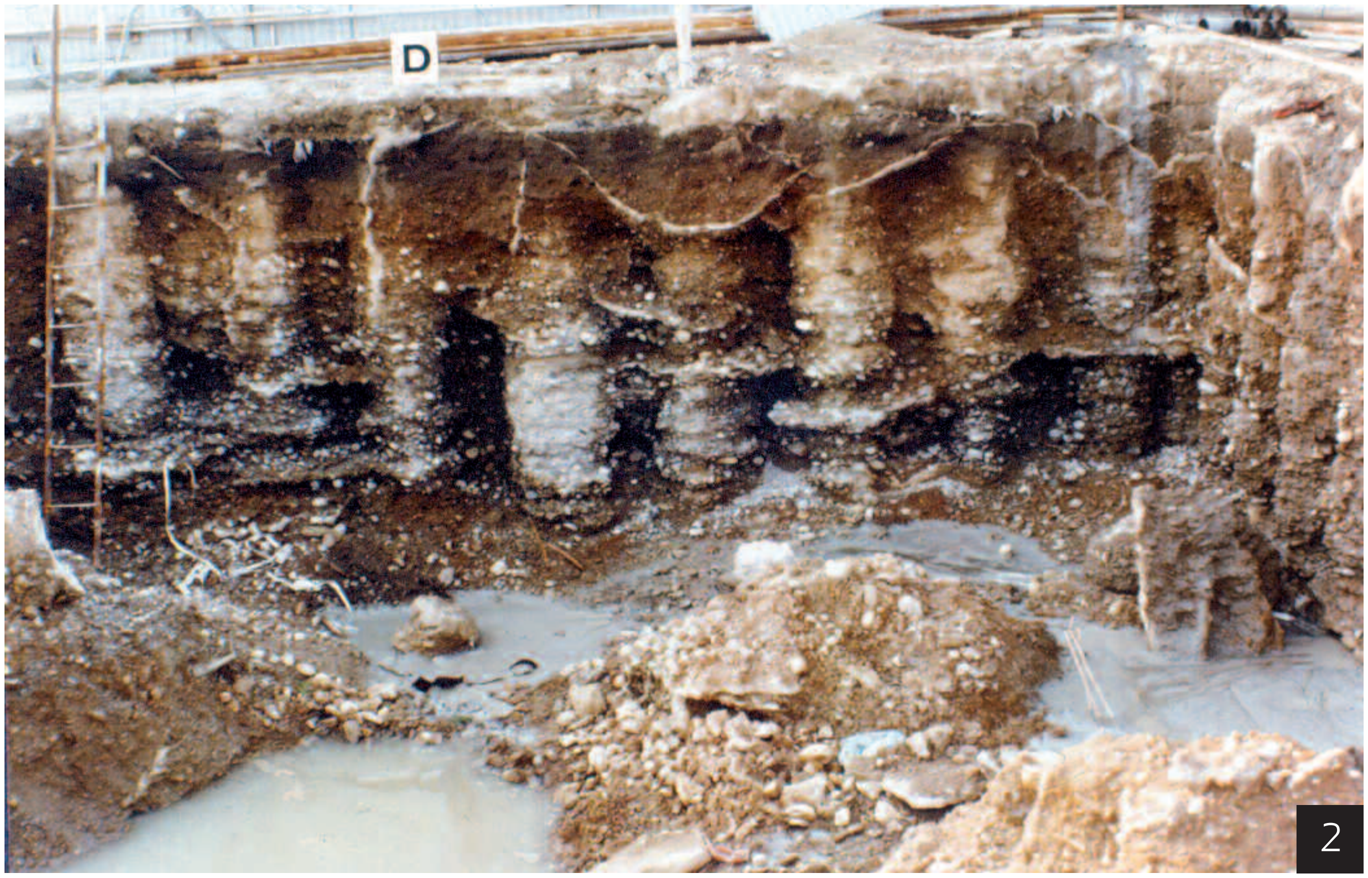
In order to find an answer to the uncertainties and to fill gaps in its knowledge, Rocksoil carried out intensive research activity on a number of levels:

▸ on a more experimental level to identify the preliminary surveys to perform on the ground to assess the feasibility of jet-grouting treatment and inform the choice of mixture and operating parameters to use;

▸ on a more theoretical level, to develop mathematical models to study the development of stresses and deformation within the volumes of treated ground and in the surrounding natural ground;

▸ on a basically practical level to identify the most appropriate monitoring systems to ensure proper execution of the ground improvement treatment;

▸ on a distinctly engineering level to conceive of and develop original designs to draw the maximum advantage from the specific characteristics of the technology.

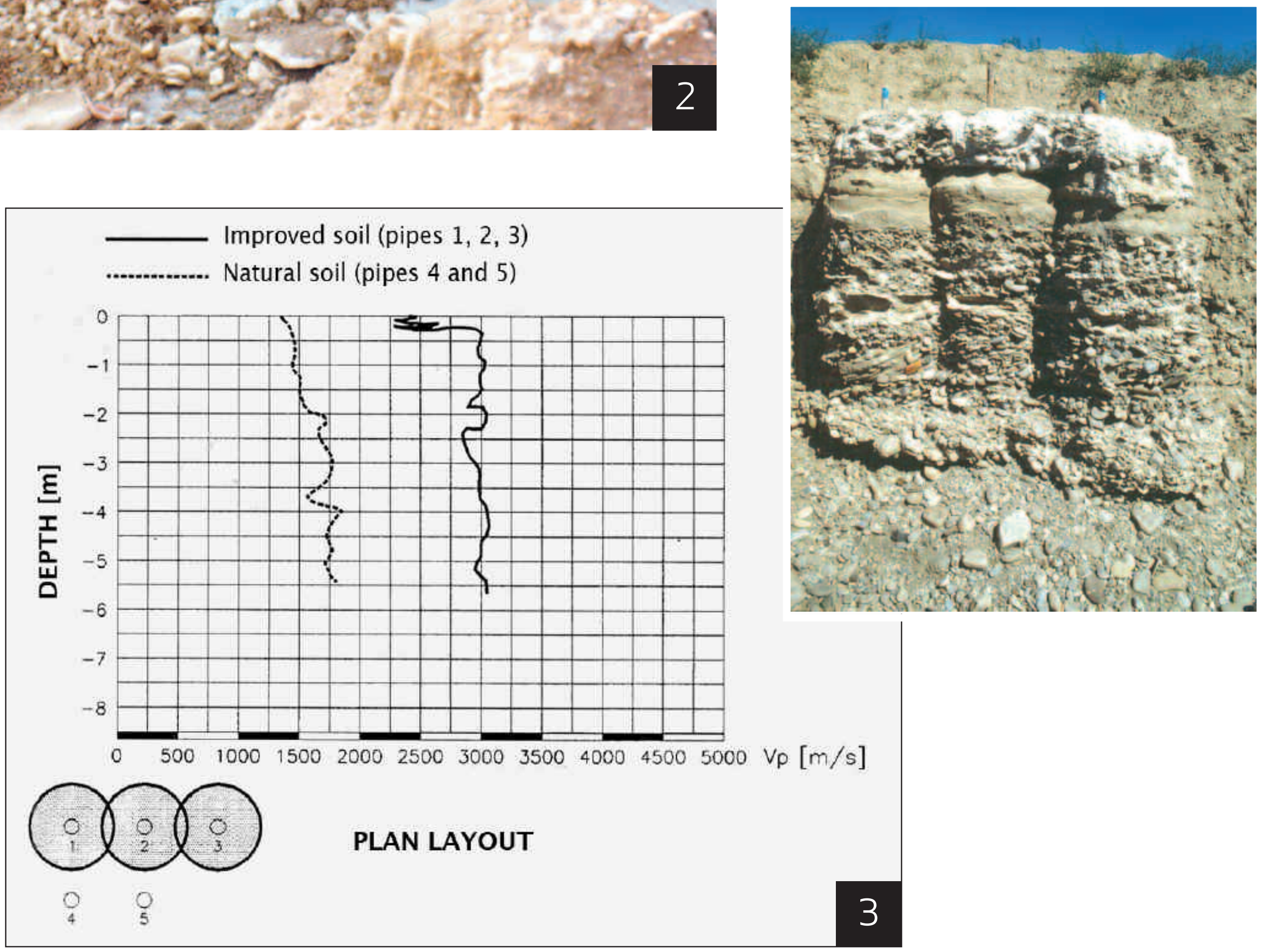


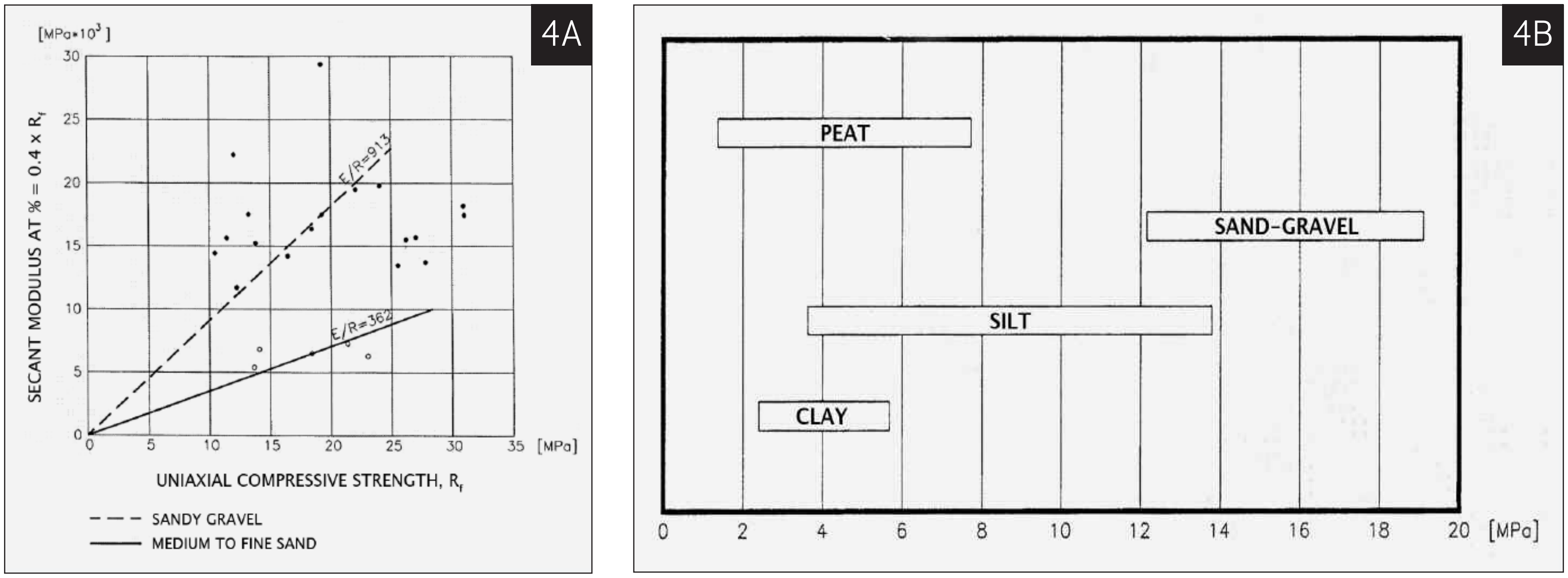
2. *JET-GROUTING FIELD TESTS*
LINE 1 OF THE MILAN METRO
SAN GIOVANNI SHAFT, 1980

3. *CONTROL TESTS USING DOWNHOLE SONIC METHODS*
PONTE TARO (PARMA), 1982

FIELD TESTS

It is important to consider the value of field tests in the field of research and experimentation.
Performed on suitable sites, generally adjacent to those where the actual works were to take place, these have not only been found to be irreplaceable for obtaining practical and immediately useable indications for the purpose of selecting the type of mixture and the operating parameters to adopt, but they have also been an extremely useful tool for acquiring knowledge of the new technology, helping to progressively eliminate uncertainties and gaps which still remained.
They consisted of carrying out a certain number of test treatments, using geometries selected on the basis of those specified for the ground improvement programme to be im-





4A AND 4B. *MONOAXIAL COMPRESSIVE STRENGTH OF JET-GROUTING APPLIED TO DIFFERENT TYPES OF GROUND*

plemented (Figure 2). The operating parameters were varied for each treatment or group of treatments in order to be able to then choose the most appropriate combinations on the basis of the results obtained.

The following was performed on completion of the tests:

- control tests using downhole sonic methods (Figure 3), to assess the mechanical quality of the improved ground at different levels;
- direct examinations of the results of the treatment, after first uncovering and revealing it, to visually check the geometrical dimensions, the structural continuity and possible co-penetration between treatments;
- destructive tests on core samples taken from the treated material, performed in different directions, to assess the geomechanical parameters obtained by means of laboratory tests.

A reliable knowledge of the characteristics of the new technology was obtained from an analysis of the results observed in the very numerous field test performed, both with regard to its use in different types of ground and to the geomechanical characteristics that can be obtained with the treatment (Figure 4).

INNOVATIONS

Considerable advances have been made over the years in the use of jet-grouting technology. These advances have been seen both in the equipment, the execution techniques and studies of the mixtures and also in the development of original designs appropriate for its use.

As concerns the technological progress, research focused mainly on the pumping mechanics, the experimentation of high pressure systems and the study of appropriate nozzles to achieve the following:

- increase the fracturing power of the jet in order to achieve wider radii of action;
- allow the addition of sand aggregates into the binding mixture in order to obtain stronger improved ground structures;
- allow injection to be performed while drilling is in progress;
- reduce or eliminate the effects of reflection which normally occur due to low rod withdrawal or rotation speeds.

In this respect it is possible today to obtain treated volumes in the shape not only of a column but also in the form of true and genuine diaphragms of in situ improved ground with just a single drilling.

It is also worth mentioning the results achieved in

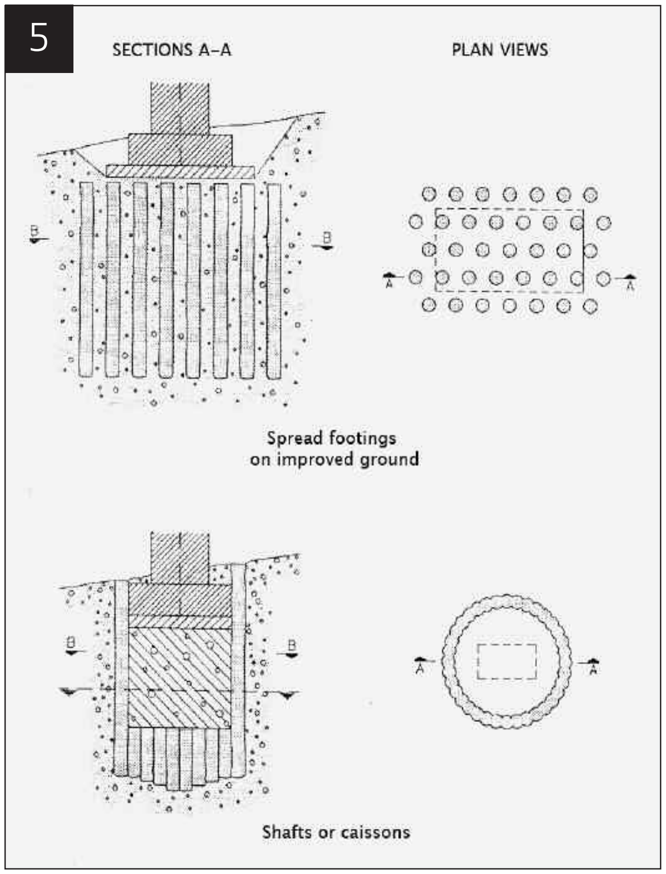
some applications in rock to create impermeable screens in circumstances where conventional grouting with low pressure injections gave poor results due to the tendency of the injected mixture to migrate through discontinuities without filling them sufficiently.
As concerns use in designs for different works, these can be divided, for the sake of clarity, into vertical jet-grouting applications and horizontal jet-grouting applications.

VERTICAL JET-GROUTING

From a historical viewpoint, the first applications of jet-grouting technology, perhaps due to the column shape of the treatment, which is similar to that of a pile, were of the vertical type for use in foundation works, underpinning and restoration.
Rocksoil has made a considerable contribution with regard to foundation works, above all for its design and development of two innovative types of foundation based on jet-grouting technology (Figure 5):

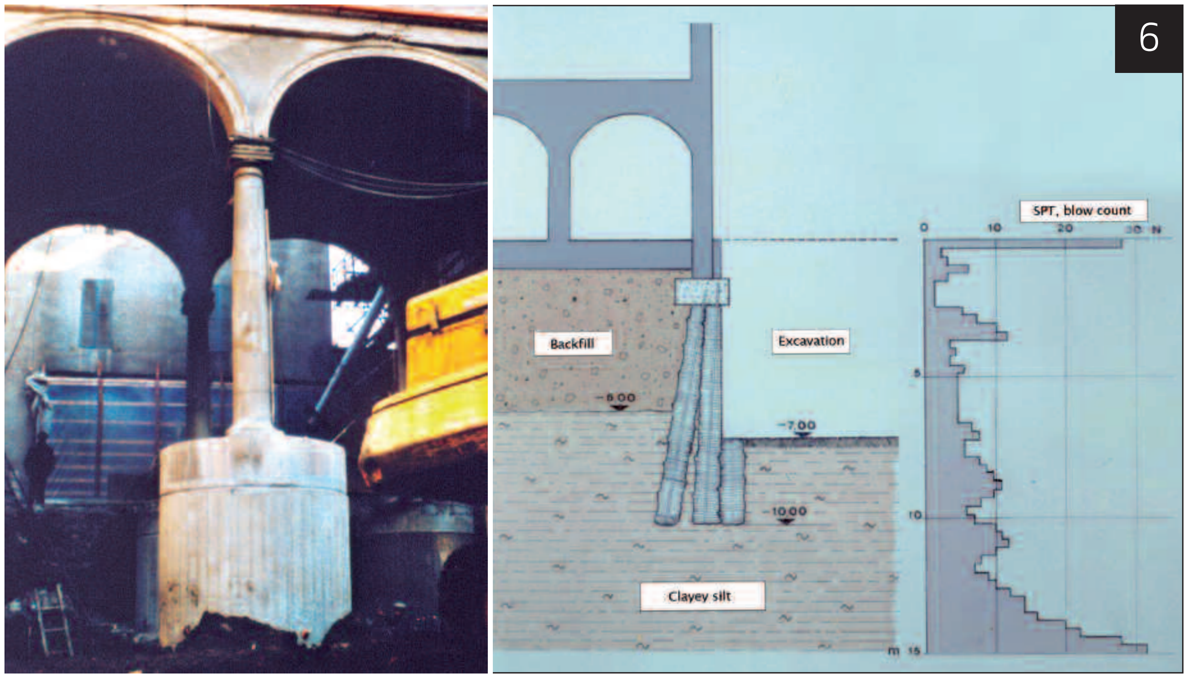
- "spread footings on improved ground", which consist of using jet-grouting to create a series of columns of improved ground of an appropriate diameter and length, properly distributed under the footprint of the future foundation plinth. This type of foundation is particularly suitable for seismic zones because the safe transfer of stresses from the structure to the ground is ensured by the gradual increase in the stiffness of the structures. Foundations of this type were used for the first time in the construction of viaducts at Bardonecchia (Frejus motorway);
- the famous "shaft" foundations, widely employed in constructions to be built on slopes or on river beds, in which jet-grouting technology is usefully employed to create a continuous ring around the perimeter of the future shaft and a bottom plug, consisting of co-penetrating columns of improved ground. The shaft can then be excavated working dry, under the protection of the ring, until the bottom plug is reached, when it is filled with concrete to footing level. The first implementation of a shaft foundation under the water table performed using this system was in the Carnia area in 1983, for the construction of the foundations of a railway bridge in the alluvial river bed of the River Fella (Udine-Tarvisio state railway line). Solutions for underpinning and restoration works designed by Rocksoil using jet-grouting technology set precedents that were followed.

Figure 6 shows the method used in the works for the Banca del Monte in Parma in 1982. In situations like this, where the work had to be done very close to ancient masonry buildings, if properly implemented jet-grouting, which allows the radius of action of the jet to be controlled by adjusting the operating parameters, can be used to inject and improve the ground without causing dangerous heaving of the existing foundations.
With regard to restoration, the work to restore the foundations of a bridge in the bed of the River Taro was impressive. It had been damaged by exceptional flooding which had caused the partial collapse of the bridge in November 1982 (Figure 7). The ground



__5.__ "SPREAD FOOTINGS ON IMPROVED GROUND" AND "SHAFT" VERTICAL JET-GROUTING FOUNDATION

6



6. UNDERPINNING USING JET-GROUTING

THE DEL MONTE DI PARMA BANK BUILDING

7. RESTORATION WORK USING JET-GROUTING

RAILWAY BRIDGE FOUNDATIONS ON THE RIVER TARO AT PARMA

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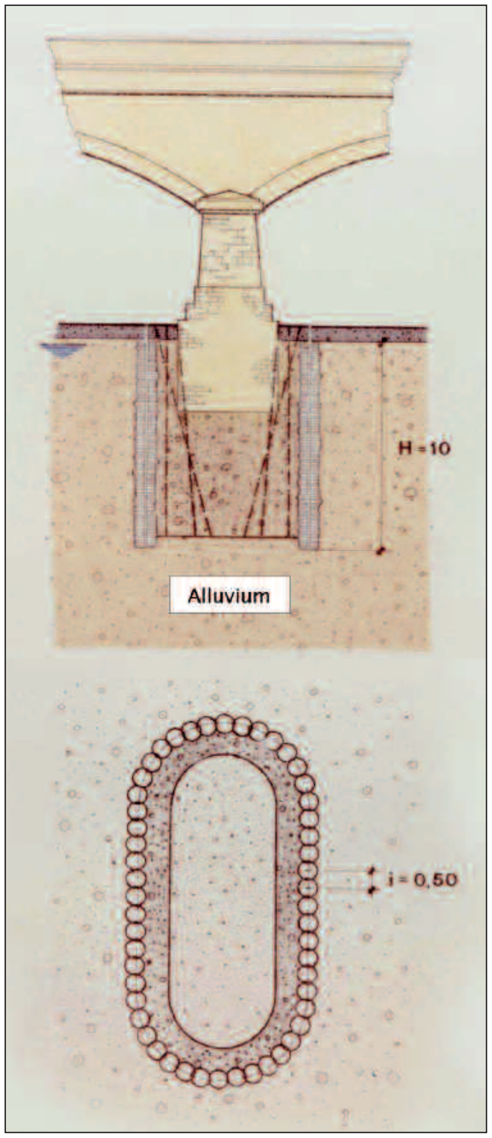


improvement work using jet-grouting designed by Rocksoil S.p.A. was in two parts as follows:

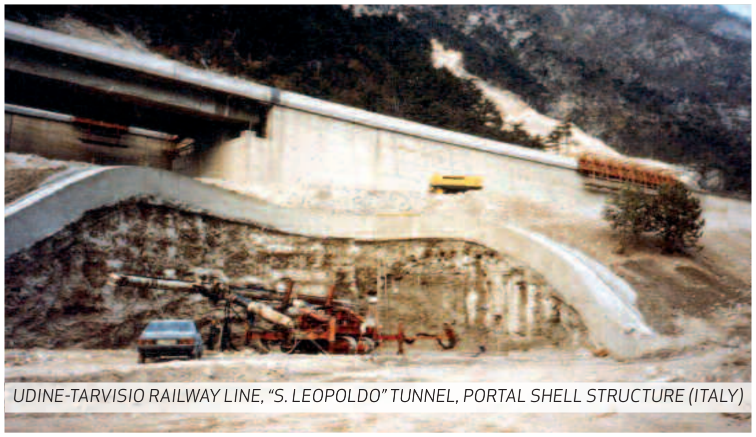
1. the creation of a continuous protective ring of jet-grouted columns around the foundations of each pier, with a thickness and depth sufficient both to ensure effective action to contrast the undermining action of the water flowing in the river bed and near the banks, and also to exert significant confinement action to contain lateral decompression of the ground under the foundation, which was essential if the load bearing capacity of the ground was to be conserved;

2. the ground within the protective ring was treated by means of ordinary grouting techniques to reduce its permeability and increase its bearing strength.

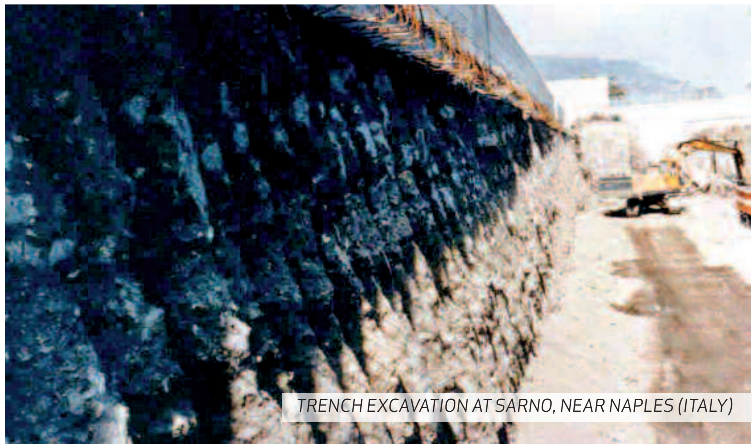
Open and trench confinement and excavation stabilisation works constitute a field in which Rocksoil's research has produced considerable progress in terms of design (Figure 8).

8. VERTICAL JET-GROUTING
DIFFERENT TYPES OF SUPPORT WORKS

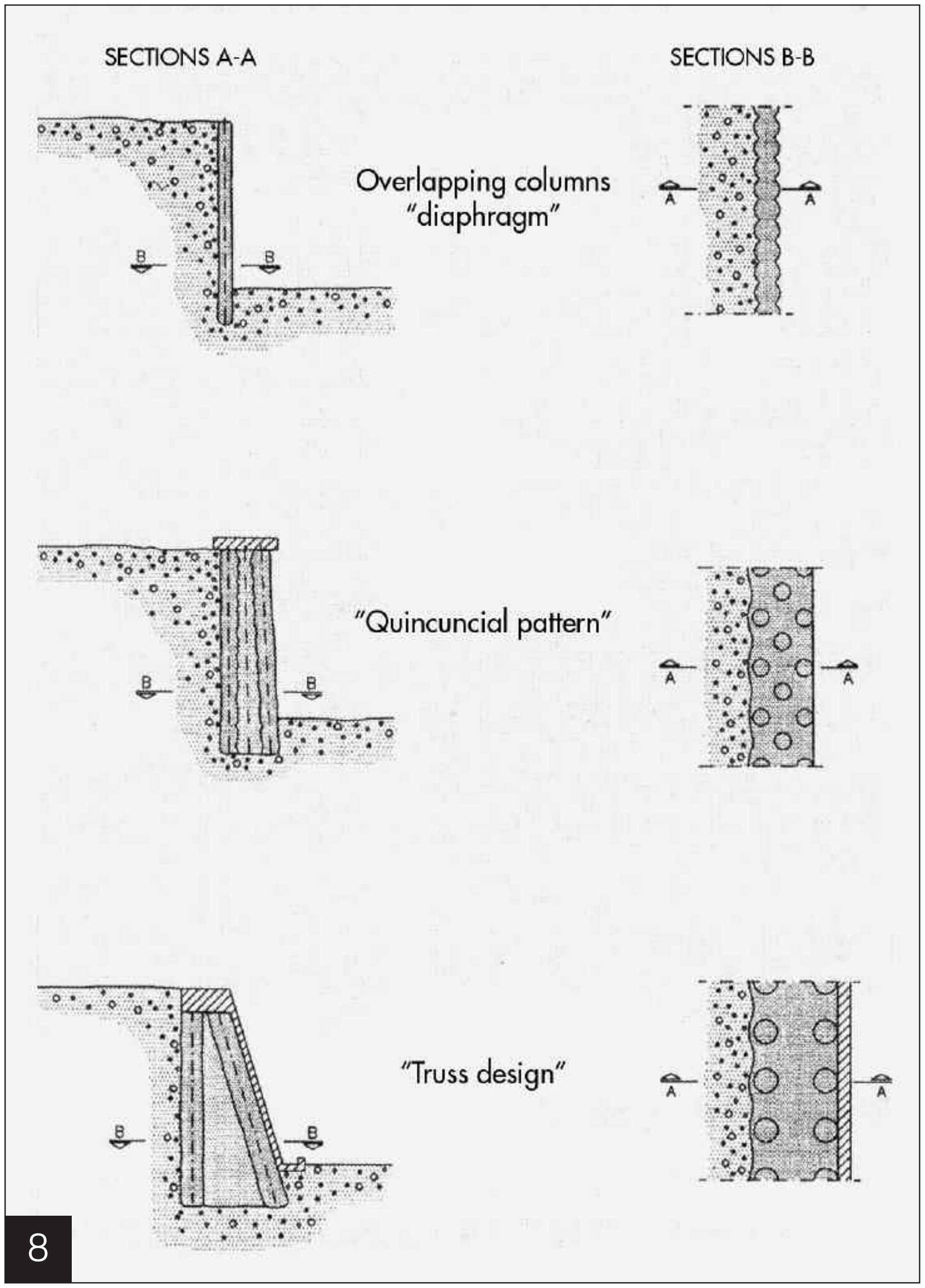
LYONS METRO, LINE D, "SANS SOUCI" STATION (FRANCE)



UDINE-TARVISIO RAILWAY LINE, "S. LEOPOLDO" TUNNEL, PORTAL SHELL STRUCTURE (ITALY)



TRENCH EXCAVATION AT SARNO, NEAR NAPLES (ITALY)



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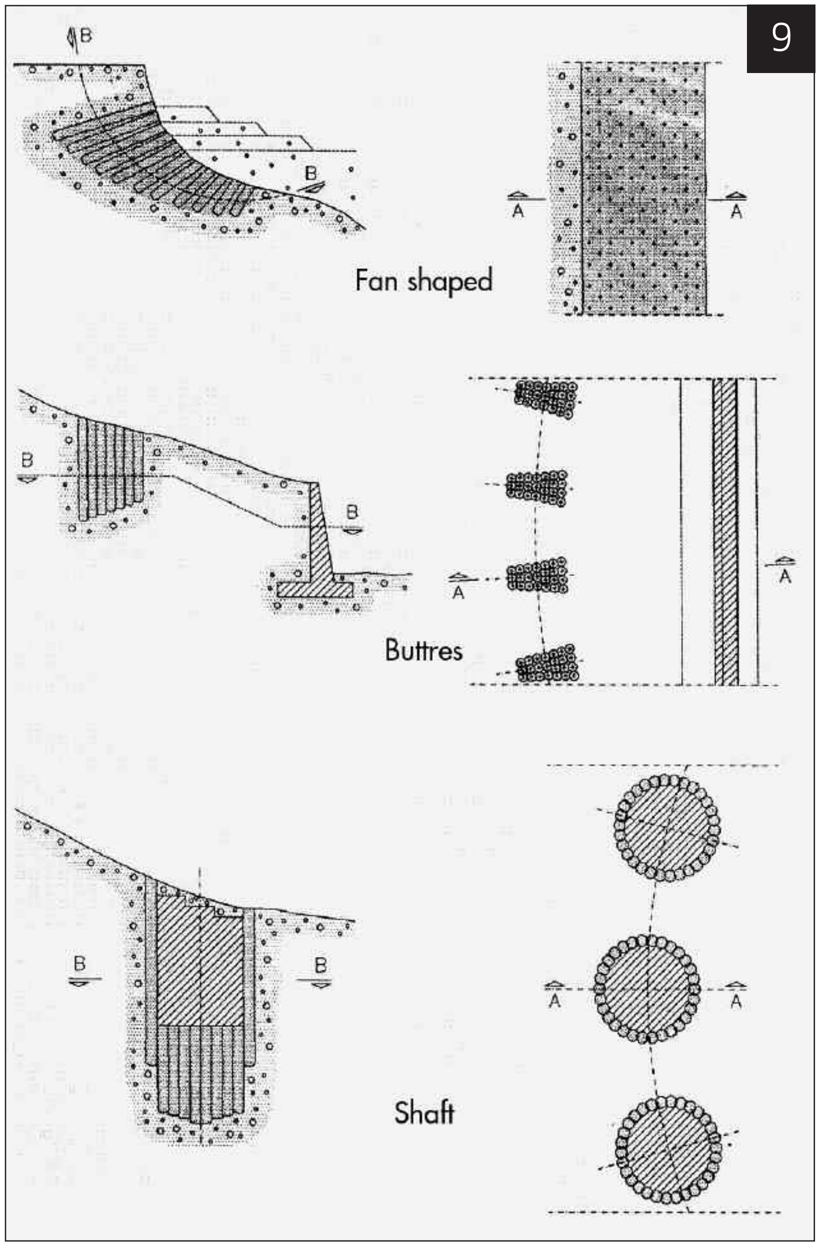


9. VERTICAL JET-GROUTING
DIFFERENT TYPES OF SLOPE STABILISATION WORKS

The first geometrical designs basically consisted of a row of columns of improved ground set more or less side-by-side, positioned around the edge of a future excavation. One such design was proposed by Rocksoil and then used, for example, in Milan in 1980 for the construction of a shaft at Sesto San Giovanni for the station of the same name on Line 1 of the Milan metro (first ever application of the technique in Italy) and then in Lyon in France to protect open excavations for the Sans Souci Station on line D of the Lyon metro (in the photo on page 191).

While effective for temporary confinement of excavations in alluvial or cohesive soils, with fair geomechanical characteristics, geometrical designs of this type did not seem sufficiently reliable to provide permanent protection. For these purposes more complex designs were developed, which involved several rows of columns arranged in a quincuncial pattern.

Thanks to the excellent results obtained and the ease and speed with which it could be implemented, this new solution came rapidly into use, replacing more conventional technologies. Evidence of this can be seen in the success that it enjoyed, when combined with sub horizontal jet-grouting, in the construction of tunnel portals in non cohesive or semi cohesive soils where, because of the shallow overburdens required to commence bored tunnels, it is possible to minimise the volumes of earth excavated, thereby reducing the risk of destabilising the slope and obtaining substantial benefits from a landscape and environmental viewpoint.

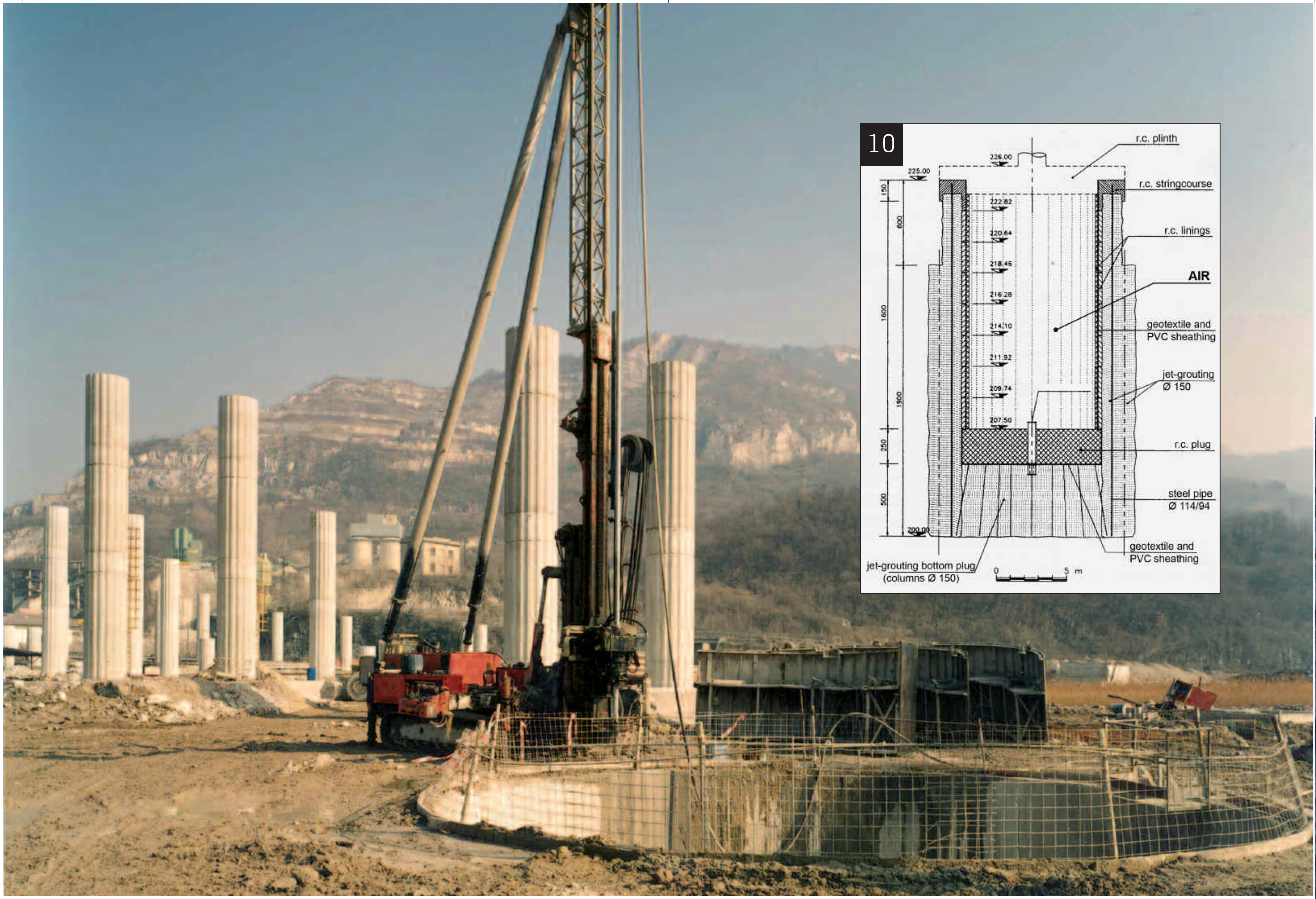
The first application in the world for the construction of tunnel portals occurred in 1985 in Carnia for the construction of the T1 portal on the Pontebba side of the S. Leopoldo Tunnel on the Pontebba-Tarvisio railway line (in the photo on page 191).

Finally, Figure 8 shows the "truss" design that was adopted in Naples in 1986 along the Sarno railway embankment.

It had the advantage of making the ground trapped between the two rows of jet-grouted columns contribute to the stability of the overall structure. In this manner, in addition to being improved by means of claquage, the ground remained permanently under triaxial stress conditions.

The legacy left by Rocksoil with the experience it had acquired in the field of slope stabilisation contains a wealth of designs implemented using jet-grouting technology and Figure 9 summarises some of the main types. They range from "fan" designs to "buttres" confinement of improved ground arranged in a radial direction along the arc of a circumference in a plan view and finally to the "shaft" design, which is again very effective.

SIBARI TUNNEL PORTAL
SIBARI-COSENZA RAILWAY LINE (ITALY)



10. *SHAFT FOUNDATIONS*

CIVATE VIADUCT
THE "MONTE BARRO"
STATE ROADS 639 AND 36

The first was applied for the first time by Rocksoil S.p.A. in 1982 at Gela, in combination with sub-horizontal drains. The land slide had in fact been triggered by erosion produced at the foot of a natural slope by water percolation at the contact with the impermeable substrate.

While drains eliminated the cause of the problem, the use of jet-grouting to improve the ground achieved the objective of guaranteeing an adequate coefficient of safety to the stability of the slope, in consideration of the buildings sited on the summit.

The "buttres" type was used for the first time by Rocksoil S.p.A. in 1984 in Val Topina, to confine the landslide of a layer of silty debris which had been generated by works to construct a lay-by on the "Flaminia" state road No. 3.

The use of jet-grouting technology enabled the following to be achieved:

- avoid the removal of material and thereby cause further relaxation of the slope;
- avoid any vibrations during the works, which might have triggered a new landslide;
- not overloading the slope with heavy equipment;

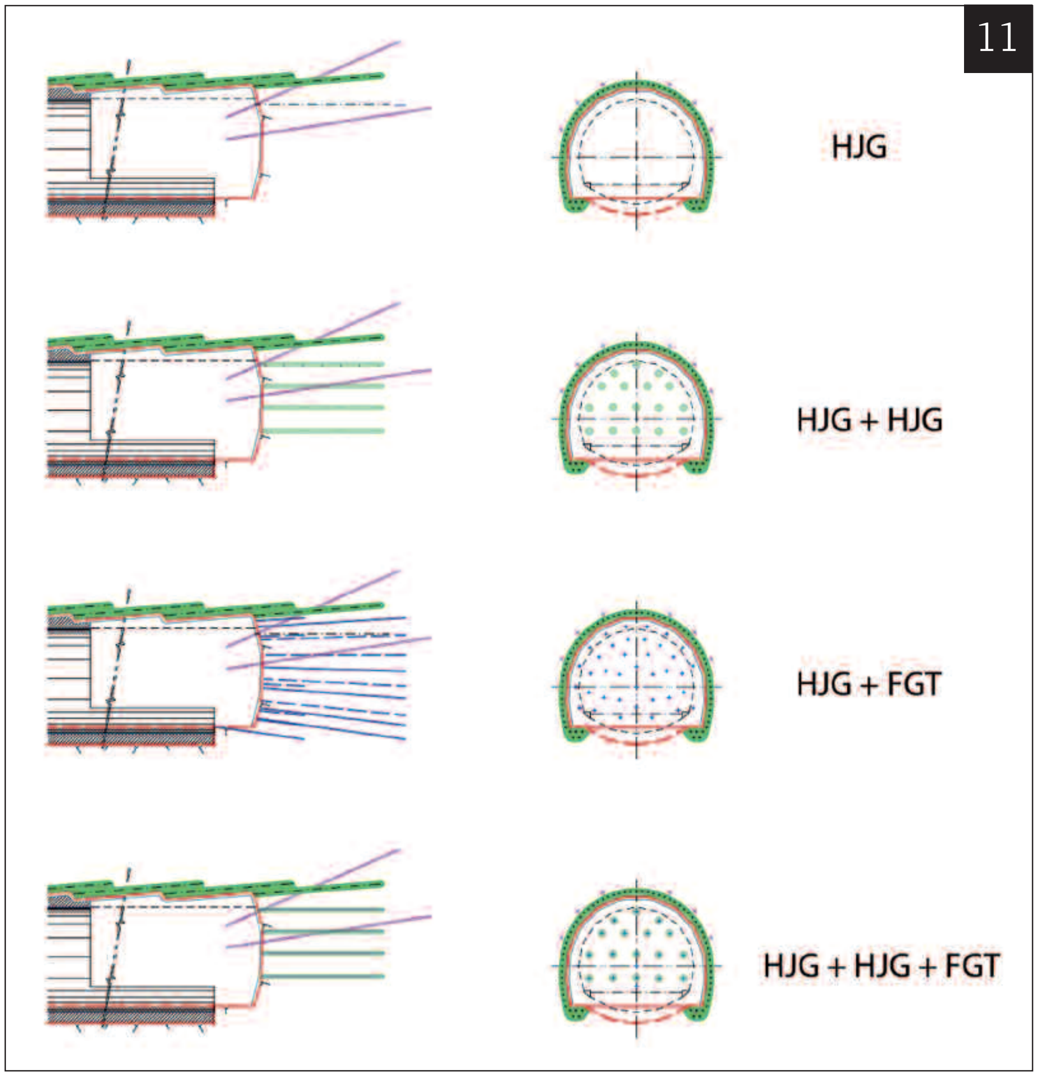
▸ full and complete claquage of the ground in situ.
Finally the "shaft" design was employed in numerous works, including those for the empty floating shaft foundations (see the diagram in Figure 10) constructed for some of the viaducts on the No. 639 and No. 36 state roads on the "Monte Barro" near Civate, where the very poor geotechnical quality of the soils (highly compressible peats) and a water table close to the surface meant that conventional solutions were not feasible.

HORIZONTAL JET-GROUTING

Horizontal jet-grouting is one of the most successful and innovative technologies conceived of by Prof. Ing. Pietro Lunardi and developed by Rocksoil. It made it possible to solve all the difficulties connected with tunnel advance in non cohesive soils at one stroke.
The famous "truncated cone umbrella" treatment launched in advance, with which an "arch effect" is triggered in the ground ahead of the face, was care-

GROUND IMPROVEMENT IN THE FACE AND AROUND THE TUNNEL

VASTO TUNNEL
ANCORA-BARI STATE RAILWAY LINE
∅ = 12 M.
GROUND: SILTY SANDS AND SANDY SILTS
OVERBURDEN: ~ 50 M.



11. HORIZONTAL JET-GROUTING THE PRINCIPAL TYPES

fully studied on the basis of the characteristics of the ground treated using the new technology, in order to be able to make the improved ground work mainly in terms of compressive and shear strength and to ensure that stresses are efficiently transmitted from the arch of improved ground to the natural ground. The truncated-cone "umbrellas" or canopies formed by setting, side-by-side, partially overlapping sub-horizontal columns of improved ground are constructed around the future tunnel in such a way as to provide a strong pre-vault able to exert effective preconfinement action of the ground around the future cavity.

The deviation of the stresses produced by the presence of a strong artificial arch ahead of the face (shell of improved ground), makes it possible to take the load off the face-core and protect it, preventing or at least limiting relaxation of the ground following excavation, thereby countering the reduction to zero of the principal minor stress σ_3. Within the framework of the ADECO-RS approach, sub horizontal jet-grouting technology forms part of cavity preconfinement operations. It can be performed in combination with other ground improvement work ahead of the face and classified into four types, to be selected on the basis of the type of ground and the stress-strain conditions to be tackled (Figure 11):

1. (HJG): simple protection of the core-face through the construction of pre-vaults of improved ground around it using horizontal jet-grouting (a direct conservative technique);

2. (HJG+HJG): reinforcement of the core-face by the creation of treated columns of ground using horizontal jet-grouting and protection of the core at the same time by means of pre-vaults of improved ground created again using horizontal jet-grouting (a mixed conservative technique);

TABLE 1. A SPECIFIC PROJECT IN WHICH THE TYPE WAS USED EXPERIMENTALLY CAN BE ASSOCIATED WITH EACH OF THE MAIN TYPES

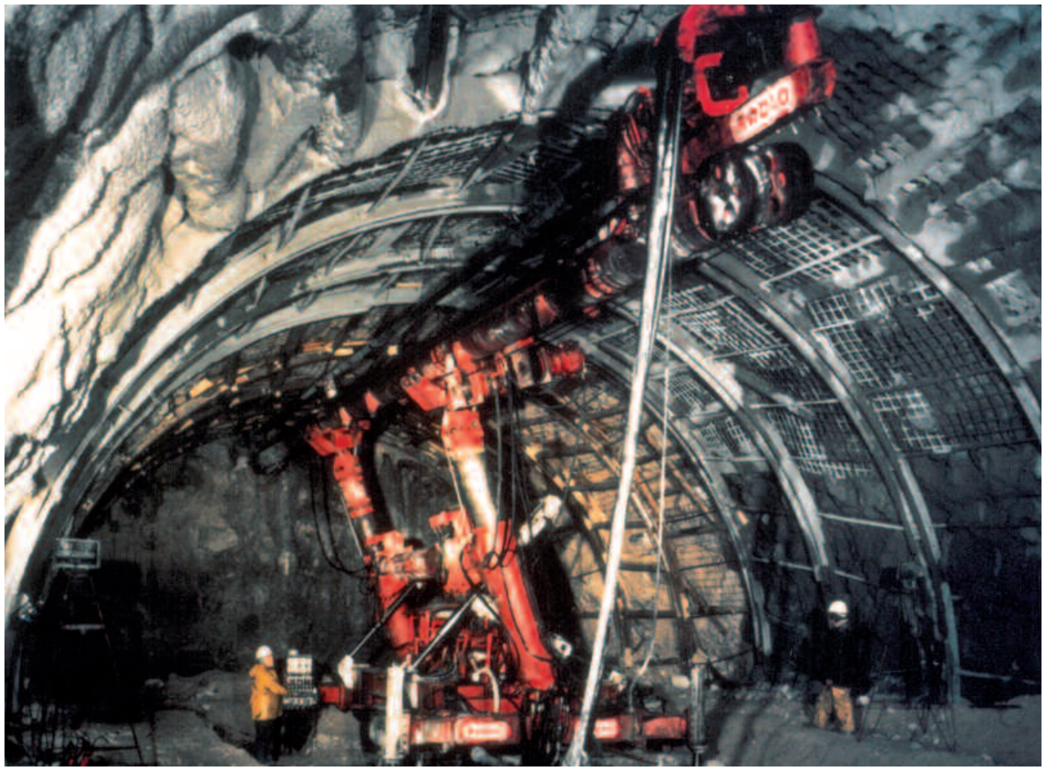
Relative project		Tunnel length [m]	Type of ground	Max overburden [m]	Diameter [m]	Type of ground improvement work
Work	Tunnel					
Carnia-Tarvisio railway line [1983]	Campiolo	1840	Rubble-slope	350	12	HJG
Cristoforo Colombo road underpass in Rome [1989]	Capitan Bavastro	100	Backfill on clayey silty deposit	4	12.50	HJG+HJG
Ancona-Bari railway line [1993]	Vasto	6800	Silty clays	135	12	HJG+FGT
Bologna-Firenze high speed railway line [1996]	Firenzuola	14340	Silty sands with gravelly intercalations	560	13.5	HJG+HJG+FGT

3. (HJG+FGT): fibre glass reinforcement of the core-face and protection of the core at the same time by means of pre-vaults of improved ground created using horizontal jet-grouting (mixed conservative works);
4. (HJG+HJG+FGT): reinforcement of the core-face by the creation of a series of horizontal micro-columns of improved ground using micro-jet-grouting reinforced with the insertion of fibre glass tubes and protection of the core at the same time by means of pre-vaults of improved ground created using horizontal jet-grouting (a mixed conservative technique).
Each of these types can be associated with the relative project (see Table 1) in which it was used experimentally for the first time.

THE FIRST APPLICATION AT CAMPIOLO (1983)

The construction of the Campiolo Tunnel on the twin track Udine-Tarvisio railway line required an initial section of 170 m. of tunnel to be driven from the Udine portal through rubble slopes, with boulders, some of large dimension, dispersed in the sandy silty matrix, under an overburden varying from 0 m. to 70 m.
Until then, tunnel advance through this type of material had always presented considerable difficulty with regard to the instability of the face and was generally tackled by partial advance on several headings or using a core (nose) at the face seeking to "support" the ground around the profile of the excavation with spiles and ribs.
While it did not at all exclude the fall-in of material, this manner of proceeding was not even able to guarantee the adequate safety of site personnel. On the other hand, the alternative was to improve the ground ahead using conventional grout injections, but this was often costly financially and the results remained uncertain since it was difficult

12. THE USE OF HORIZONTAL JET-GROUTING FOR GROUND IMPROVEMENT
"CAMPIOLO" TUNNEL
THE UDINE-TARVISIO STATE RAILWAY LINE
∅ = 12.20 M.
GROUND: RUBBLE SLOPES
MAX OVERBURDEN: 70 M.



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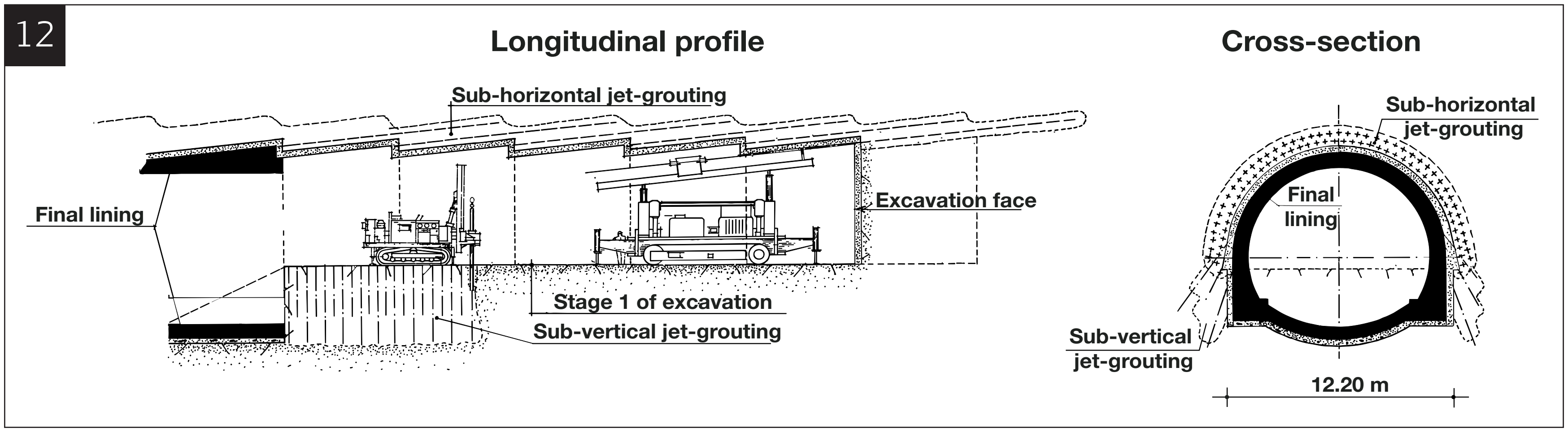


DETAIL OF THE GROUND AROUND THE CAVITY IMPROVED BY MEANS OF SUB-HORIZONTAL JET-GROUTING

"CAMPIOLO" TUNNEL, UDINE-TARVISIO STATE RAILWAY LINE

∅ = 12.20 M.
GROUND: RUBBLE-SLOPE
MAX OVERBURDEN: 70 M.

to control how the mixture would really migrate through the ground.
It was in this context that in 1983, the idea of experimenting with a new tunnel advance technology was proposed. It had been studied as part of research then in progress to develop "conservative" action, capable that is of exerting controlled cavity preconfinement action, by acting appropriately on the rigidity of the advance core. The technology known today as horizontal jet-grouting was therefore tried for the first time in the world at Campiolo. It is used to create a shell of improved ground ahead of the face around the profile of the future excavation, which is strong enough to guarantee the necessary protection for the ground which constitutes the advance core and therefore the formation of a transverse arch effect very close to the profile of the excavation.

The geometry that was used for the treatment, which on that particular historic occasion was performed half-face, since it was considered too audacious to apply the new technology immediately with full-face advance, is shown in Figure 12.
The results achieved were reassuring from both a technical and an economical viewpoint. The deformation measured was practically zero in terms of both extrusion and convergence and furthermore, not only did the new technology guarantee extremely fast advance rates for the geotechnical conditions (an average of 1.7 m/day of finished tunnel), but it also enabled excellent programming of the works and great operational safety.
The choice of jet-grouting technology to improve the ground was so successful that it also solved problems of pollution and permeation, dependent on the granulometry of the material treated, typically encountered with conventional grouting methods. At a distance of over twenty years, the technology has demonstrated that it is also very long lasting.

SPREAD AND EVOLUTION OF THE TECHNOLOGY

The success achieved at Campiolo determined a true and genuine revolution in the tunnelling world, which for the first time could count on technology able to work effectively in soils with no cohesion. After Campiolo no tunnel would be driven through loose soils except by using horizontal jet-grouting.
The experience acquired from a very large number of applications implemented since then led to continuous evolution of the technology, the stages of which can be summarised by referring to the relative projects (Table 1):

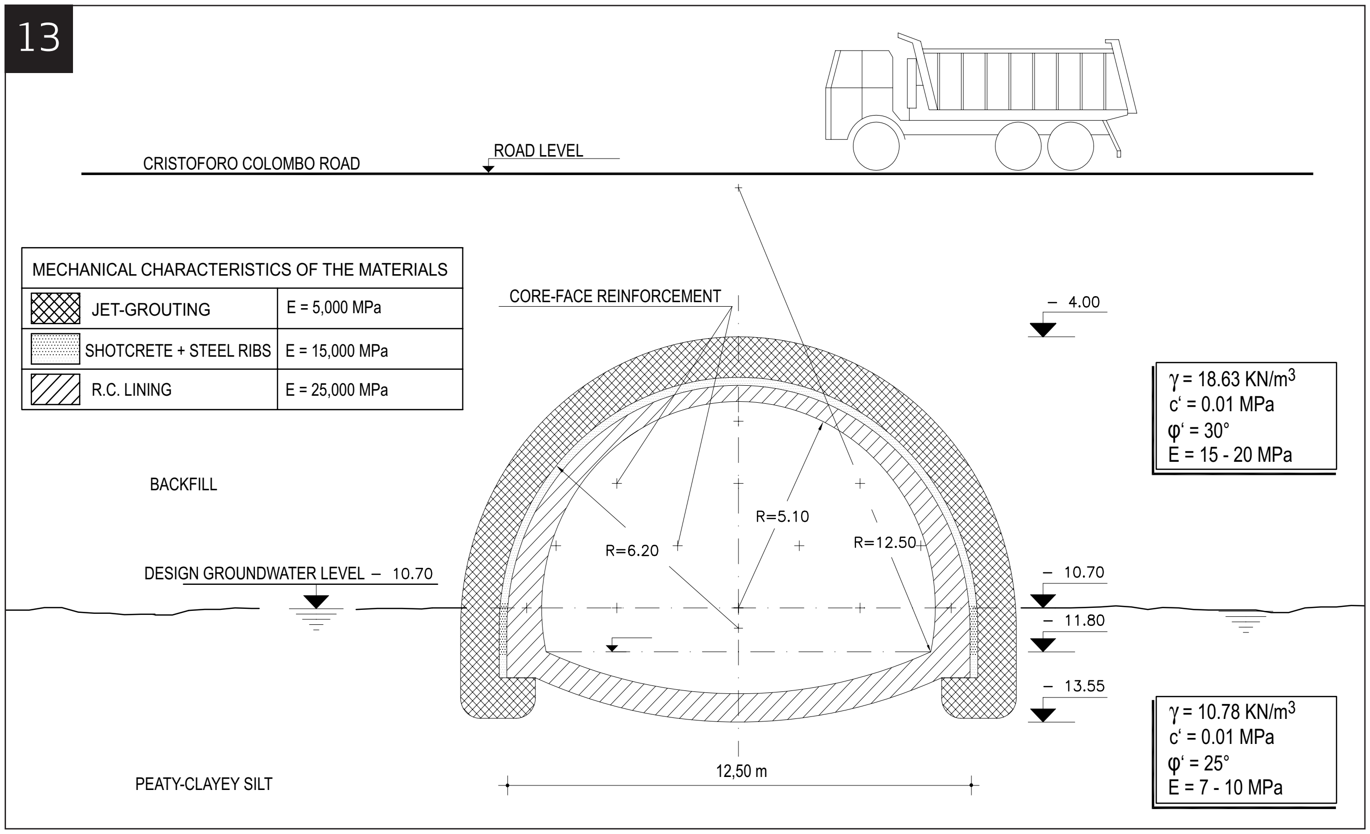
- the "Capitan Bavastro" Tunnel (100 m. in length and 12.5 m. in diameter of excavation) for the via Cristoforo Colombo underpass in Rome. This project introduced the application of full face technology for the first time using a JGO+JGO design and the geometry shown in Figure 13. The problems were typical of those encountered in driving an underground tunnel through backfill material, partially under the water table, with cover of around 4 m. and under the action of dangerous dynamic loads exerted by vehicle flow above. The solution proposed by Rocksoil enabled the problems of the statics and stability to be solved effectively. It was implemented extremely rapidly and with costs decidedly lower than those forecast in the original design (cut and cover between diaphragm walls, with surface vehicle traffic interrupted), avoiding not only all the traffic problems, but also those of moving underground services which had rendered it unfeasible;

- the "Vasto" tunnel for the Ancona-Bari railway line, in silty clays, for which advance using conventional technology (headings and preliminary stabilisation using steel ribs, radial roof bolts and shotcrete) had given rise to uncontrollable extrusion of the core-face.

Tunnel advance resumed on the basis of a completely new design by Rocksoil S.p.A. formulated according to the principles of the ADECO-RS approach. It employed a JGO+VTR design for the first time (horizontal jet-grouting ahead of the face around the cavity + fibre glass reinforcement cemented into the core-face, see Figure 14), which enabled the tunnel to be completed at an average advance rate of approximately 50 m. per month. Table 1 gives the parameters for the ground improvement works;

- the Firenzuola Tunnel on the new high speed/high capacity Milan-Rome-Naples railway line in the section between Bologna and Florence, approximately 13.50 m. in diameter of excavation, driven under an overburden of over 500 m. in grounds consisting of silty sands with gravelly interbedding (see Figure 15), where the JGO+JGO+VTR design was used for the first time.

A list is given below of all the most important projects implemented using Rocksoil designs, which involved the use of jet-grouting technology.



13. *VIA CRISTOFORO COLOMBO UNDERPASS*

THE "CAPITAN BAVASTRO" TUNNEL ROME

∅ = 12.50 M.
GROUND: BACKFILL
OVERBURDEN: 4 M

14. *HJG+FGT GROUND IMPROVEMENT WITH 55 FIBRE-GLASS TUBE REINFORCEMENTS 18 M. IN LENGTH*

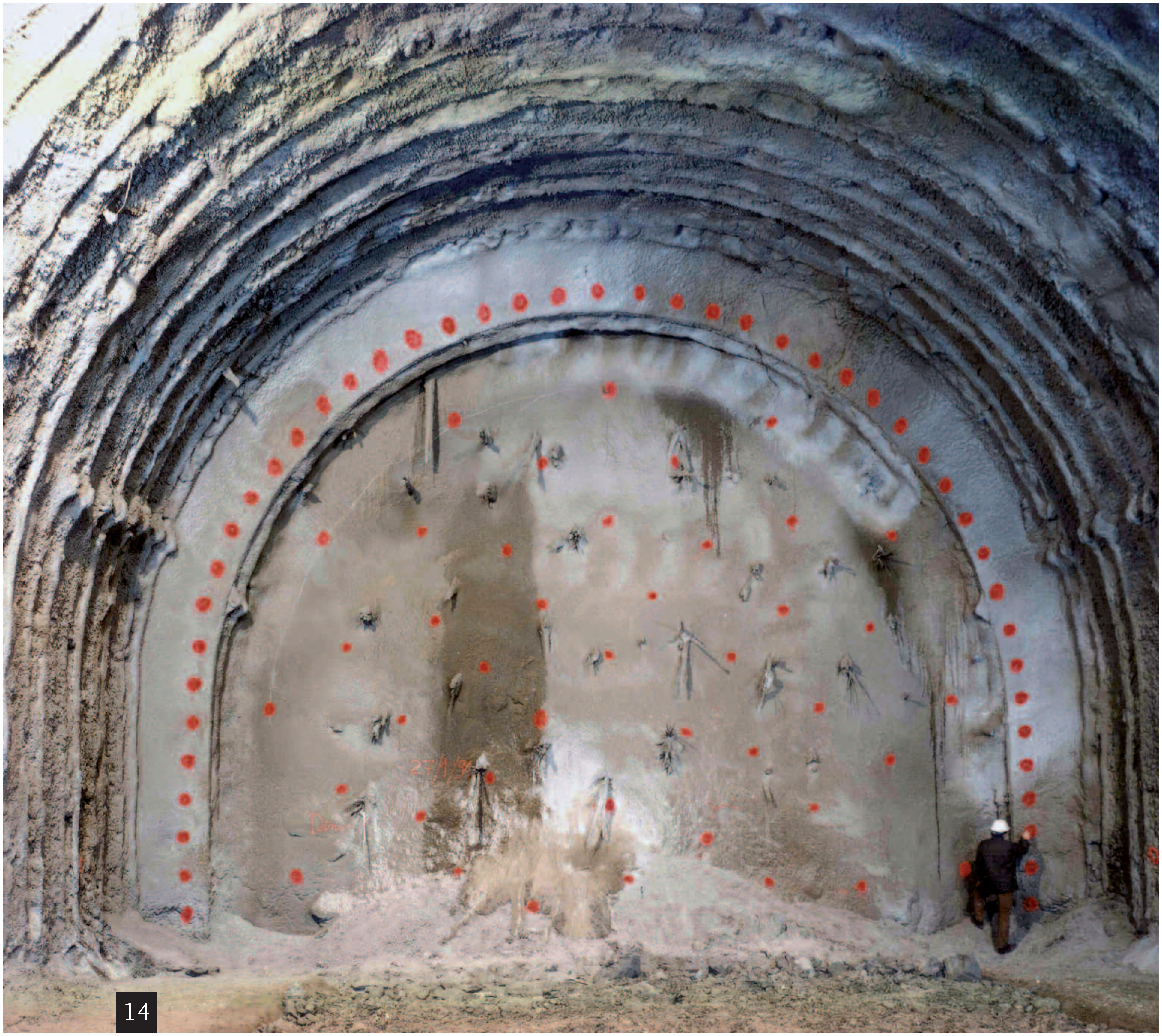
"VASTO" TUNNEL
ANCONA-BARI STATE RAILWAY LINE

∅ = 12 M.
GROUND: CLAYEY SILTS AND SILTY CLAYS
MAX. OVERBURDEN: ~ 110 M.

15. *HJG+HJG+FGT GROUND IMPROVEMENT*

"FIRENZUOLA" TUNNEL
HIGH SPEED/CAPACITY
MILAN-ROME-NAPLES
STATE RAILWAY LINE
BOLOGNA-FLORENCE SECTION

∅ = 13.5 M.
GROUND: SILTY SANDS WITH GRAVEL INTERBEDDING
MAX. OVERBURDEN: ~ 500 M.



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JET-GROUNTING

15

JET-GROUNTING

Start	End	Job	Client	Ground	Max overburden [m]	Span [m]	Length [m]	Special techniques
2009	In progress	Pedemontana Lombarda: Solbiate Olona tunnel / Grandate tunnel / Morazzone tunnel	Pedelombarda S.p.A.	Gravelly Sands, conglomerates / Glaciofluvial deposits, sandstones / Moraine, marly sandstones	15 / 40 / 70	18 / 16 / 16	2x460 / 2x400 / 2x2050	VJG, HJG / HJG, FGT / HJG FGT
2009	In progress	Palermo railway junction - Through tunnel	SIS S.p.A.	Sands and calcarenites	15	8	1700	1700 m HJG, FGT
2008	In progress	Quadrilatero delle Marche - 14 tunnels	C.M.C. G.L.F. Strabag	Loose deposits, limestones, marly limestones, marls	350	14	30.000	BU, HJG, FGT
2008	In progress	Asti-Cuneo Motorway Link - Alba and Verduno tunnels	Sina S.p.A.	Sandstone and gypsum	95	15	2x3400	VJG, HJG, FGT
1991	In progress	Milan-Naples A1 motorway modernization: tunnels in the Bologna-Florence section	Toto S.p.A. Todini S.p.A. Baldassini Tognozzi Pontello S.p.A. Impresa S.p.A.	Sands, clays, scaly clays, flysch	400	14.5	2x45000	FGT, HJG, BU
2008	2009	Quadrilatero delle Marche - 14 tunnels	Sina S.p.A.	Loose deposits, limestones, marly limestones, marls	350	14	30.000	BU, HJG, FGT
2000	2006	Tunnels along the slip road between A4 and Valtrompia motorway	Autostrada BS VR VI PD S.p.A.	Limestones		12	1700	
1996	2006	New high speed Bologna-Florence railway line: Pianoro tunnel	Maire Engineering	Scaly clays, marls, silts, conglomerates	160	13.5	10706	9126 m FGT, 311 m HJG
1996	2006	New high speed Bologna-Florence railway line: Pianoro large chamber	Maire Engineering	Marne, scaly clays	105	30	418	
1996	2006	New high speed Bologna-Florence railway line: Raticosa tunnel	Maire Engineering	Clays scagliose, marne e sandstones	515	13.5	10380	6109 m FGT, 40 m HJG, BU
2002	2005	Provincial road No. 169-166: Parscera tunnel	Locatelli			12	1700	
1996	2005	New high speed Bologna-Florence railway line: Monte Bibele tunnel	Maire Engineering	Marne e sandstones	285	13.5	9118	2327 m FGT, 62 m HJG, BU
1996	2005	New high speed Bologna-Florence railway line: Vaglia tunnel	Maire Engineering	Argillites, marly limestones, sandstones	500	13.5	18647	7342 m FGT, 525 m HJG, BU
2001	2004	Rome Road System: underground connection between Foro Italico Road and Pineta Sacchetti Road	Astaldi	Sands and clayey silts	35	14.7	2500	FGT, HJG
1996	2004	New high speed Bologna-Florence railway line: Sadurano tunnel	Maire Engineering	Conglomerates, silty sandstones	240	13.5	3778	877 m FGT, 65 m HJG, A.G.O.
2002	2003	Catania-Siracusa motorway: 5 tunnels	Metropolitana Milanese S.p.A.			12	2x6500	
2000	2003	Bologna Airport ring connection: artificial tunnels	COOP Costruttori/CCC	Silts, sands and gravels	5	11	1900	
1999	2003	Salerno-Reggio Calabria motorway: Vetrano 1 and Vetrano 2 tunnels	Toto	Conglomerates, silty sandstones	30	16	2x1000	

Start	End	Job	Client	Ground	Max overburden [m]	Span [m]	Length [m]	Special techniques
1998	2003	State Road No. 42, Darlo-Edolo section: Capo di Ponte tunnel	Grandi Lavori Fincosit	Sandstones ans siltites	90	12.5	800	
1996	2003	New high speed Bologna-Florence railway line: Firenzuola tunnel	Maire Engineering	Marly-arenaceous Formation	560	13.5	14340	3319 m FGT, 1543 m J.G.O
2000	2002	New Viola Canal: hydraulic tunnels and water intake works	Impregilo Valviola	Dolomites, phyllites	800	4	14000	
1998	2002	State Road No. 1 "Aurelia": Marinasco tunnel	San Benedetto	Sandstones and argillites	25	12	2x500	
1990	2002	"Aurelia" State Road No. 1: Montenero tunnel	Impregilo	Scaly clay Formation	50	11	2x2150	2x150 m HJG, 2x2350 m FGT
1999	2001	"Aurelia" State Road No. 1 (variant road near Savona): 4 tunnels	Bonifica	Limestones	20	13	6000	
1998	2001	"Aurelia" State Road No. 1 (variant road near La Spezia): 5 tunnels	Bonifica	Argillites	60	13	10000	
1998	2001	Aosta-Monte Bianco motorway: Pre Saint Didier tunnel	Spea Ingegneria Europea S.p.A.	Quaternary deposits, "Tarantasia" Flysch, sandstones, schists and calceschists	660	12	2x2650	2x250 m HJG
1997	2001	Aosta-Mont Blanc motorway: Morgex tunnel	R.A.V.	Schists, calceschists and Tarantasia Flysch	350	12	2x2300	2x450 m HJG
1996	2001	New high speed Bologna-Florence railway line: Borgo Rinzelli tunnel	Maire Engineering	Clays	10	13.5	455	160 m HJG, 295 m MP, 295 m FGT, A.G.O.
1996	2001	New high speed Bologna-Florence railway line: Morticine tunnel	Maire Engineering	Marly arenaceous siltites	10	13.5	273	193 m FGT, 80 m HJG, A.G.O.
1996	2001	Ravone railway yard at Bologna: Ravone tunnel	C.M.C. Adanti	Sands and silty gravels	14	18	2x900	2x900 m HJG
1994	2001	New high speed Rome-Naples railway line: Collatina tunnel	Iricav Uno	Pyroclastites	8	13.5	55	
1994	2001	New high speed Rome-Naples railway line: Massimo tunnel	Iricav Uno	Pyroclastites, lava	34	13.5	1139	120 m FGT
1994	2001	New high speed Rome-Naples railway line: Colli Albani tunnel	Iricav Uno	"Colli Albani" volcanites	75	13.5	6357	353 m HJG, 90 m FGT
1994	2001	New high speed Rome-Naples railway line: Sgurgola tunnel	Iricav Uno	Limestones Formation	114	13.5	2237	FGT
1994	2001	New high speed Rome-Naples railway line: Macchia Piana 1 tunne	Iricav Uno	"Valle del Sacco" volcanites	43	13.5	970	103 m FGT
1994	2001	New high speed Rome-Naples railway line: Macchia Piana 2 tunnel	Iricav Uno	Pyroclastites	15	13.5	480	480 m FGT
1994	2001	New high speed Rome-Naples railway line: La Botte tunnel	Iricav Uno	Pyroclastites, lava, clays	52	13.5	1185	498 m FGT
1994	2001	New high speed Rome-Naples railway line: Castellona tunnel	Iricav Uno	Clays	60	13,5	469	119 m FGT

Start	End	Job	Client	Ground	Max overburden [m]	Span [m]	Length [m]	Special techniques
1994	2001	New high speed Rome-Naples railway line: St. Arcangelo tunnel	Iricav Uno	Pyroclastites, marls	48	13.5	580	
1994	2001	New high speed Rome-Naples railway line: Selva Piana tunnel	Iricav Uno	Pyroclastites	14	13.5	132	34 m HJG, 98 m FGT
1994	2001	New high speed Rome-Naples railway line: Collevento tunnel	Iricav Uno	Pyroclastites, clays	19	13.5	380	380 m FGT
1994	2001	New high speed Rome-Naples railway line: Selvotta tunnel	Iricav Uno	Clays	11	13.5	163	65 m HJG, 48 FGT
1994	2001	New high speed Rome-Naples railway line: Colle Pece tunnel	Iricav Uno	Scaly clays	33	13.5	873	HJG, FGT
1994	2001	New high speed Rome-Naples railway line: Campo Zillone 1 tunnel	Iricav Uno	"Rocca Monfina" volcanites	48	13.5	2616	FGT, HJG
1994	2001	New high speed Rome-Naples railway line: Campo Zillone 2 tunnel	Iricav Uno	Pyroclastites	25	13.5	350	17 m HJG
1994	2001	New high speed Rome-Naples railway line: Piccilli 1 tunnel	Iricav Uno	Pyroclastites	17	13.5	842	288 m FGT, A.G.O.
1994	2001	New high speed Rome-Naples railway line: Lompari tunnel	Iricav Uno	Clays	13	13.5	200	46 m HJG, A.G.O.
1994	2001	New high speed Rome-Naples railway line: Caianello tunnel	Iricav Uno	Pyroclastites	10	13.5	830	42 m HJG, A.G.O.
1994	2001	New high speed Rome-Naples railway line: Briccelle tunnel	Iricav Uno	Marly-Arenaceous Complex, Limestone and Ignimbrite Formations	78	13.5	1033	4 m HJG, 288m FGT
1994	2001	New high speed Rome-Naples railway line: Castagne tunnel	Iricav Uno	Pyroclastites	8	13.5	289	A.G.O.
1998	2000	P.te Mammolo-Via della Bufalotta road link: Capo di Ponte urban tunnel	Comune di Roma					
1993	1999	Ancona-Bari railway line: Vasto tunnel	Fioroni	Silty clays	135	12	6800	2260 m HJG, 2600m MP, 4970 m FGT
1987	1999	Udine-Tarvisio railway line: Camporosso tunnel	Carnia	Alluvium and lacustrine depositts		12	6900	650 m HJG
1985	1999	Sibari-Cosenza railway line: 1, 2, 3 and 4 tunnels	Asfalti Sintex	Pliocenic Calabrian Formation	115	10	7000	1300m HJG, 2300 m MP, 2300 m FGT
1986	1998	Udine-Tarvisio railway line: Malborghetto tunnel	Carnia	Dolomites, argillites, sandstones, limestones, rubble-slope	800	12	8000	150 m HJG
1985	1998	Udine-Tarvisio railway line modernisation: S. Leopoldo tunnel	Carnia	Limestones, dolomites, rubber-slope	600	12	5600	50 m HJG
1984	1998	Messina-Palermo motorway: S. Elia tunnel	Costruzioni Callisto Pontello S.p.A.	"Reitano" Flysch	150	12	2x1100	2x150 m HJG
1987	1997	Florence-Empoli railway line: S. Vito and Bellosguardo tunnels	Firem	Le Piatre marls, chaotic complex in calcareous-marly facies, M. Modino sandstones	160	12	3510	255 m HJG, 1600 m FGT

Start	End	Job	Client	Ground	Max overburden [m]	Span [m]	Length [m]	Special techniques
1992	1996	E 45 - Orte-Ravenna motorway: Quarto tunnel	Toto	Sandstones, marls and sandy silts	150	11	2x2500	2x100 m HJG, 2x200 m FGT
1991	1996	Caserta-Foggia railway line: S. Vitale tunnel	San Vitale Scarl	Scaly clays, limestones, argillites	100	12	2500	300 m MP, 1300 m FGT
1991	1996	Como Lake and Spluga No. 639 and 36 State Road: Monte Barro tunnel		Limestones and dolomites	300	11	7000	2 x 100 m HJG
1991	1996	"Val d'Esino" No. 76 State Road: Balzete, Mavaioli, Vetralia and Monticelli tunnels	Pizzarotti	Sedimentary rocks	80	10	6000	BU
1990	1996	Aosta-Mont Blanc motorway: Leverogne tunnel	R.A.V.	Morainic deposits	300	12	2x1800	2x270 m HJG
1990	1996	Aosta-Mont Blanc motorway: Les Cretes tunnel	R.A.V.	Morainic deposits	60	12	2x1300	2x450 m HJG
1990	1996	Aosta-Mont Blanc motorway: Chabodey tunnel	R.A.V.					
1989	1996	Aosta-Mont Blanc motorway: Avise tunnel	R.A.V.	Paragneiss and calceschists	400	12	2x2700	2x290 m HJG
1988	1996	"Trafori" Motorway: Mottarone 1 and Mottarone 2 tunnels	Italstrade	Micaschists and moraine		11	2900	HJG
1991	1995	"Val Brembana" No. 470 State Road: Lenna tunnel	Valbrembana	"Esnio" limestone	350	12	2150	30 m HJG
1990	1994	Livorno-Civitavecchia motorway: Pipistrello tunnel	Sotecni	Calcarenites and marls	50	10	2x700	
1990	1994	Aosta-Mont Blanc motorway: Villaret tunnel	R.A.V.	Morainic deposits	300	12	2x2700	2x240 m HJG
1989	1994	Aosta-Mont Blanc motorway: Villeneuve tunnel	R.A.V.	Calceschists, micaschists and fractured paragneiss, morainic deposits			2x3220	275 m HJG
1984	1994	Udine-Tarvisio railway line: Monte Palis tunnel	Cogefar	Dolomites, marly limestones	200	12	3734	200 m HJG, BU, DR
1984	1994	Udine-Tarvisio railway line modernisation: Zuc del Bor and S. Rocco tunnels	Carnia	Dolomites, morainic deposits, limestones	1400	12	9200	HJG
1983	1994	Udine-Tarvisio railway line: Campiolo tunnel	Italstrade	Rubber slope	350	12	1840	170 m HJG
1989	1993	Livorno-Civitavecchia motorway: Rimazzano tunnel	Sotecni	Sands e gravels in silty matrix, pliocenic clays	20	12	1800	2x900 m HJG, 2x900 m FGT
1988	1993	Nuraxi Figus mining inclined shaft	Torno S.p.A.	Volcanites, "Cixerri" Formation, eocenic series	460	8		150 m HJG, FGT
1991	1992	State Road No. 45 Bis (Lot 3): Monte Covolo tunnel	Tormini	Limestones	100	11	3500	HJG, IN
1991	1992	"Sebina Orientale" No. 510 State Road - Lot 5: Campiolo and Palis tunnels	Secol	Limestones	150	11	4000	
1989	1992	Livorno-Civitavecchia motorway: Malenchini tunnel	Eurocons	Fine silty sands	20	12	2x900	2x900 m HJG

Start	End	Job	Client	Ground	Max overburden [m]	Span [m]	Length [m]	Special techniques
1989	1991	Ofanto Aqueduct - Pavoncelli tunnel	Sidion	Marly formation	100	5	4800	200 m FGT
1989	1991	Underpass of Via C.Colombo (Rome): Capitan Bavastro tunnel	Comune di Roma	Sandy silts	7	12.5	150	150 m HJG.
1988	1991	Rome-L'Aquila-Teramo motorway- Lot 4: Colledara tunnel	Colledara	Marls and lacustrine deposits	50	10	1600	
1987	1991	"Trafori" motorway - Gattico-Carpugnino section: Massino Visconti tunnel	Selp	Micaschists e moraine	10	11	2x2800	HJG
1987	1991	"Direttissima" Rome-Florence railway line: Talleto and Caprenne tunnels	FE.SPI	Sandy silts	80	7	2700+2700	2500m HJG, 5500m MP, 2700m FGT
1987	1991	"Direttissima" Rome-Florence railway line: Poggio Orlandi tunnel	FE.SPI	Sandy silts	50	13.5	1200	250 m HJG, 600 m FGT
1987	1991	"Direttissima" Rome-Florence railway line: Crepacuore tunnel	FE.SPI	Sandy silts	50	13.5	700	60m HJG, 120 m FGT
1987	1991	"Direttissima" Rome-Florence railway line: Tasso tunnel	FE.SPI	Sandy silts	50	13.5	2000	150m HJG, 1650m FGT
1987	1991	"Direttissima" Rome-Florence railway line: Terranova Le Ville tunnel	FE.SPI	Lacustrine deposits	50	13.5	2600	200 m HJG, 1800 m MP, 2200 m FGT
1989	1989	"Tonale and Mendola" No. 42 State Road: Lovere tunnel	C.S.P.	Limestones and dolomites	300	11	2800	200 m HJG
1984	1989	Milan-Chiasso railway line: Monte Olimpino 2 tunnel	Cogefar	Alluvium, sandstones, marls		12	7200	800 m HJG, CG
1987	1988	Campinas railway yard underpass	Comune di Campinas (Brasile)	Sands	2.5÷4.0	12.5÷16.5	250	HJG
1986	1988	Doubling of the Bari - Taranto railway line: Madonna del Carmine tunnel	BA.TA	Pleistocenic clays	60	12	4000	200 m HJG
1986	1988	Doubling of the Bari - Taranto railway line: St. Francesco tunnel	BA.TA	Pleistocenic clays	50	12	2600	100 m HJG
1984	1985	Paola-Cosenza railway line: Santomarco tunnel	Gambogi	Weathered phyllites	80	7	350	150 m HJG
1983	1983	Railway underpasses in Barcellona (Spain)		Backfill	2.5	8	700	HJG
1994		Roccella Jonica No. 106 State Road: Le Grazie tunnel		Silty sands, clayey silts	45	12	780	100 m HJG 160 m
1993		Roccella Jonica No. 106 State Road: Lofiri tunnel		Clays, marls	35	12	350	250 m HJG
1993		Roccella Jonica No. 106 State Road: Giulia tunnel		Granitic sands and scaly clays	40	12	780	500 m HJG, 280 m FGT

Start	End	Job	Client	Ground	Max overburden [m]	Span [m]	Length [m]	Special techniques
1992		Milan-Rome A1 motorway: M. Mario tunnel		Silty clays, sands	270	14.5	4600	1200 m MP, 25 m HJG, 350 m FGT, 150 m BU
1991		"Sebina Orientale" No. 510 State Road - Lot 6 tunnels	Secol	Limestones and dolomites	150	11	5000	900 m HJG, 400 m FGT
1991		"Sebina Orientale" No. 510 State Road - Lot 7 tunnels	Secol	"Lombard Cerrucano" limestones, gypsum and anhydrites	150	11	5500	900 m HJG, 400 m FGT
1991		State Road No. 237: Sabbio tunnel				11		300 m HJG
1990		State Road No. 38: Valmaggiore and Bolladore tunnels	Secol	Diorites, gneiss, gabbros, phyllites	300	12	2400	150 m HJG, FGT
1990		State Road No. 38: Mondadizza tunnel	Secol	Diorites, gneiss, gabbros, phyllites	300	12	1400	60 m HJG, FGT
1990		State Road No. 38: Le Prese and Verzedo tunnels	Secol	Diorites, gneiss, gabbros, phyllites	300	12	3100	100 m HJG, FGT
1990		State Road No. 38: St. Antonio tunnel	Secol	Diorites, gneiss, gabbros, phyllites	300	12	2300	40 m HJG, BU
1990		State Road No. 38: Tola tunnel	Secol	Diorites, gneiss, gabbros, phyllites	300	12	1500	80 m J.G.O, BU
1990		State Road No. 38: Cepina tunnel	Secol	Diorites, gneiss, gabbros, phyllites	300	12	2900	70 m HJG, BU
1990		"Trafori" motorway - Lot 3: Vevera tunnel	Autostrade	Sands	5	12	300	HJG
1989		Catanzaro East by-pass: St. Giovanni tunnel		Stratified marly clays and yellowish sands and silts alternate under water-table	40	12	400	
1988		"Trafori" motorway: Valsesia tunnel		Moraine	26	15	2x600	2x600 m HJG
1988		Roma-L'Aquila-Teramo motorway - Lot 4: Sodera tunnel		Marls and lacustrine deposits		10		
1987		Udine-Tarvisio railway line: Tarvisio tunnel	Carnia	Dolomites, sandy and silty gravels	70	12		1000 m HJG
1987		Reggio Calabria C.Le-Metaponto railway line: Capo d'Armi tunnel		Limestones	70	12	1000	125 m HJG, 360 m FGT
1987		"Trafori" motorway - Gattico-Carpugnino section: Campiglia tunnel		Micaschists and moraine		12	2x2900	HJG
1987		Doubling of Circumflegrea (Naples) railway line: Varo Pecore, Astroni and Grotta del Sole tunnels		Tufs with fluvial-lacustrine intercalations	30	6	600	
1986		Sonico-Cedegolo hydraulic tunnel in Val Camonica	Selm Montedison	Scaly clays		4.5		
1985		West Campania waterworks: Cassino tunnel	Cogefar	Limestones, dolomites		4.5	3200	
1983		Cadì Tunnel (Spain)		Marls		11	5028	

JET GROUTING PORTAL SHELL STRUCTURE

MESSINA-PALERMO MOTORWAY

∅ = 12 M.
GROUND: SILTY-CLAYEY DEBRIS
MIN. OVERBURDEN: 2 M.

BIBLIOGRAPHY

LUNARDI P., CASSANI G., GATTI M., 2008. Design Aspects of the construction of the new Apennines crossing on the A1 Milan-Naples motorway: the Base Tunnel, Proceedings of the International Congress on "Le Souterrain: espace d'avenir", Monaco, Principato di Monaco, 6-8 ottobre 2008

LUNARDI P., 2008. Design and construction of tunnels - Analysis of Controlled Deformation in Rock and Soils (ADECO-RS), Ed. SPRINGER, 576 pagine, marzo 2008

LUNARDI P., CASSANI G., DE GIUDICI C., 2007. Design and executive aspects of the construction of the line 1 of the Naples metro, Proceedings of the International Symposium on "Tunnelling for urban Development", Pattaya City, Thailand, 14-15 dicembre 2007

LUNARDI P., BINDI R., CASSANI G., 2007. From the ADECO-RS approach to the tunnelling industrialisation, Atti del Simposio Internazionale su "Tunnelling for urban development", Pattaya City (Thailand), 14 - 15 dicembre 2007

LUNARDI P., 2007. Progetto e costruzione di gallerie - Analisi delle deformazioni controllate nelle rocce e nei suoli (ADECO-RS), Ed. HOEPLI, 575 pagine, agosto 2007

LUNARDI P., VOZA A., 2007. L'assistenza tecnica in corso d'opera durante lo scavo delle gallerie Dolonne, Strade e Autostrade, 2-2007

DE GIUDICI C., ROTUNDI F., LAROSA I., 2006. Fermata "Quattro Venti". Le scelte costruttive e il monitoraggio, Gallerie e Grandi Opere Sotterranee, 1-2006

LUNARDI P., BINDI R., CASSANI G., 2006. Prime evidenze e risultati dell'impiego dell'approccio ADECO-RS per la realizzazione di oltre 73 Km di gallerie di linea, Strade e Autostrade, 1-2006

BINDI R., CASSANI G., 2004. New prospects for tunnelling with the ADECO-RS approach, Proceedings of EUROCK 2004 & 53[th] Geomechanics Colloquy - Salisburgo, 7-9 ottobre 2004

LUNARDI P., BINDI R., 2004. The evolution of reinforcement of the advance core using fibre glass elements for short and long term stability of tunnels under difficult stress-strain conditions: design, technologies and operating methods, Felsbau, 4-2004

LUNARDI P., CASSANI G., 2003. The Genoa Underground: construction of the Principe-Caricamento-Le Grazie stretch, Proceedings of the ITA-AITES World Tunnel Congress on "(Re)Claiming the underground space", Amsterdam, 12-17 aprile 2003

LUNARDI P., CASSANI G., 2002. La realizzazione del sottovia allo scalo ferroviario Ravone nella città di Bologna: aspetti progettuali e costruttivi, Strade & Autostrade, 3-2002

LUNARDI P., 2001. Attendibilità della progettazione: risultato dell'applicazione dell'approccio ADECO-RS per il progetto e la costruzione delle opere in sotterraneo della linea ferroviaria ad alta velocità Roma-Napoli. Confronto tra i progetti esecutivo, costruttivo ed "as built", Strade & Autostrade, 5-2001

LUNARDI P., 2001. The ADECO-RS approach in the design and construction of the underground works of Rome to Naples High Speed Railway Line: a comparison between final design specification, construction design and "as built", Atti del Congresso Internazionale su "Progress in Tunnelling after 2000", Milano, 10-13 giugno 2001

LUNARDI P., CASSANI G., 2001. Construction of an underpass at the Ravone railway yard in the city of Bologna: aspects of the design and construction, Atti del Congresso Internazionale su "Progress in Tunnelling after 2000", Milano, 10 - 13 giugno 2001

LUNARDI P., BINDI R., 2001. The evolution of reinforcement of the advance core using fibre glass elements for short and long term stability of tunnels under difficult stress-strain conditions: design, technologies and operating methods, Atti del Congresso Internazionale su "Progress in Tunnelling after 2000", Milano, 10 - 13 giugno 2001

LUNARDI P., LEONI F., VALENTE A., 2001. Full scale test of grouting and freezing for new extension of Rome underground, line B1, Atti del Congresso Internazionale su "Progress in Tunnelling after 2000", Milano, 10-13 giugno 2001

LUNARDI P., 2001. Progetto e costruzione di gallerie – Approccio ADECO-RS, Quarry and Construction, maggio 2001

LUNARDI P., 2000. Aspetti progettuali e costruttivi della realizzazione di un viadotto ferroviario su terreni solubili ed aggressivi, Strade & Autostrade, 6-2000

LUNARDI P., PANTALEONE V., 2000. Problematiche affrontate nello scavo delle gallerie Bellosguardo e San Vito, Quarry and Construction, ottobre 2000.

LUNARDI P., 2000. Design & constructing tunnels – ADECO-RS approach, T&T International special supplement, May 2000

LUNARDI P., 2000. Tunnelling under the Via Appia Antica in Rome, Tunnels & Tunnelling International, aprile 2000

LUNARDI P., MINENNA M., 2000. La galleria di sottopasso della strada consolare romana "Appia Antica", Strade e autostrade, n. 2 anno 2000

LUNARDI P., 1999. L'influenza della rigidezza del nucleo d'avanzamento sulla stabilità degli scavi in galleria/ The influence of the rigidity of the advance core on the stability of tunnel excavations, Gallerie e grandi opere sotterranee, n. 59 anno 1999

LUNARDI P. E ALTRI, 1999. L'impiego di pozzi-prova nella realizzazione di opere in sotterraneo in area urbana, Atti del XX Convegno Nazionale di Geotecnica - Parma, 22 - 25 settembre 1999

LUNARDI P., 1997. L'influenza della rigidezza del nucleo d'avanzamento sulla sicurezza degli scavi in galleria, Gallerie e grandi opere sotterranee, n. 52 anno 1997

LUNARDI P. E ALTRI, 1997. Aspetti progettuali e costruttivi della galleria "Vasto", Quarry and Construction, agosto 1997

LUNARDI P., 1997. Ground improvement by means of jet-grouting, Ground improvement, vol. I, n. 2, aprile 1997

LUNARDI P., ARRIGONI E., 1997. Scavi profondi in zona urbana: opere di sottofondazione e di contenimento di scavi profondi sotto un edificio antico in Milano, Quarry and Construction, marzo 1997

LUNARDI P.,POZZI V., 1996. Aspetti progettuali ed esecutivi della realizzazione della galleria Avise, Le Strade, marzo 1996

LUNARDI P., ARRIGONI E., 1995. Fondazioni di un viadotto stradale su terreni altamente compressibili, Quarry and Construction, ottobre 1995

LUNARDI P., AGOSTONI S., PRESBITERO M., 1995. Valtellina 1987, the Val Pola landslide: nature and cause of the phenomenon, first emergency interventions, final plan of interventions, 44[th] Geomechanics Colloquy - Salisburgo, 12-13 ottobre 1995

LUNARDI P., 1995. L'importanza del precontenimento del cavo in relazione ai nuovi orientamenti in tema di progetto e costruzione di gallerie, Gallerie e grandi opere in sotterraneo, n. 45 anno 1995

LUNARDI P., 1993. La stabilité du front de taille dans les ouvrages souterraines en terrain meuble: etudes et experiences sur le renforcement du noyau d'avancement, Symposium international "Renforcement des sols: experimentations en vraie grandeur des annes 80", Parigi, 18 novembre 1993

LUNARDI P., 1993. Evolution des technologies d'excavation en souterrain, dans des terrains meubles, Colloque chez le "Comité Marocain des Grands Barrages" - Rabat, 30 settembre 1993

LUNARDI P. E ALTRI, 1993. Tunnelling in soil: design, construction materials and monitoring, Seminario Russo-Italiano - Central Institute for Advanced Training of Construction Engineers (TSMIPKS), Mosca, 11 Marzo 1993

LUNARDI P., 1993. La Torre di Pisa - Note su una proposta d'intervento alternativa, Materiali e strutture, gennaio 1993

LUNARDI P. E ALTRI, 1992. Tunnel face reinforcement in soft ground design and controls during excavation, Convegno Internazionale su "Towards New Worlds in Tunnelling" - Acapulco 16-20 Maggio 1992

LUNARDI P., 1992. Il consolidamento del terreno mediante jet-grouting, Quarry and Construction, marzo 1992

LUNARDI P., 1991. Le gallerie idrauliche di by-pass nel progetto di regimazione del lago formato dalla frana di Val Pola, Convegno su "I grandi trasferimenti d'acqua. Sistemi integrati" - Cortina d'Ampezzo 17-19 luglio 1991

LUNARDI P. E ALTRI, 1991. Potenziamento linea a doppio binario Verona-Brennero: metodologia progettuale delle opere in sotterraneo, Quarry and Construction, giugno 1991

LUNARDI P., 1991. Aspetti progettuali e costruttivi nella realizzazione di gallerie in situazioni difficili: interventi di precontenimento del cavo, Convegno Internazionale su "Il consolidamento del suolo e delle rocce nelle realizzazioni in sotterraneo" - Milano 18-20 Marzo 1991

LUNARDI P., 1991. L'impiego del jet-grouting per l'attacco di gallerie in terreni soffici, Convegno Internazionale su "Il consolidamento del suolo e delle rocce nelle realizzazioni in sotterraneo" - Milano 18-20 Marzo 1991

GOLINELLI G., LUNARDI P., PERELLI CIPPO A., 1991. La prima applicazione del jet-grouting in orizzontale come precontenimento delle scavo di gallerie in terreni incoerenti, Convegno Internazionale su "Il consolidamento del suolo e delle rocce nelle realizzazioni in sotterraneo" - Milano 18-20 Marzo 1991

LUNARDI P. E ALTRI, 1991. Il preconsolidamento del terreno a mezzo di iniezioni jet-grouting per la galleria a tre binari "Tarvisio" della Linea F.S. Udine-Tarvisio, Convegno Internazionale su "Il consolidamento del suolo e delle rocce nelle realizzazioni in sotterraneo" - Milano 18-20 Marzo 1991

LUNARDI P. E ALTRI, 1991. Preconsolidamento del terreno mediante jet-grouting nel sottopasso della Tangenziale Est di Milano da parte della Linea 3 della metropolitana, Convegno Internazionale su "Il consolidamento del suolo e delle rocce nelle realizzazioni in sotterraneo" - Milano 18-20 Marzo 1991

LUNARDI P. E ALTRI, 1991. L'impiego del jet-grouting per lo scavo di una galleria urbana a debole profondità, Convegno Internazionale su "Il consolidamento del suolo e delle rocce nelle realizzazioni in sotterraneo" - Milano 18-20 Marzo 1991

BIAGI G., PELLEGRINI V., LUNARDI P., 1991. L'applicazione del sistema jet-grouting nel progetto e nella costruzione delle gallerie dell'autostrada Livorno-Civitavecchia, Convegno Internazionale su "Il consolidamento del suolo e delle rocce nelle realizzazioni in sotterraneo" - Milano 18-20 Marzo 1991

LUNARDI P. E ALTRI, 1991. Il ripristino di gallerie collassate sull'autostrada Messina-Palermo, lotto 33 bis, mediante armatura radiale del terreno, Convegno Internazionale su "Il consolidamento del suolo e delle rocce nelle realizzazioni in sotterraneo" - Milano 18-20 Marzo 1991

LUNARDI P. E ALTRI, 1990. Soft ground tunnelling in the Milan Metro and Milan Railway Link. Case histories, Soft Ground Tunnelling Course - Institution of Civil Engineers - Londra 10-12 Luglio 1990

LUNARDI P., AIELLO E., RIZZO S., MONGILARDI E., 1990. Una frana in terreni geologicamente e strutturalmente complessi interessante l'abitato della Presura di Impruneta (Firenze), Quarry and Construction, n. 4 anno 1990

LUNARDI P. E ALTRI, 1989. Nouvelles orientations pour le project et la constru-ction des tunnels dans des terrains meubles. Etudes et experiences sur le preconfinement de la cavite et la preconsolidation du noyau au front, Colloque International "Tunnels et micro-tunnels en terrain meuble - Parigi 7-10 Febbraio 1989

LUNARDI P., 1988. La costruzione della galleria S. Elia lungo l'autostrada Messina-Palermo, Costruzioni, dicembre 1988

LUNARDI P., 1988. Emergenza Valtellina - frana di Val Pola - gallerie idrauliche di by-pass, Quarry and Construction, n. 11 anno 1988

CAMARGO GUAZZELLI J.L., LUNARDI P., 1988. Tunel de Campinas: evolucion del diseno y problemas de operacion, Atti del Congresso Internazionale su "Le gallerie e l'Acqua" - Madrid, 12-15 Giugno 1988

LUNARDI P., 1988. Emergence "Valtellina"; éboulement de la Val Pola; galeries hydrauliques de by-pass, Atti del Congresso Internazionale su "Le gallerie e l'Acqua" - Madrid, 12-15 Giugno 1988

LUNARDI P., MONGILARDI E., TORNAGHI R., 1986. Il preconsolidamento mediante jet-grouting nella realizzazione di opere in sotterraneo, Atti del Congresso Internazionale su "Grandi Opere Sotterranee" - Firenze 8-11 Giugno 1986

BALOSSI RESTELLI A.,COLOMBO A.,GERVASO F., LUNARDI P., 1986. Tecnologie speciali per il sostegno di scavi nelle alluvioni di Milano in occasione della costruzione della linea 3 della metropolitana milanese, Atti del Congresso Internazionale su "Grandi Opere Sotterranee" - Firenze 8-11 Giugno 1986

FAORO L., LUNARDI P., PASQUINI M., 1986. Il jet-grouting nel consolidamento di imbocchi di gallerie autostradali, Atti del Congresso Internazionale su "Grandi Opere Sotterranee" - Firenze 8-11 Giugno 1986

LOUIS C., LUNARDI P., 1984. Consolidation des sols par la technique de jet- grouting. Etat des connaissances et experiences, Colloque International "Renforcement en place des sols des roches". Parigi, Ottobre 1984.

LUNARDI P. LOUIS C., 1984. Méthodes de présoutènèment et pré-étanchement pour les travaux en souterrain, Journèes d'etudes internationales - Lione, Novembre 1984

LUNARDI P., 1984. La piena del fiume Taro del novembre 1982. Danni, ripristino provvisorio, ricostruzione definitiva del ponte ferroviario distrutto dalla piena. II Convegno di Idraulica Padana - Parma, Giugno 1984.

LUNARDI P., 1982. Nouvelle Mèthode de construction des écrans d'étanchéité dans les barrages en remblai, 14[th] International Congress on "Large Dam" - Rio de Janeiro, 6-7 Maggio 1982.

1985 MECHANICAL PRECUTTING

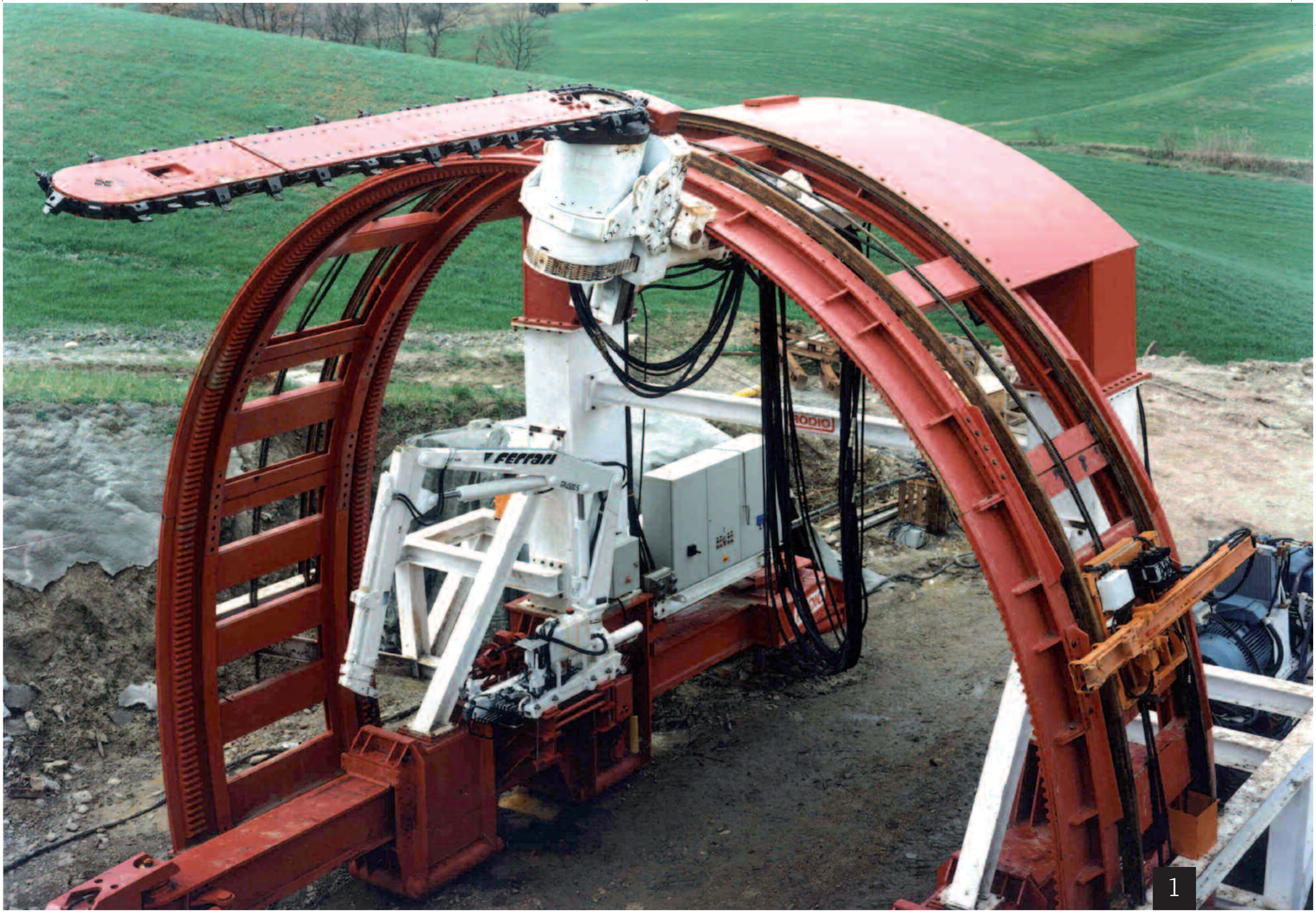
Mechanical precutting employed full-face

In 1985 Prof. Ing. Pietro Lunardi introduced mechanical precutting to Italy and he proposed its use for full-face applications in tunnels for the first time. Before then tunnel designers did not possess the necessary technology for excavation in soft cohesive soils and they were obliged to design tunnel advance operations consisting of a number of headings, and to control deformation by using very generic means such as steel ribs, rock bolts and shotcrete. The introduction of full-face mechanical precutting in 1985 therefore constituted a true and genuine milestone in the history of tunnelling under difficult stress-strain conditions.

The technology consists of making an incision of a predetermined thickness and length around the profile of the extrados of the future tunnel. The incision is made by using a special machine equipped with a chain cutter which moves on a rack and pinion portal that reproduces the shape of the tunnel outline (Figure 1) and it is immediately filled with fibre reinforced sprayed concrete with appropriate additives to give it rapid and excellent strength.

A pre-lining "tile" is thereby created with a truncated cone shape and high quality mechanical characteristics, which projects well ahead of the face to

TERRANOVA LE VILLE TUNNEL
ROME-FLORENCE STATE RAILWAY LINE
∅ = 13.50 M.
GROUND: SILTY SANDS
OVERBURDEN: ~ 50 M.



1

1. THE FIRST MACHINE FOR FULL FACE MECHANICAL PRECUTTING
SIBARI-COSENZA STATE RAILWAY LINE

provide radial preconfinement of the surrounding ground sufficient to prevent the rock mass around it from loosening. Its truncated cone shape allows a succession of partially overlapping tiles to be cast (Figure 2), alternating the placing of each tile with appropriate tunnel advance.

A practically continuous lining arch is obtained in this manner, which is immediately held rigid by casting the side walls and the tunnel invert to close the ring.

The scope of application ranges from soft rocks to clayey soils and silty-sandy soils, including heterogeneous grounds and aquifers, provided that they allow the incision to remain open, perhaps with a little assistance, for the whole time needed to fill it.

Important features of the method are as follows:

- the almost total elimination of overbreak with a consequent appreciable reduction in the need for backfill grout injections between the preliminary lining and the ground;
- a reduction in the amount of temporary confinement placed because it is practically all replaced by the precut shell;

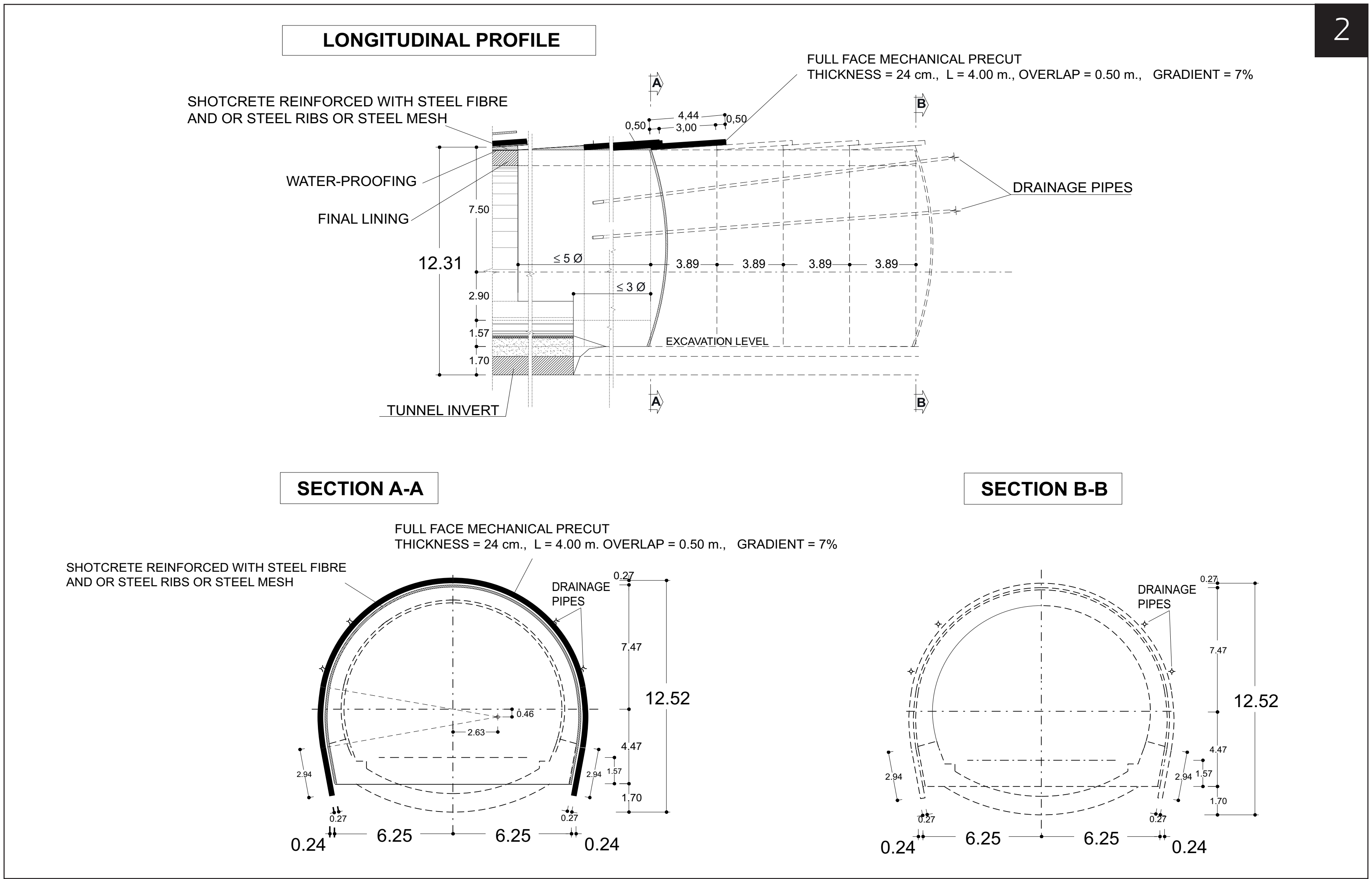
THE CORE-FACE OF A TUNNEL
PROTECTED BY A PRECUT SHELL

SIBARI-COSENZA STATE RAILWAY LINE
TUNNEL NO. 2

∅ = ~ 10 M.
GROUND: CLAY AND SILTS
OVERBURDEN: ~ 110 M.

DETAIL OF MECHANICAL
PRECUT AND FILLING THE
PRECUT WITH SHOTCRETE

2



▸ the very high degree of mechanisation and regular advance rates with advantageous repercussions on construction site costs and the production rates that can be achieved;

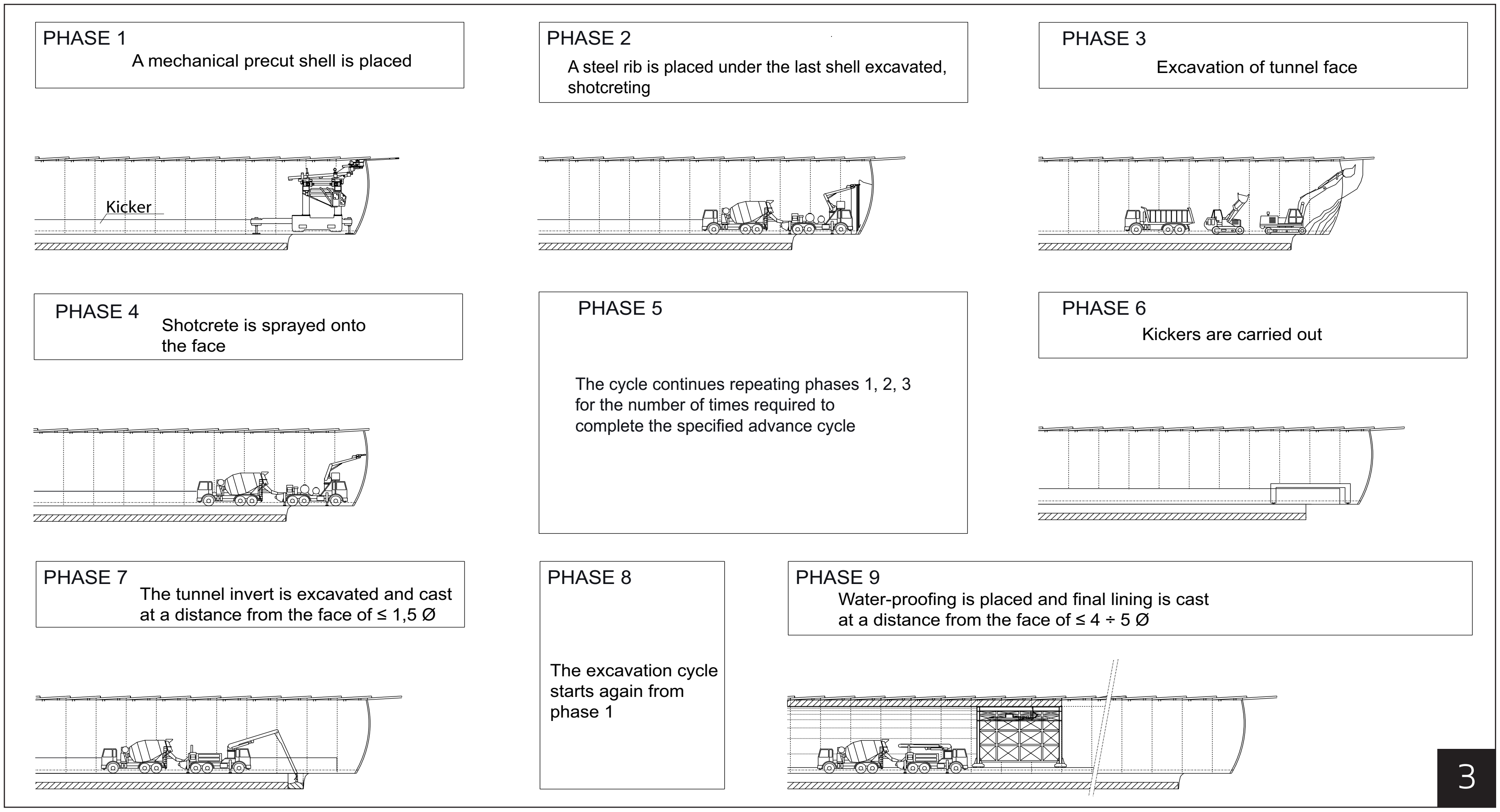
▸ the construction of a preliminary lining which works as closely as possible with the statics of the final lining, so that the thickness of the latter can be conveniently reduced, if the specifications allow the effect of that combined action to be taken into account.

An example of a tunnel section type that employs full face mechanical precutting is given in Figure 2, while Figure 3 illustrates the operational stages of tunnel advance. Within the framework of the ADECO-RS approach, the technology forms part of cavity preconfinement operations and it can be performed in combination with other ground improvement work ahead of the face and classified into two types, to be selected on the basis of the type of ground and the stress-strain conditions to be tackled (Figure 4):

1. (PT): simple protection of the core-face by means of pre-vaults of shotcrete created in advance by using mechanical precutting (indirect conservative works);

2. ((PT+VTR): fibre glass reinforcement of the face-core and protection of the core at the same time by means of pre-vaults of shotcrete created in advance by using mechanical precutting (mixed conservative technique).

2. *AN EXAMPLE OF A FULL FACE MECHANICAL PRECUTTING SECTION TYPE*



3. *THE OPERATIONAL STAGES FOR FULL FACE MECHANICAL PRECUTTING*

TABLE 1. *EACH OF THESE MAIN TYPES CAN BE ASSOCIATED WITH THE RELATIVE PROJECT IN WHICH IT WAS USED EXPERIMENTALLY*

Relative project		Tunnel length [m]	Type of ground	Max overburden [m]	Diameter [m]	Type of ground improvement work
Work	Tunnel					
Sibari-Cosenza railway line [1983]	Tunnel no. 1 and Tunnel no. 2	2200	Sandy silt	115	10	MP
Rome metro [1997]	Baldo degli Ubaldi	120	Clays and clayey silts	4	22	MP+FGT
A1 Rome-Milan motorway	Nazzano	2x600	Sands	45	21	MP+FGT

Each of these types can be associated with the relative project (see Table 1) in which it was either used experimentally for the first time or developed in a particular way.

A SHORT HISTORY OF THE TECHNOLOGY

The idea of creating a prelining shell in advance by immediately filling a cut of predetermined thickness and length with sprayed concrete, made by means of "predecoupage mecanique" technology on the face, around the profile of the extrados of the tunnel to be bored, was conceived of and developed in 1981 in France, where it was experimented half-face during the construction of the Lille metro on a section consisting of cohesive argillites.

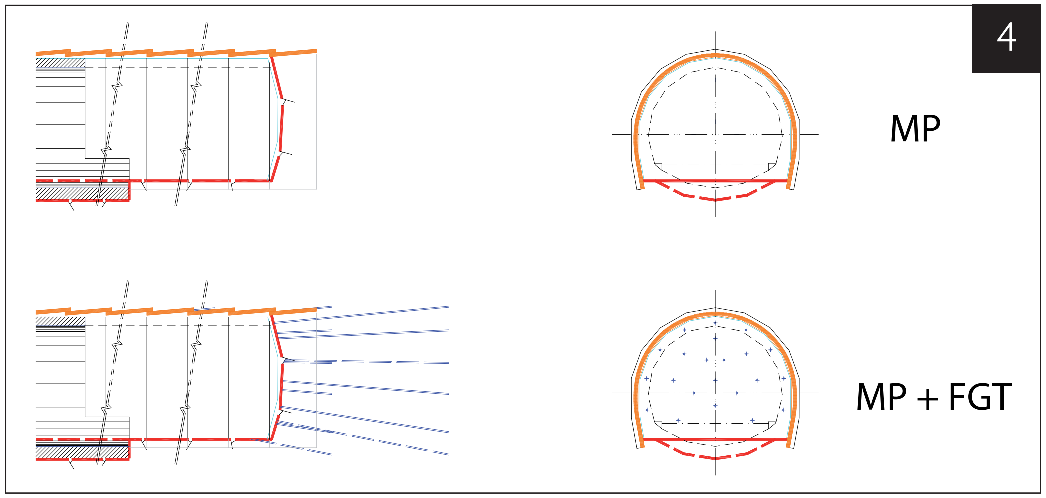
The prelining shell, created before excavating the section of the advance core below it, prevented the disappearance of the confinement action which the core had previously exerted, thereby improving the safety of site personnel and appreciably reducing settlement on the surface.

This French initiative aroused great interest on the part of Rocksoil, which at that time was about to design some difficult tunnels on the Sibari-Cosenza railway line, between the S. Marco Roggiano and Mongrassano-Cervicati stations. The grounds to be tunnelled, with maximum overburdens of around 100 m., did in fact consist of very soft grey-blue clays belonging to the Pliocenica-Calabriana formation.

The calculations performed to assess the behaviour of the tunnels during excavation in the absence of intervention predicted serious face and cavity instability. In similar situations, tunnel advance using conventional systems had always created a series of difficulties in planning the works, which were conducted under precarious safety conditions with frequent incidents, making any forecast of construction times and costs uncertain. In fact the triggering of enormous and irreversible deformation phenomena which it was not possible to avoid with those systems, caused basically by the disappearance of the confinement action exerted by the core of ground ahead of the face, led to frequent tunnel collapses.

After the positive experience with horizontal jet-grouting acquired on the Campiolo tunnel, it seemed clear that the only way of working successfully would be to prevent deformation phenomena by intervening before the arrival of the face with appropriate stabilisation systems. The problem to be addressed was therefore to conceive of a new type of intervention appropriate to the fairly cohesive nature of the ground in question, which like horizontal jet-grouting, was capable of anticipating deformation by developing continuous and effective preconfinement and confinement action before, during and after the arrival of the face.

To achieve this, Prof. Ing. Pietro Lunardi provided the input to revise the French idea in order to transform it into a method that could be applied effectively in cohesive soils which were very much softer than those at Lille. Since one of the most delicate stages of tunnel advance in poor quality soils is that of excavating down from half to full face, the technology was redesigned to allow it to be applied full-face. This required, amongst other things, careful study to perfect the geometry of the precut shells and appropriate modifications to ensure that they rested on a reliable base.



4. FULL FACE MECHANICAL PRECUTTING: THE MAIN TYPES OF IMPLEMENTATION

The effectiveness of the new technology was clear immediately. The extreme regularity of tunnel advance rates, which stabilised at average speeds of around 3.00 m./day, despite the difficult ground, allowed construction operations, times and costs to be planned in a manner never before possible working in similar materials.

The numerous in situ measurements taken to study the behaviour of the excavation with the new technology confirmed the designer's expectations. They showed that the movements, which usually develop in the ground before the passage of the face had been practically eliminated, while the maximum gradient of deformation was reached during the excavation of the tunnel invert, which when cast, nevertheless determined the immediate stabilisation of the cavity. Since the ground had conserved most of its original characteristics, thanks to the preconfinement, even the long term stress acting on the lining had been considerably reduced compared to the stresses that would have been expected with conventional tunnel advance.

EVOLUTION OF THE TECHNOLOGY

The potential of mechanical precutting has increased considerably since it was first used. Modern technology for cutting the ground and filling the incision will cut tiles to a depth of more than 4.5 m. with a thickness of 24 cm., while in terms of tunnel diameter, the system has already been successfully applied on spans of around 21.5 m. (Baldo degli Ubaldi Station on Line A of the Rome metro).

One particular application of mechanical precutting recently developed and tried out was its use as part of the "Nazzano" method (see page 280), with which it is possible to widen an existing tunnel without interrupting traffic.

A list is given below of all the most important projects implemented using Rocksoil designs, which involved the use of full-face mechanical precutting technology.

Start	End	Job	Client	Ground	Max overburden [m]	Span [m]	Length [m]	Special techniques
2006	In progress	A14 motorway – Widening to 3 lanes for each direction - 4 new tunnels and widening of Monte Domini tunnel by using "Nazzano method"	Spea Ingegneria Europea S.P.A.	Clays	20	16	2x280	Patented method to widen the old tunnel without interrupting traffic
2000	2007	Milan-Naples A1 motorway: Nazzano tunnel widening without interrupting traffic	Autostrade per l'italia S.p.A.	Sands	45	21	1200	Patented method to widen the old tunnel without interrupting traffic
1996	2001	New high speed Bologna-Florence railway line: Borgo Rinzelli tunnel	Maire Engineering	Clays	10	13.5	455	160 m HJG, 295 m MP, 295 m FGT, A.G.O.
1993	1999	Ancona-Bari railway line: Vasto tunnel	Fioroni	Silty clays	135	12	6800	2260 m HJG, 2600 m MP, 4970 m FGT
1985	1999	Sibari-Cosenza railway line: 1, 2, 3 and 4 tunnels	Asfalti Sintex	Pliocenic Calabrian Formation	115	10	7000	1300 m HJG, 2300 m MP, 2300 m FGT
1996	1999	Milan-Rome A1 motorway: Monte Mario tunnel	Astaldi	Sands and cemented sands, silty sands, clayey silts	270	14.5	2x2300	1200 m MP, 25 m HJG, 350 m FGT 150 m BU
1989	1998	Sardinian Dorsal railway line: Campeda tunnel	Cofesar	Volcanic grounds	300	12	3800	400 m MP, 400 m FGT
1988	1997	Rome metro - Line A - "Baldo degli Ubaldi" station	Intermetro S.p.A.	Clayey and sandy-silty formations	22	22	120	MP, FGT, AA
1991	1996	Caserta-Foggia railway tunnel: San Vitale tunnel	San Vitale Scarl	Scaly clays, limestones, argillites	100	12	2500	300 m MP, 1300 m FGT
1989	1994	Bicocca-Siracusa railway line: Targia tunnel	Collini S.p.A.	Hyaloclastites, calcarenites	60	12	3300	850 m MP, 1000 m FGT
1987	1991	Linea Ferroviaria "Direttissima" Roma-Firenze: gallerie Talleto e Caprenne	FE.SPI	Sandy silts	80	7	2700+2700	2500 m HJG, 5500 m MP, 2700 m FGT
1987	1991	"Direttissima" Rome-Florence railway line: Terranova Le Ville tunnel	FE.SPI	Lacustrine deposits	50	13.5	2600	200 m HJG, 1800 m MP, 2200 m FGT
1989	1991	Catanzaro East by-pass: S. Giovanni tunnel	Sincat Scarl	Stratified marly clays and yellowish sands and silts alternate under water-table	40	12	400	400 m MP, 400 m FGT
1987	1989	Doubling of Circumflegrea (Naples) railway line: Varo Pecore, Astroni and Grotta del Sole tunnels		Tufs with fluvial-lacustrine intercalations	30	6	600	600 m MP, 600 m FGT

"BALDO DEGLI UBALDI" STATION

ROME METRO, LINE "A"

BIBLIOGRAPHY

LUNARDI P., CASSANI G., GATTI M., 2008. Design Aspects of the construction of the new Apennines crossing on the A1 Milan-Naples motorway: the Base Tunnel, Proceedings of the International Congress on "Le Souterrain: espace d'avenir", Monaco, Principato di Monaco, 6-8 ottobre 2008

LUNARDI P., 2008. Design and construction of tunnels - Analysis of Controlled Deformation in Rock and Soils (ADECO-RS), Ed. SPRINGER, 576 pagine, marzo 2008

LUNARDI P., BINDI R., CASSANI G., 2007. From the ADECO-RS approach to the tunnelling industrialisation, Atti del Simposio Internazionale su "Tunnelling for urban development", Pattaya City (Thailand), 14-15 dicembre 2007

LUNARDI P., LUNARDI G., CASSANI G., 2007. Widening the Nazzano motorway tunnel from two to three lanes + an emergency lane without interrupting traffic, Atti del convegno Internazionale su "Tunnels, drivers of change", Madrid, 5-7 Novembre 2007

LUNARDI P., 2007. Progetto e costruzione di gallerie - Analisi delle deformazioni controllate nelle rocce e nei suoli (ADECO-RS), Ed. HOEPLI, 575 pagine, agosto 2007

LUNARDI P., BINDI R., CASSANI G., 2006. Prime evidenze e risultati dell'impiego dell'approccio ADECO-RS per la realizzazione di oltre 73 Km di gallerie di linea, Strade e Autostrade, 1-2006

LUNARDI P., 2003. Widening the load at Nazzano, Tunnels & Tunnelling International, Luglio 2003

LUNARDI P., 2003. Un metodo costruttivo innovativo per allargare gallerie stradali, autostradali o ferroviarie senza interrompere il traffico: l'applicazione alla galleria Nazzano sull'autostrada A1 Milano-Napoli, Strade & Autostrade, 2-2003

LUNARDI P. E ALTRI, 2001. Aspetti progettuali e costruttivi affrontati nell'ambito dei lavori per il raddoppio della linea ferroviaria Bicocca-Siracusa, tratto Targia-Siracusa, Atti del Congresso Internazionale su "Progress in Tunnelling after 2000", Milano, 10-13 giugno 2001

LUNARDI P., CALCERANO G., 2001. A new construction method for widening highway and railway tunnels, Atti del Congresso Internazionale su "Progress in Tunnelling after 2000", Milano, 10-13 giugno 2001

LUNARDI P., 2000. Progetto e costruzione di gallerie – Approccio ADECO-RS, Quarry and Construction, maggio 2001

LUNARDI P., 2000, The construction of large-span stations for underground railways, Tunnel, n. 8 (dicembre), anno 2000

LUNARDI P., PANTALEONE V., 2000. Problematiche affrontate nello scavo delle gallerie Bellosguardo e San Vito, Quarry and Construction, October 2000

LUNARDI P., 2000. Design & constructing tunnels – ADECO-RS approach, T&T International special supplement, May 2000

LUNARDI P., 1999. The influence of the rigidity of the advance core on the stability of tunnel excavations, Gallerie e grandi opere sotterranee, 59

LUNARDI, 1999. "Une methode de construction innovante pour elargir les tunnels routiers, autoroutiers et ferroviaires sans interrompre la circulation; son application au tunnel de Nazzano sur l'autoroute A1 Milan-Naples", Atti della Conferenza su "Instandsetzung von Tunneln" - Olten, 21 Ottobre 1999

LUNARDI P., 1999. Il preconsolidamento del terreno nelle opere in sotterraneo, Quaderno della 3° Sessione delle Conferenze Permanenti SIG sull'Alta Velocità, Bologna, 24 giugno 1999

LUNARDI P.,1999. Construction des stations de grandes portées pour métro, ETH-Tunnelbau-Symposium 99, Zurigo, 18 marzo 1999

LUNARDI P., 1998. Convergence-confinement ou extrusion-préconfinement ?, Colloque "Mécanique et Géotechnique", Laboratoire de Mécanique des Solides - École Polytechnique, Paris 19 May 1998

LUNARDI P. E ALTRI, 1998. Design and construction of a station on the Rome metro, T&T International, March 1998

LUNARDI E ALTRI, 1998. Stazione "Baldo degli Ubaldi", Quarry and Construction, marzo 1998

LUNARDI P., 1997. L'influenza della rigidezza del nucleo d'avanzamento sulla sicurezza degli scavi in galleria, Gallerie e grandi opere sotterranee, n. 52 anno 1997

LUNARDI E ALTRI, 1995. Qualcosa di nuovo in galleria, Le strade, luglio-agosto 1995

LUNARDI P., 1995. L'importanza del precontenimento del cavo in relazione ai nuovi orientamenti in tema di progetto e costruzione di gallerie, Gallerie e grandi opere in sotterraneo, n. 45 anno 1995

LUNARDI P. E ALTRI, 1994. Pianificazione tecnico-economica delle opere in sotterraneo mediante l'approccio basato sull'analisi delle deformazioni controllate nelle rocce e nei suoli, Costruzioni, Settembre 1994

LUNARDI P. E ALTRI, 1994. Il metodo del pretaglio, I treni, Agosto 1994

LUNARDI P., 1993. Evolution des technologies d'excavation en souterrain, dans des terrains meubles, Colloque chez le "Comité Marocain des Grands Barrages" - Rabat, 30 settembre 1993

LUNARDI P. E ALTRI, 1993. Technical and financial planning of underground works: the "ADECO-RS" method, VI Forum Europeo di Ingegneria Economica - Università Bocconi - Milano, 13-14 Maggio 1993

LUNARDI P., 1993. Aspetti progettuali e costruttivi nella realizzazione di gallerie in situazioni difficili. Interventi di precontenimento del cavo, Le strade, marzo-aprile 1993

LUNARDI P. E ALTRI, 1993. Tunnelling in soil: design, construction materials and monitoring, Seminario Russo-Italiano - Central Institute for Advanced Training of Construction Engineers (TSMIPKS), Mosca, 11 Marzo 1993

LUNARDI P. E ALTRI, 1992. Shotcreting applied to shell construction in the precutting tunnelling method, Convegno Internazionale su "Towards New Worlds in Tunnelling" - Acapulco 16-20 Maggio 1992

LUNARDI P. E ALTRI, 1992. Tunnel face reinforcement in soft ground: design and controls during excavation, Convegno Internazionale su "Towards New Worlds in Tunnelling" - Acapulco 16-20 May 1992

MASSARO G., LUNARDI P., 1991. Costruire presto e bene, Ingegneria ferroviaria, settembre 1991

LUNARDI P., 1991. Aspetti progettuali e costruttivi nella realizzazione di gallerie in situazioni difficili: interventi di precontenimento del cavo, Convegno Internazionale su "Il consolidamento del suolo e delle rocce nelle realizzazioni in sotterraneo" - Milano 18-20 Marzo 1991

LUNARDI P. E ALTRI, 1991. La prima applicazione in Italia del pretaglio meccanico, Convegno Internazionale su "Il consolidamento del suolo e delle rocce nelle realizzazioni in sotterraneo" - Milano 18-20 Marzo 1991

LUNARDI P. E ALTRI, 1990. Soft ground tunnelling in the Milan Metro and Milan Railway Link. Case histories, Soft Ground Tunnelling Course - Institution of Civil Engineers - Londra 10-12 Luglio 1990

LUNARDI P. E ALTRI, 1989. Nouvelles orientations pour le projet et la construction des tunnels dans des terrains meubles. Etudes et experiences sur le preconfinement de la cavité et la préconsolidation du noyau au front, Colloquio Internazionale su "Tunnels et micro-tunnels en terrain meuble - Paris 7-10 February 1989

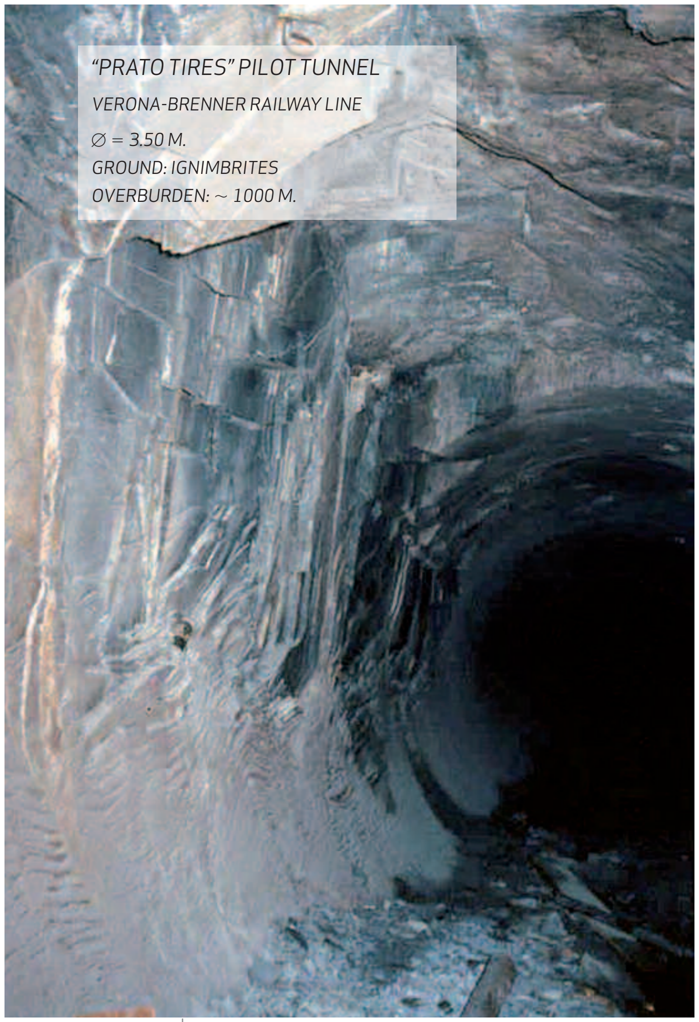
1986 | THE "RS METHOD" OF A PILOT TUNNEL

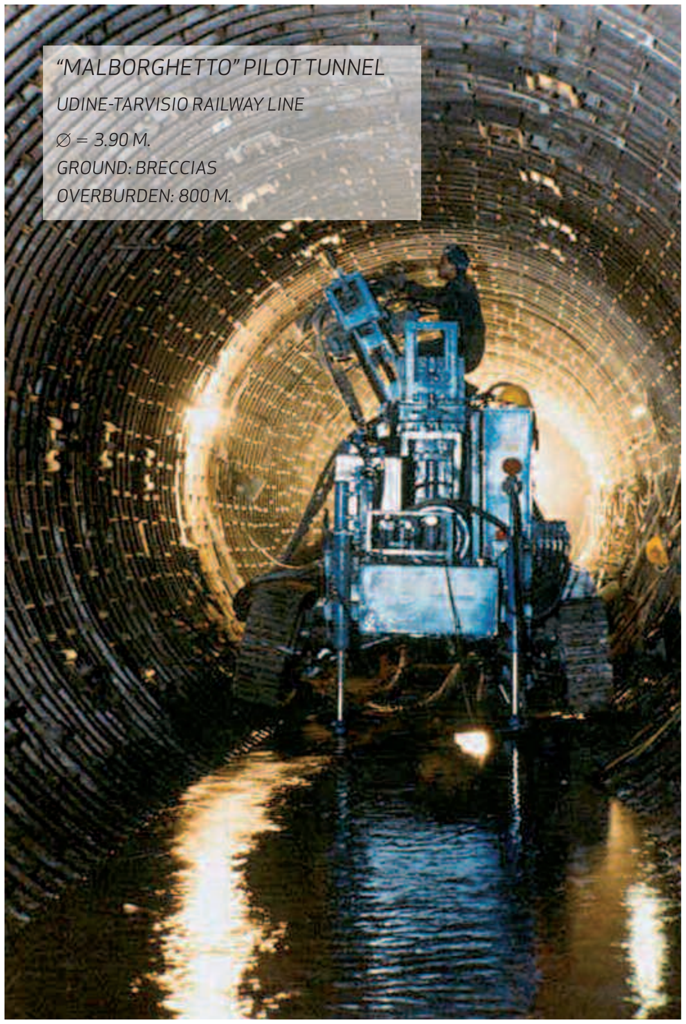
A pilot tunnel becomes a true and genuine means of design integrated in the ADECO-RS approach

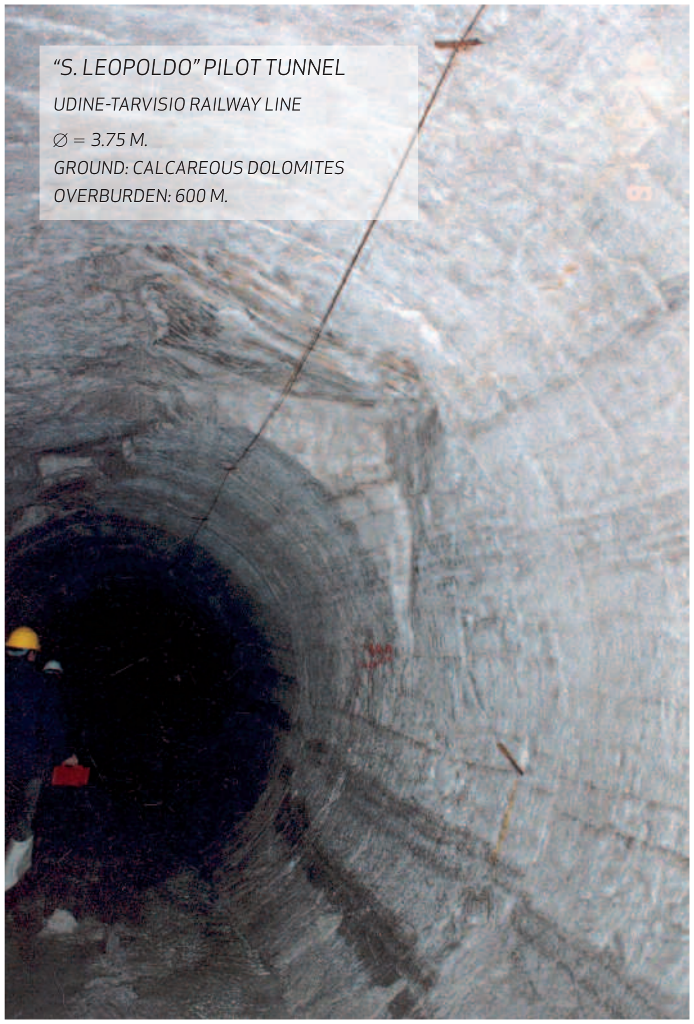
The "RS method" of a pilot tunnel is a powerful tool for the design of tunnels. Prof. Ing. Pietro Lunardi proposed this method to the scientific community in 1986, on conclusion of in-depth studies conducted by Rocksoil in co-operation with Florence University, after having tested its effectiveness during the construction of the "Prato Tires" Tunnel, also known as the "Sciliar" Tunnel, 13.2 km. in length, between Prato Isarco and Ponte Gardena, as part of the works to modernise the Verona-Brennero railway line.

Applications of the RS Method have been very numerous since then (see Table 1 on page 230), so we can definitely say that it is an established technology and that we know its merits and defects very well. It is classified within the framework of the ADECO-RS approach as a survey technology to be considered during the design of the survey stage.

PILOT TUNNELS IN ITALY





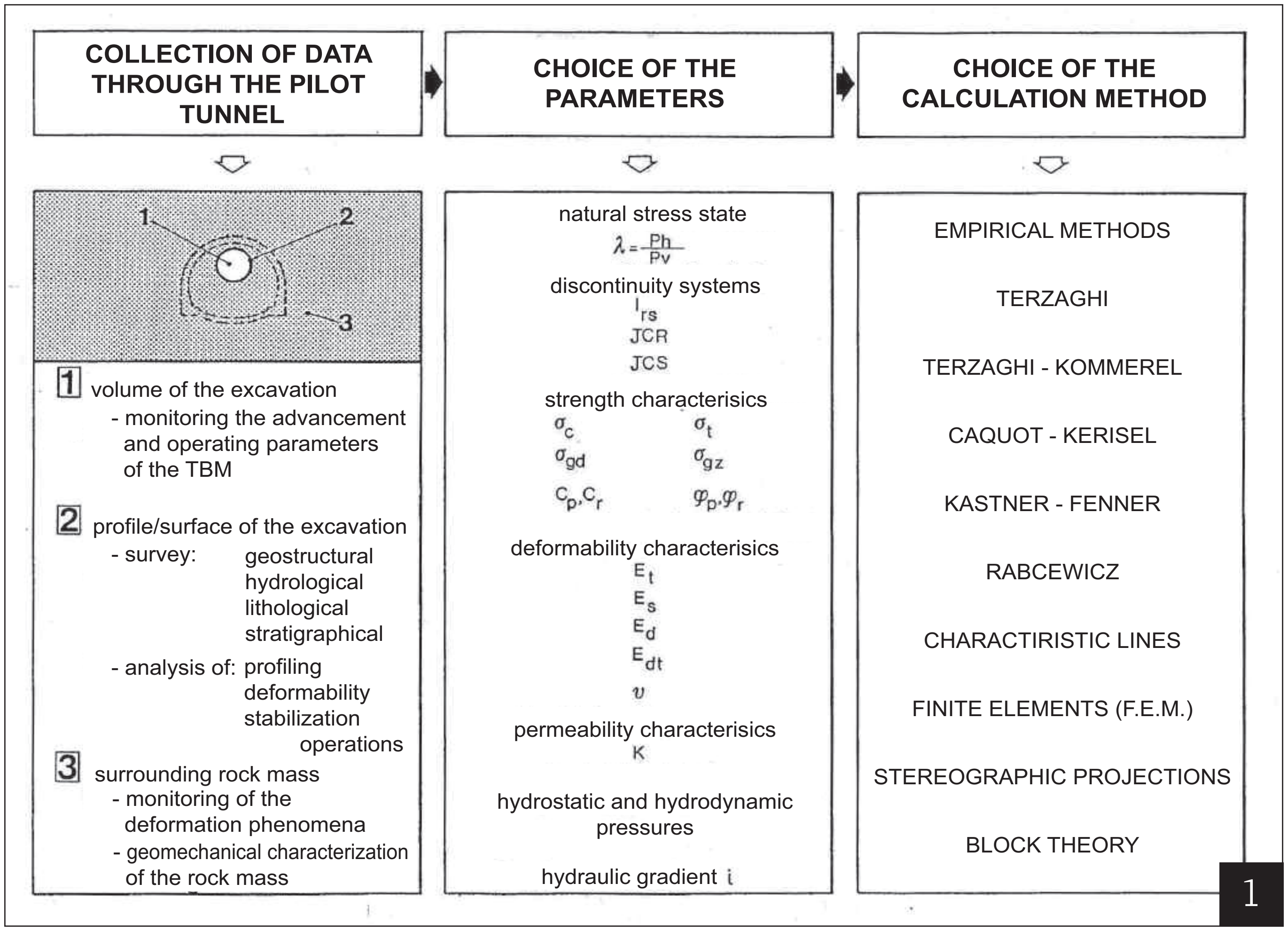


When complex geological and environmental conditions mean that the survey costs required to obtain sufficient information to guarantee a reliable design for a project are too high for an adequate geological survey to be carried out or even make this impossible, then the "RS Method", using a pilot tunnel with all the relative procedures, constitutes a genuine alternative to the normal survey methods employed in tunnel design. A pilot tunnel, driven more or less co-axially to the final tunnel to be excavated by means of a full face TBM of small diameter (3 – 4 m.), is the equivalent of a horizontal borehole performed with destruction of the core from which the rock mass concerned can be surveyed directly and then fully characterised from a lithostratigraphic, structural, geomechanical and hydrogeological viewpoint. It is a useful practice for tunnels of a certain length, in a context of decidedly rocky terrains, where it is not important to be able to count on the presence of a rigid advance core for the excavation of the final tunnel.

The necessary and indispensable conditions for correct application of the RS Method are as follows:

- a preliminary geological survey designed to establish whether a pilot tunnel is feasible. It is only after such a survey has been conducted that any knowledgeable decision can be taken concerning whether it is worthwhile, the best diameter and the most suitable machine to use;
- the use of a full-face, continuous excavation, tunnel boring machine fitted with appropriate instrumentation. This is in fact the only way of rendering the walls of an excavation fully open to inspection and it is then possible to acquire all the geostruc-

THE CONSTRUCTION OF OVER 100 KM OF TUNNELS TO-DATE ON TIME AND TO BUDGET, AFTER FIRST DRIVING A PILOT TUNNEL, CONSTITUTE CLEAR PROOF OF THE VALIDITY OF THIS NEW METHOD OF DESIGNING AND CONSTRUCTING TUNNELS



1. DESIGN USING A PILOT TUNNEL

tural information on the rock masses encountered by means of an appropriate detailed survey;

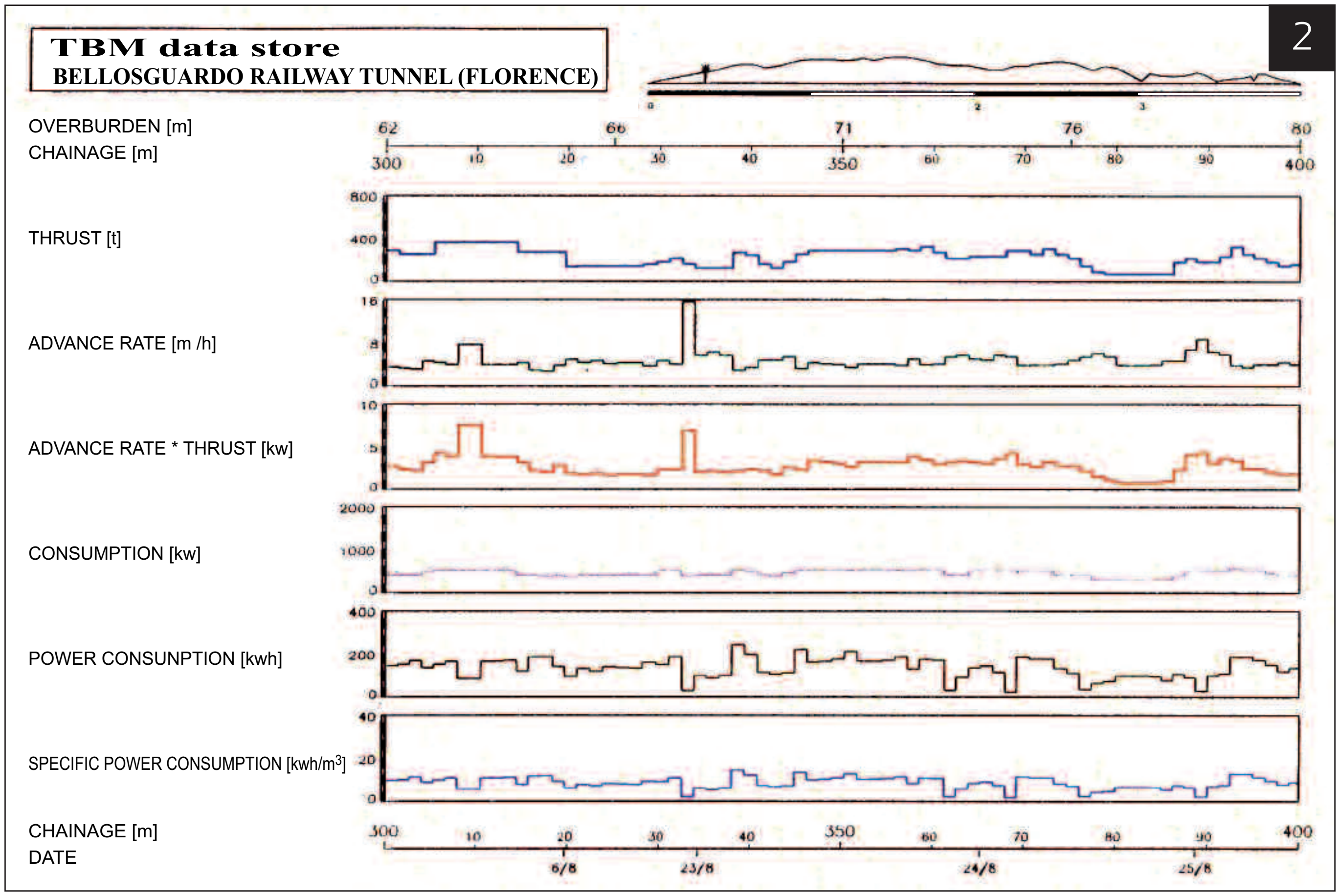
- the construction of a pilot tunnel, if the geology allows it, along the whole of the route of the tunnel to be driven.

Opening a pilot tunnel, given the means and method of execution, makes it possible to use a series of characterisation methods (analyses of the volume of the excavation, of the profile/surface of the excavation and the surrounding rock mass) which, taken together, provide a full and exhaustive picture of all the necessary elements for the subsequent final detailed design of the project (Figure 1):

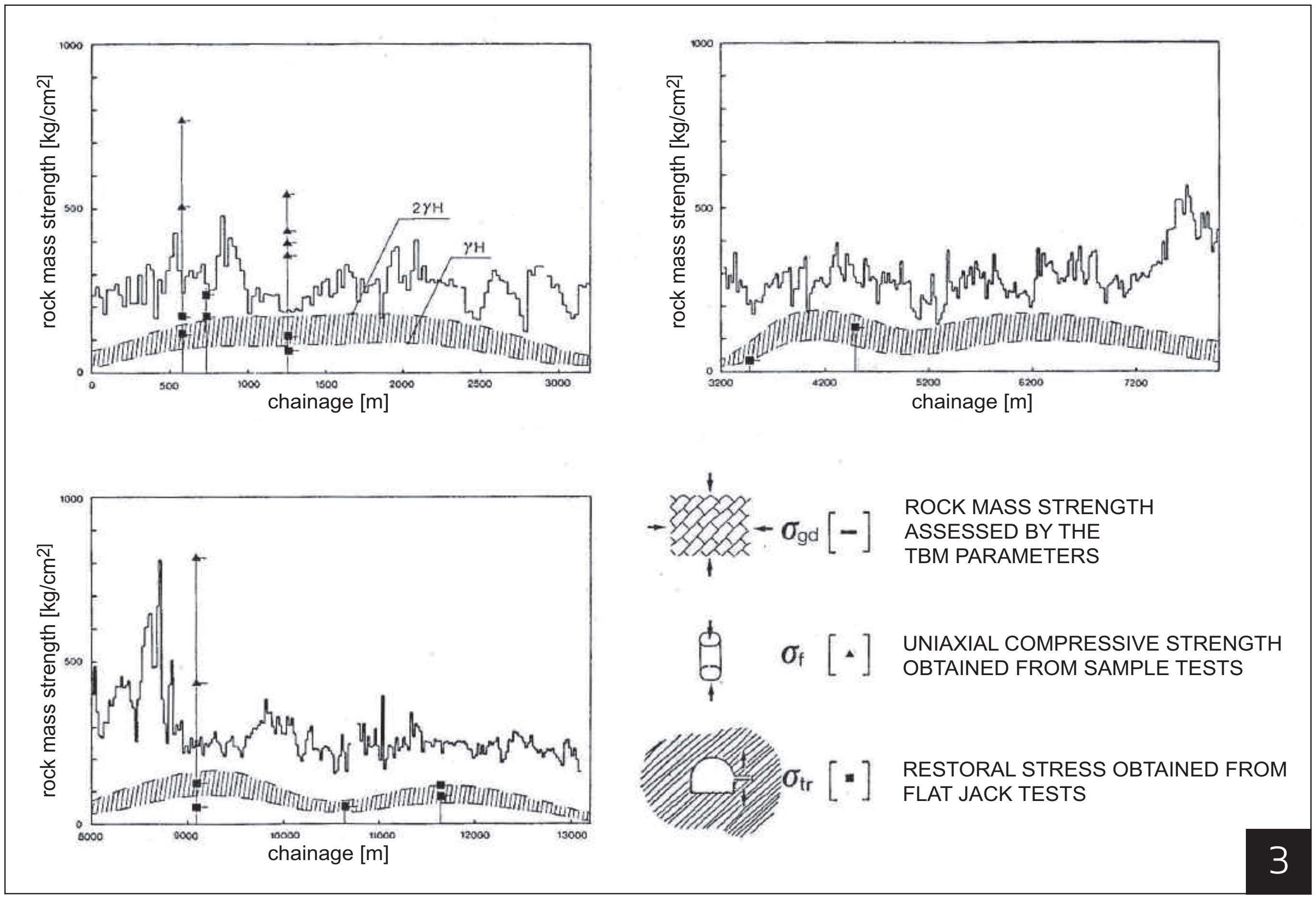
1. by monitoring the operating parameters (thrust, advance rate, power consumed) of the tunnel boring machine, which can be likened to a huge penetrometer, it is possible to calculate the precise energy required to excavate a given unit volume of material (Figure 2) and hence the strength of the rock mass (Figure 3) which correlates directly with it by means of a proportional coefficient to be determined in situ using flat jack tests (analyses of the volume excavated);

2. because of the reduced disturbance produced by the action of the TBM, the surface of the perimeter of the tunnel is like a full scale open book from which all the stratigraphic, lithological, structural, tectonic and hydrogeological features of the rock mass can be read. The "RS Method" involves general collection and automatic archiving of all this data which, with the use of special software, can subsequently be processed and reproduced graphically in any form required (Figure 4) (analysis of the profile/surface of the excavation);

3. a great variety of surveys, tests and measurements can easily be performed from within the pilot tunnel (core sampling, flat and cylindrical jack tests, geophysical investigations, a census and kinematic



2. BY MONITORING THE OPERATING PARAMETERS (THRUST, ADVANCE RATE, POWER CONSUMED) OF THE TUNNEL BORING MACHINE, WHICH CAN BE LIKENED TO A HUGE PENETROMETER, IT IS POSSIBLE TO CALCULATE THE PRECISE ENERGY REQUIRED TO EXCAVATE A GIVEN UNIT VOLUME OF MATERIAL



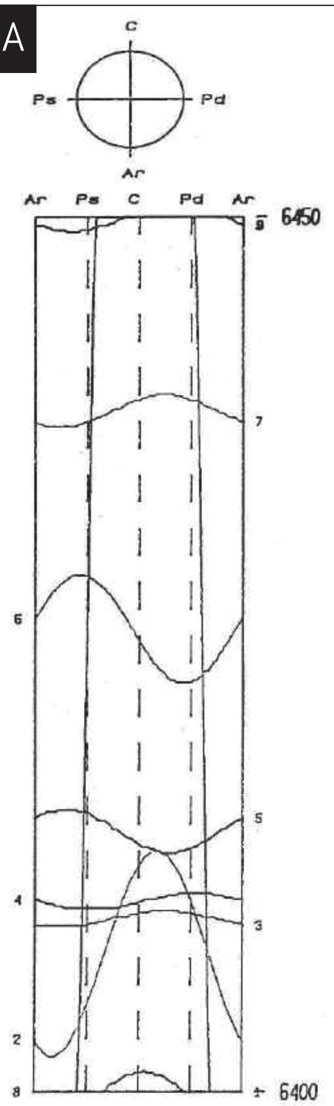
3. *STRENGTH OF THE ROCK MASS*

VERONA-BRENNERO STATE RAILWAY LINE
PRATO TIRES - PONTE GARDENA TUNNEL

VILLENEUVE PILOT TUNNEL
AOSTA - MONTE BIANCO MOTORWAY
∅ = 3.90 M.
GROUND: CALC - SCHISTS AND MICA SCHISTS WITH LENSES OF CARNIOLA CHALK
OVERBURDEN: 55 M.

"MALBORGHETTO" PILOT TUNNEL
UDINE-TARVISIO RAILWAY LINE
∅ = 3.90 M.
GROUND: UGOVIZZA BRECCIA
OVERBURDEN: 800 M.

4A



N	PROG	DIR	INCL	FILL	DEV
1	6399.5	250	45	Pl	5
2	6408	210	35	Pl	27
3	6410	250	80	Pl	11
4	6411	75	88	Pl	11
5	6415	120	70	Pl	12
6	6426.5	150	80	Pl	17
7	6439	250	70	Ce	12
8	6445	185	10	Pl	100
9	6450	250	80	Pl	6

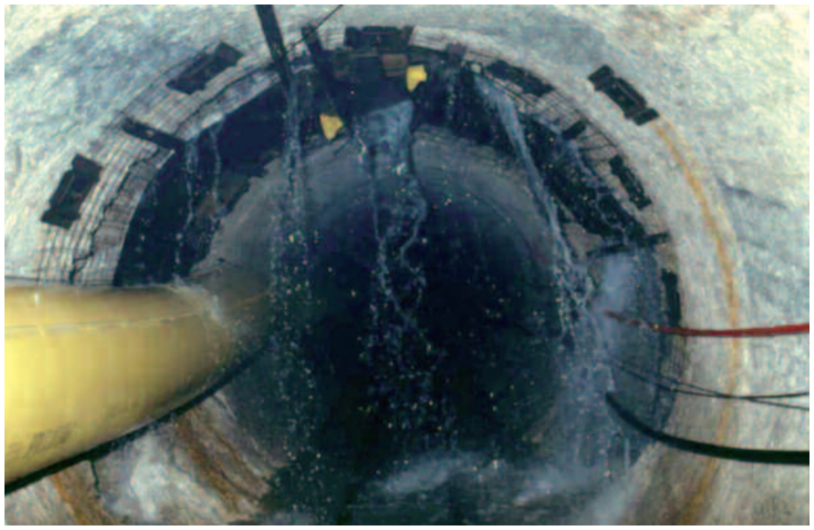
Type of filling

Ab = absent
Lo = loose
Pl = plastic
Ce = cemented

CHAINAGE 6400 - 6450

Number of discontinuities 9
Total development [m] 201
Computation area [m^2] 550

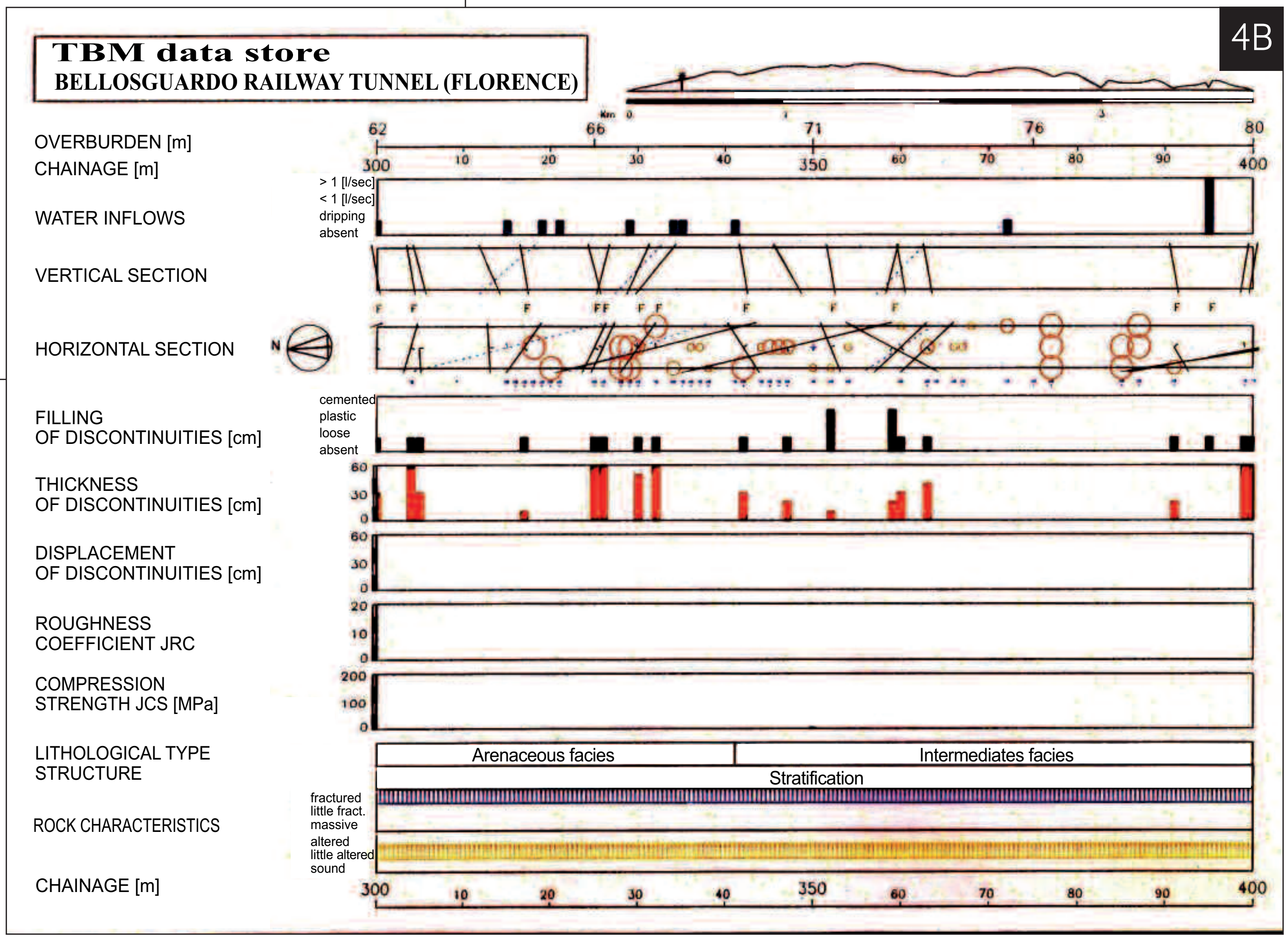
Subdivision index
Irs = .3661954 [m/m^2]



"ZUC DEL BOR"
PILOT TUNNEL
UDINE-TARVISIO RAILWAY LINE

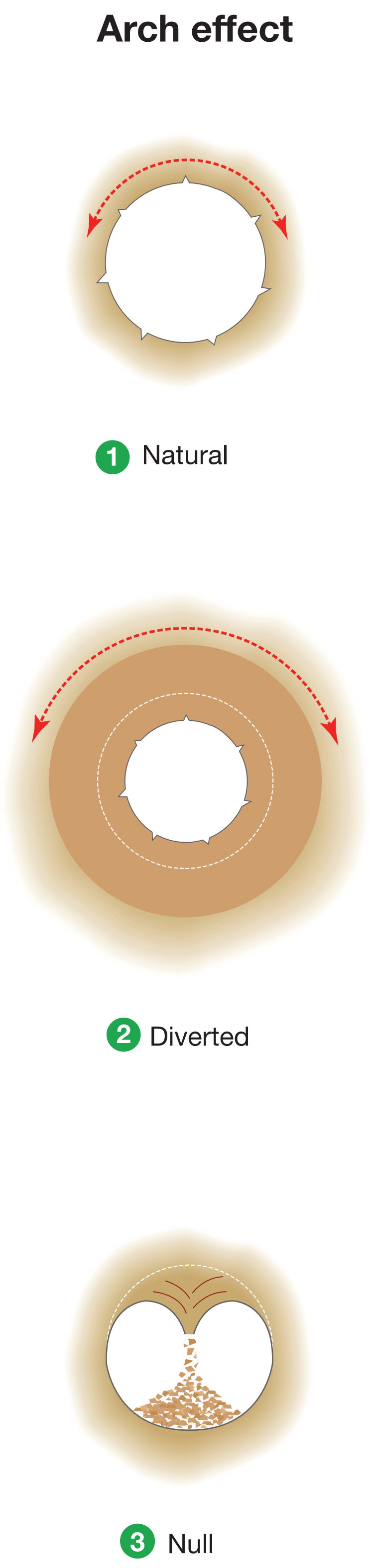
∅ = 3.50 M.
GROUND: DOLOMITES AND LIMESTONES
MAX OVERBURDEN: 1.400 M.

4B

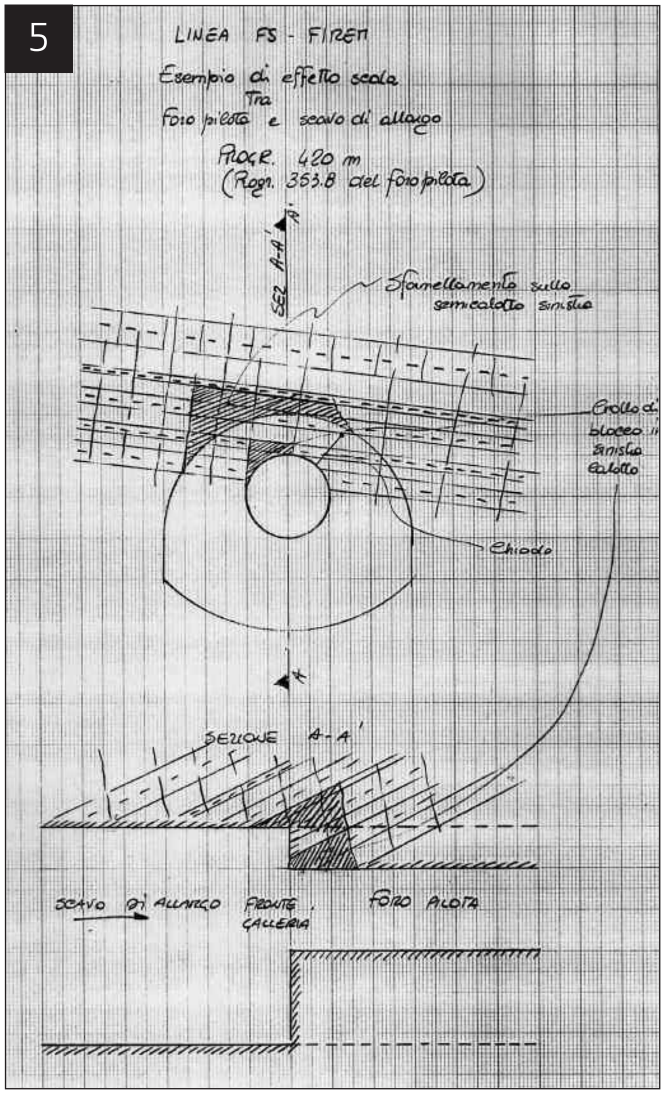


4A. CLINOMETRY OF THE DISCONTINUITIES

4B. ANALYSES OF THE PROFILE/SURFACE OF THE EXCAVATION



THANKS TO THE PILOT TUNNEL, IT WAS POSSIBLE TO CARRY OUT IMPORTANT STUDIES ON THE TRIGGERING OF AN "ARCH EFFECT" AROUND THE CAVITY



5. STUDY OF THE SCALE EFFECT ON ROCK FALLS INTO THE WIDENED TUNNEL

analysis of gravitational fall-ins [Campana M., Lunardi P., Papini M., 1993], convergence measurements, etc.), which give results that are far more reliable than those obtainable from the surface and they can be used immediately for tunnel design (analysis of the surrounding rock mass);

4. by observing and measuring the deformation behaviour of a pilot tunnel, it is possible to predict the type of behaviour that the rock mass will exhibit when widened to the full diameter, with account taken of the scale effect [Scesi L., Papini M., 1997] and the time factor.

From a construction viewpoint it is worthwhile remembering that by working from a pilot tunnel, it is possible to perform all the necessary ground improvement and reinforcement work before widening it to the final tunnel diameter.

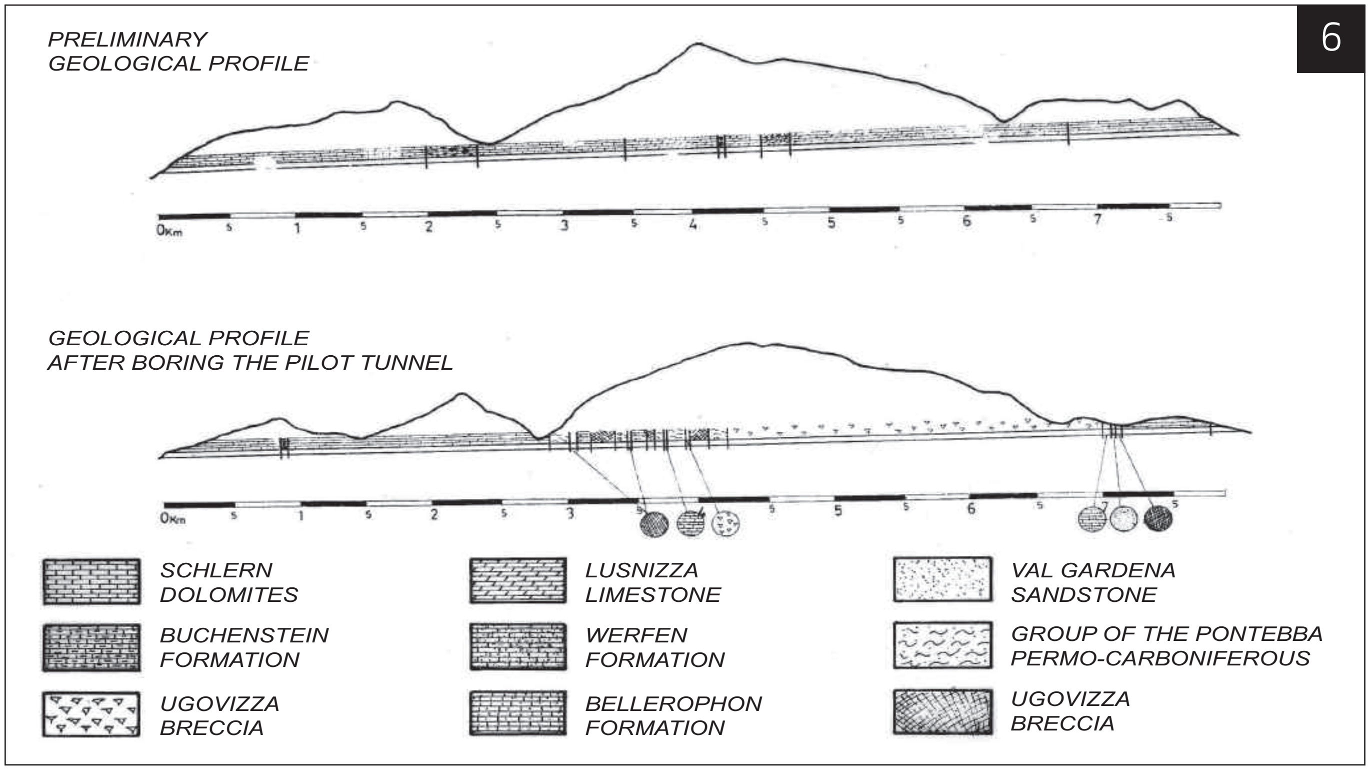
Provided the tunnel is sufficiently long (at least a few kilometres), then the initial expense incurred in driving the pilot tunnel is usually fully offset, not only by the direct savings achievable ("lighter" geological survey campaigns from the surface, lower advance costs for widening, lower ventilation costs during construction, etc.), but above all because all the operations for widening the tunnel to the full diameter can be programmed to fit the actual reality with consequent certainty of execution times and costs.

THE REFERENCE PROJECTS

While great experience has been acquired from the construction of many important railway and motorway tunnels using the "RS Method" of a pilot tunnel, reports on a few of the most significant projects are given below.

MALBORGHETTO TUNNEL
(UDINE-TARVISIO RAILWAY LINE) - 1985

The construction of the new "Pontebbana" railway line provided the chance, in the 1980s, to fully test the potential of the RS pilot tunnel method. This technology was used on a total of three tunnels located one after the other between Pontebba and Tarvisio: on the Malborghetto, S. Leopoldo and Camporosso tunnels (for a total of over 20 km of excavation).



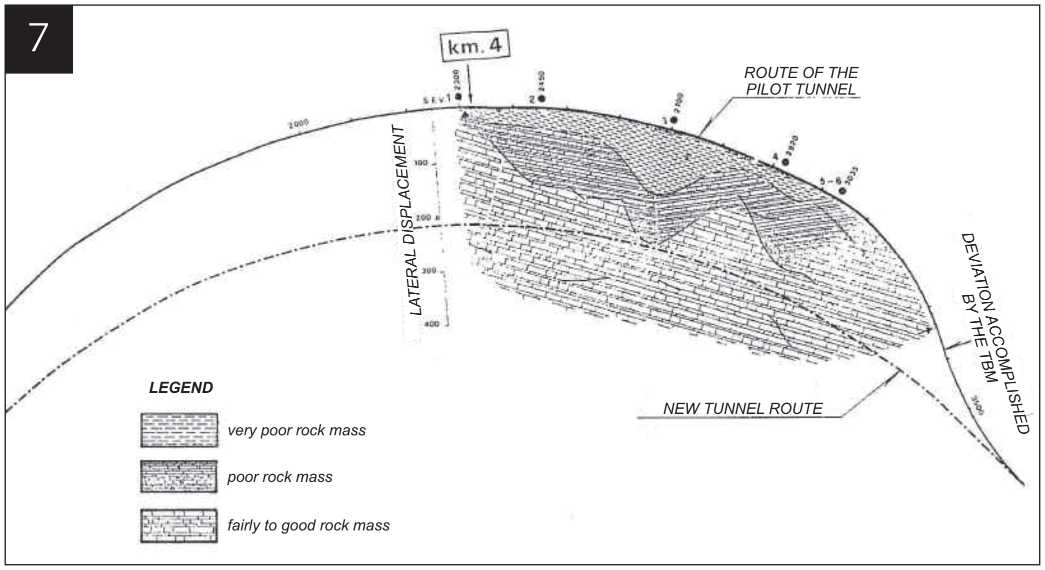
6. *A COMPARISON BETWEEN THE PREDICTED GEOLOGY ALONG THE TUNNEL ALIGNMENT BEFORE THE PILOT TUNNEL WAS DRIVEN AND THE REAL GEOLOGY FOUND AFTER DRIVING THE PILOT TUNNEL*
MALBORGHETTO TUNNEL
UDINE-TARVISIO STATE RAILWAY LINE

The works to drive the pilot tunnel for the Malborghetto Tunnel, 7,845 m. in length and 3.90 m. in diameter, began in November 1985 and finished 317 working days later in December 1986.
The overall average production rate (inclusive of the passages through fault zones) was approximately 25 m./day. Figure 6 shows a comparison between the predicted geology along the tunnel alignment before the pilot tunnel was driven and the real geology found after driving the tunnel. Substantial differences can be seen, two of which were particularly important:
1. the detection, thanks to the pilot tunnel, of a long intensely disturbed section from chainage 2700 to chainage 4200, which in addition to gas related difficulties caused face and tunnel wall stability problems to the extent that 255 m. of tunnel had to be driven with systematic lining of the tunnel walls using steel "liner-plates";
2. passing through the "Breccia di Ugovizza" formation from chainage 4200 to chainage 6900, again by the pilot tunnel, which had not been anticipated at all in that section. A large number of rock bolts had to be installed to stabilise the tunnel walls.

In both cases, the subsequent excavation to widen the tunnel to its full diameter benefited considerably from suitable and appropriate ground improvement of the surrounding rock mass performed from inside the pilot tunnel.

S. LEOPOLDO TUNNEL
(UDINE-TARVISIO RAILWAY LINE) - 1986

This pilot tunnel, 3.75 m. in diameter and approximately 5,900 m. in length, driven for the S. Leopoldo Tunnel, constituted an extremely particular case, because driving it actually led to a radical change in the planned alignment.
In fact half way through the excavations (Figure 7), the pilot tunnel unexpectedly ran into rocky material which was so poor in quality that after approximately 1,000 m. of very difficult tunnel advance, the TBM took a sharp turn downwards in the hope of meeting more reliable geological formations, which were actually met. This was based on geological knowledge of the region and on the results of seismic reflection surveys carried out from the tunnel that had been excavated to that point.

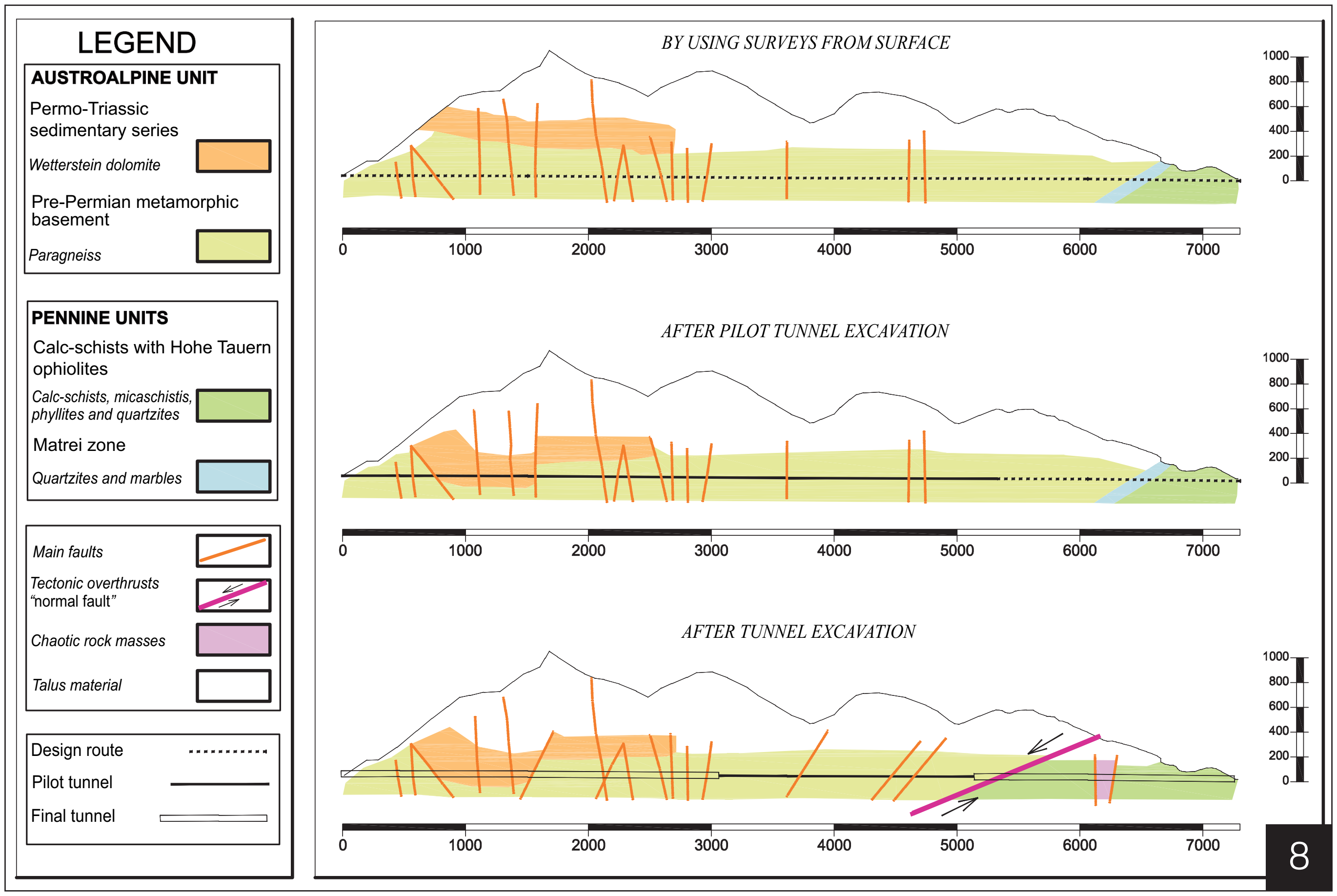


7. *A PILOT TUNNEL 3.75 M. IN DIAMETER AND APPROXIMATELY 5,900 M. IN LENGTH DRIVEN FOR THE S. LEOPOLDO TUNNEL RESULTED IN A RADICAL CHANGE IN THE ALIGNMENT*
UDINE-TARVISIO STATE RAILWAY LINE

It is quite clear here too, that the use of a pilot tunnel was of great use, because it allowed a change to the alignment to be made through rock masses which were very much more reliable for the safety of site personnel and decidedly better in terms of construction times and costs.

FLERES TUNNEL
(VERONA-BRENNERO RAILWAY LINE) - 1987

The works to drive the Fleres pilot tunnel, 3.80 m. in diameter, began from the south portal in January 1987.
Knowledge of the geology of the zone, the object of numerous surveys, which were not always in agreement, contained various gaps, which it had not been possible to fill by using conventional survey methods from the surface, because of the huge overburdens along the tunnel alignment and the inaccessibility of the places.
After the first 1,000 metres of advance at an average rate of 30 m./day through the paragneiss of the Austroalpine Unit, the pilot tunnel passed through



8. *COMPARISON BETWEEN FORECAST AND ACTUAL GEOLOGY, DISCOVERED THANKS TO A PILOT TUNNEL*
FLERES TUNNEL
VERONA-BRENNERO STATE RAILWAY LINE

the intensely fractured dolostone of the Wettestein with large inflows of water, to then return to the paragneiss, which, however, possessed very much poorer geomechanical properties than the previous paragneiss, due to a higher mica content. Tunnel advance became extremely difficult and production rates fell below 15 m./day.
At chainage 4500 it became necessary to reinforce the ground in the core ahead of the face, after retracting the TBM to prevent it from becoming jammed, while convergence of the tunnel walls reached exceptionally high values of over 35 cm., making it necessary to fully line the cavity with steel. At chainage 5248, tunnel advance was halted, because it had become extremely dangerous.
Figure 8 shows a comparison between the predicted geology along the tunnel alignment before the pilot tunnel was driven and the real geology found after driving it.
Here too the differences are considerable, sufficient to fully justify the costs incurred to drive the pilot tunnel. Notwithstanding the problems brought to light by the pilot tunnel, it was nevertheless decided to maintain the tunnel alignment already defined, because a study of alternative routes, closer to the valley walls, performed from within the pilot tunnel, revealed a danger of intersecting a zone of recurring subparallel faults in the Isarco Valley, and consequently encountering even greater difficulties.

RUPE TUNNEL

(STATE ROAD 43 TO FAI DELLA PAGANELLA BETWEEN NORTH TRENT AND ROCCHETTA) - 2002

The huge overburdens and the inaccessibility of the places made it impractical to carry out conventional geological surveys for this tunnel too. The geological studies performed on the basis of outcroppings found during the surface surveys led to the hypoth-

9. GEOLOGICAL PROFILE ON COMPLETION OF THE PILOT TUNNEL FOR THE "RUPE" TUNNEL

STATE ROAD 43 TO FAI DELLA PAGANELLA

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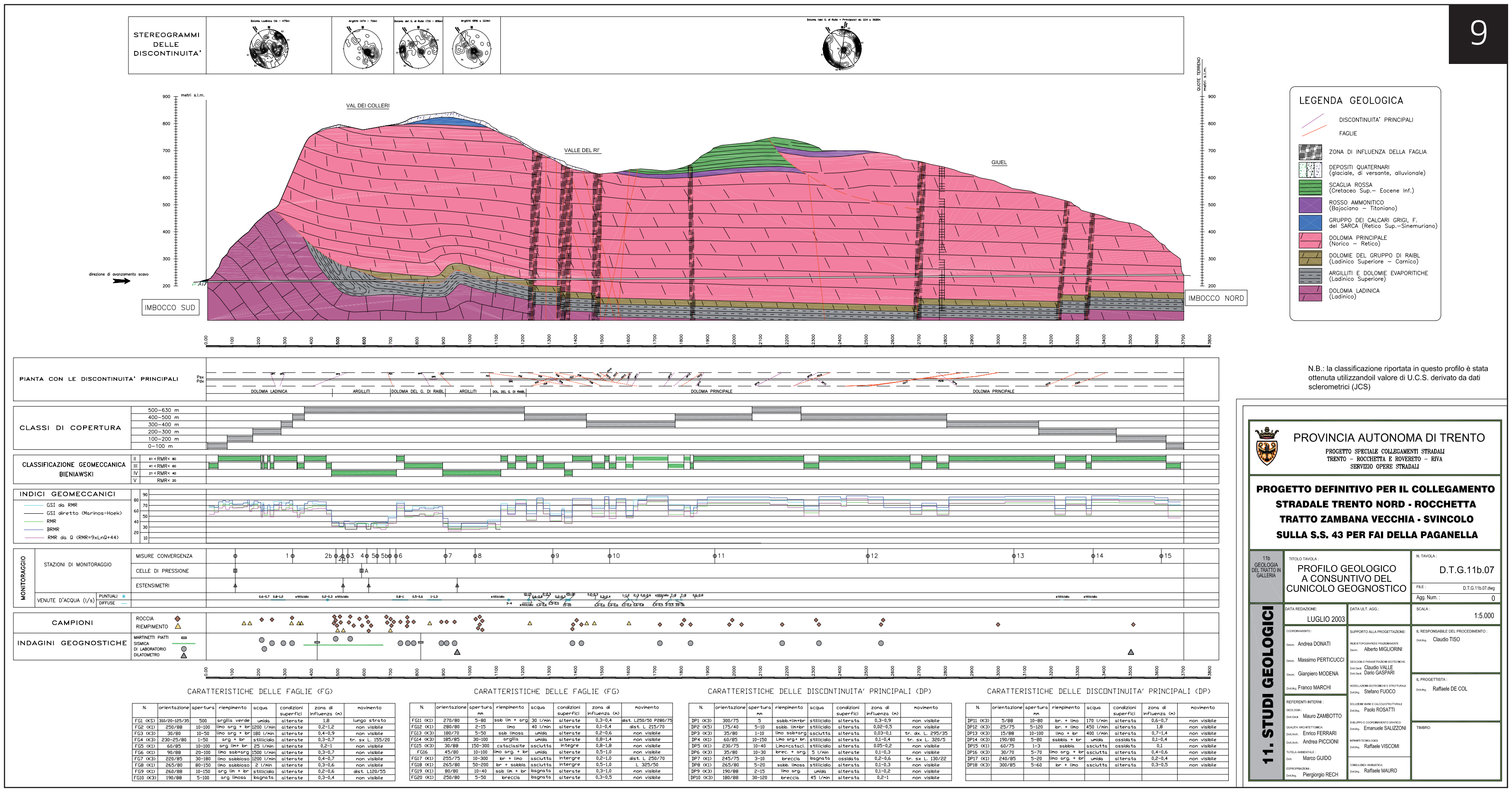


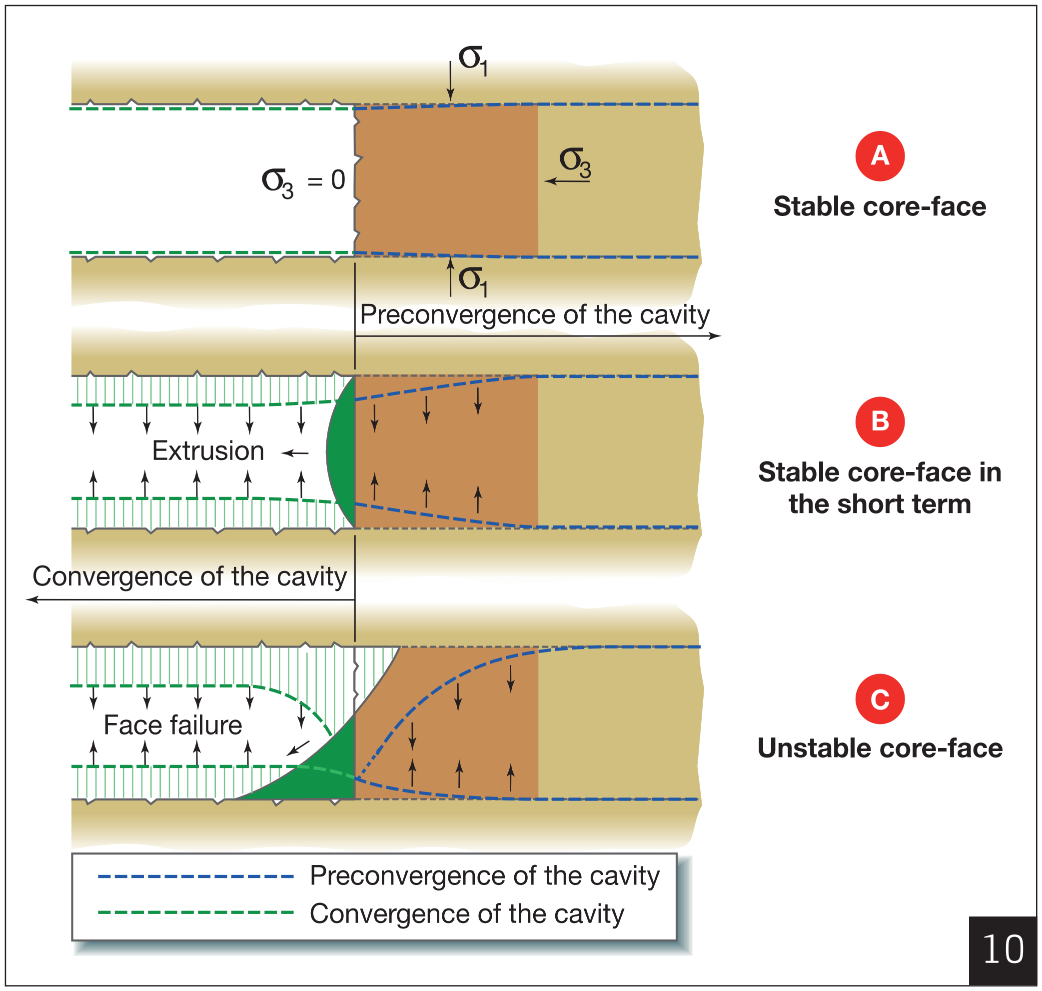
esis of an underground route which passed entirely through the principal dolomite rock. However, the available information seemed insufficient to formulate an appropriately reliable design.

As a consequence it was considered advisable to drive a pilot tunnel through all 3,688 m., of the planned underground alignment. After approximately 480 m. the pilot tunnel, on which work had begun from the south portal at the end of 2001, ran into a layer of argillites which continued until chainage 1200.

Here too, the pilot tunnel allowed the discovery in good time of the otherwise unpredictable reality of geology which was in effect more difficult than that assumed before it was driven. In fact no outcroppings of the argillite layer existed on the surface (Figure 9).

10. *DEFINTIONS OF BEHAVIOUR CATEGORIES*

WITH REFERENCE TO THE CORE-FACE SEEN AS A STABILISATION INSTRUMENT



FINAL CONSIDERATIONS

If correctly used following a feasibility study for the project, pilot tunnel technology, integrated with the "RS Method", can be used to achieve a series of undeniable advantages:

- the opportunity to conduct a "lighter" survey stage, with the sole purpose of driving a pilot tunnel, which is therefore less costly;
- the opportunity to formulate a final working drawing design with a degree of detail, thoroughness and certainty that is definitely much higher than that obtainable from any other type of survey;
- the opportunity to draft specifications and a tender contract which fits the project to be completed perfectly;
- finally, it gives the opportunity on the one hand for participants in contract competitions to make accurate and documented bids and on the other hand for clients to assess bids properly and above all to subsequently manage the works almost completely free from any unexpected events in terms of times and costs.

The decision of whether to drive a pilot tunnel or not must inevitably be the result of a technical and financial analysis which compares different methods of tunnel construction, with account taken of all the most important findings of the feasibility study.

From a purely design viewpoint, a pilot tunnel should only be excluded if its excavation would cause an appreciable decrease in the quality of the geomechanical characteristics of the rock mass to be tunnelled, thereby making it more difficult to drive the final tunnel.

The reference here is above all to stress-strain conditions which the ADECO-RS approach classifies in the behaviour category C (Figure 10) and under which it is imperative to proceed with a rigid advance core, a task that is clearly difficult to achieve once this has been partially drilled out by a pilot tunnel.

Unless it is not planned to use it as a supplementary service or safety tunnel or in some other way as part of the design of a project, employment of

a pilot tunnel should be excluded from a construction viewpoint when it cannot be driven using a TBM and also when it seems probable that frequent use of linings will be needed in the time elapsing between construction of the pilot tunnel and widening excavation.
This is not only because of the costs of placing and then removing linings during widening operations, but also because of greater construction site organisation difficulties and the longer times required.
From a financial viewpoint, a pilot tunnel may be attractive because it could be advantageous for a client to accept a higher absolute investment but with a more favourable spending curve and that is closer to the period when a tunnel will be used, with consequent lower financial charges.

MONTE OLIMPINO TUNNEL 2
MILANO-CHIASSO STATE RAILWAY LINE

TABLE 1. *PILOT TUNNEL DRIVEN WITH A TBM*

DRIVEN IN ITALY SINCE 1983

Owner	Tunnel	Length [m]	Ø [m]	Date	Grounds	Average advance rates [m/g]	Remarks
ANAS	Tarvisio (UD)	2x2,300	3.60	1982 - 1983	Dolomites	25	
ANAS	S. Martino (LC)	3,700	3.60	1984			vibrations
ANAS	Forca di Cerro (PG)	4,030	3.50	1984 ?			
ANAS	Forche Canapine (PG)	4,440	3.60	01/1984 - 07/1984	Limestones, Clayey intercalations	30	
F.S.	M. Olimpino 2 (CO)	4,500	3.60	1984			vibrations
F.S.	Sciliar (BZ)	13,200	3.50	12/1984 - 04/1986	Ignimbrites, Tuffs, Conglomerates, Phyllites	33	
F.S.	Domegliara-Dolcé (VR)	4,075	3.95	1984 ?			
F.S.	Malborghetto (UD)	7,845	3.90	11/1985 - 12/1986	Limestones, Breccias	25	gas
F.S.	S. Leopoldo (UD)	5,900	3.75	1986	Calcareous dolomites		
F.S.	Camporosso (UD)	7,000	3.75	1986 ?	Calcari, Marne, Argilloscisti, Arenarie, Brecce		
F.S.	Zuc del Bor (UD)	3,800	3.50	1986		20	
F.S.	Caponero (IM)	3,570	3.50		Marls, Marly limestones, Sandstones	25 ÷ 30	vibrations
F.S.	Fleres (BZ)	5,248	3.80	1987	Paragneiss, Dolomites	15 - 30	
F.S.	Ceraino (VR)	4,000		1987 ?	Limestones		karst
F.S.	Monte Leila (UD)	3,200	3.75				
F.S.	Peloritani (ME)	2,700	4.50	1988 ?	Gneiss, Paragneiss		gas
F.S.	Bellosguardo (FI)	3,500	3.90	1987 - 1988	M. Modino sandstones	40	
ANAS	Montezemolo (CN)	1,730	3.50				
ANAS	Fugona (SV)	1,900	4.00	1989 ?	Gneissic schists, Phyllites, Sericite schists		
ANAS	Arvier (AO)	2x2,360	3.90	02/1989 - 05/1989	Micascisti, Gneiss	29	
ANAS	Leverogne (AO)	2x1,630	3.90	07/1989 - 12/1989	Calc-schists, Gneiss, Micaschists	24	vibrations
ANAS	Villeneuve (AO)	2,750 570 2,200	3.90 4.94 3.75	07/1989 - 04/1990	Calc-schists and Micaschists with lens of cellular dolomites	21.3 13.60 25	
ANAS	Avise (AO)	1,285+2,638	4.50	04/1990 - 06/1991	Gneiss, Micaschists	20.5	
ANAS	Cantarana (CN)	2,874		1990	Carbonatic rocks		
Prov. BZ	Depuratore Brunico (BZ)	1,248	3.90	1992	Quartz phyllites		
ANAS	Lenna (BG)	2,100	3.90	1994 ?	Limestones, Dolomitic limestone, Calcareous dolomites		
ANAS	Frasnadello (BG)	1,708	3.90	01/1994-06/1994	Dolomites, Argillites	15	
ANAS	Antea (BG)	660	3.90	01/1994-03-1994	Dolomites	20	
ANAS	Prè Saint Didier (AO)	2,145	3.90	1994-1996	Calc- schists, Arenaceous schists, Sandstones	16.6	
ANAS	Camionabile BO-FI Galleria di base	2x2,000 2x500	3.90	1998 - 1999	Sandstones-pelites alternation, Scaly argillites	30	gas
Prov. TN	Rupe (TN)	3,688	5,20	2002 - 2003	Dolomites, Argillites	16	

Today, with the availability of extremely effective and flexible tunnelling technologies such as horizontal jet-grouting and reinforcement of the advance-core and/or the ground around the cavity using fibre glass reinforcement, engineers are very much better equipped than previously to deal with geological dangers. Nevertheless, a pilot tunnel integrated with the "RS Method" is definitely an attractive technology and is often indispensible for long and deep tunnels, because it is the only survey, design and construction instrument that allows us to drive them without taking unacceptable risks.
This is fully demonstrated by the results achieved in the construction of over 115 km. of tunnels driven in Italy using this technology.
Table 1 contains a list of the most important projects that have been implemented by Rocksoil using the "RS Method" of the pilot tunnel.

ONE OF THE TBMS USED TO DRIVE PILOT TUNNELS IN CARNIA

BIBLIOGRAPHY

LUNARDI P., 2008. Design and construction of tunnels - Analysis of Controlled Deformation in Rock and Soils (ADECO-RS), Ed. SPRINGER, 576 pagine, marzo 2008

LUNARDI P., 2007. Progetto e costruzione di gallerie - Analisi delle deformazioni controllate nelle rocce e nei suoli (ADECO-RS), Ed. HOEPLI, 575 pagine, agosto 2007

COSTANZO G., 2001, "SS 470 Val Brembana - Construction of San Pellegrino alternative route", AITES-ITA 2001 World Tunnel Congress, Milano, 10-13 giugno 2001

BARLA G., 2000, "Lessons lernt from the excavation of a large diameter TBM tunnel in complex hydrogeological conditions, Atti del Convegno Internazionale "GeoEng 2000", Melbourne, 19-24 novembre 2000, Vol.1: Invited papers pp. 933-995

LUNARDI P., 2000, "Problematiche affrontate nello scavo delle gallerie Bellosguardo e San Vito", Quarry and Construction, ottobre 2000

LUNARDI P., 2000, "Design & constructing tunnels – ADECO-RS approach", Tunnels & Tunnelling International, supplemento speciale al numero di Maggio 2000

DE FRANCESCHI E., CASSANI G., 1999, "Problematiche incontrate durante lo scavo della galleria Fleres", Convegno Europeo su: "Gallerie profonde: progetto, realizzazione ed esercizio", Verona, 4-5 maggio 1999

PIGNATELLI M., 1999, "Autostrada Monte Bianco-Aosta. Realizzazione di cunicoli esplorativi con frese tipo TBM. Potenzialità del mezzo meccanico", Gallerie e grandi opere sotterranee, n. 57, aprile 1999

PIGNATELLI M., 1999, "Autostrada Monte Bianco-Aosta. Realizzazione di cunicoli esplorativi con frese tipo TBM. Potenzialità del mezzo meccanico", Gallerie e grandi opere sotterranee, n. 57, aprile 1999

LUNARDI P., POZZI V., 1996, "Aspetti progettuali ed esecutivi della realizzazione della galleria Avise", Le Strade, 3/96

SCESI L., PAPINI M., 1997, "Da cunicolo pilota ad allargo: studio dell'effetto scala", Gallerie e grandi opere sotterranee, n. 52, luglio 1997

CICOLANI A.M., 1993, "La metodologia del preforo", 1° Corso Interdisciplinare di Geoingegneria S.I.GI "Progetto e costruzione integrata nell'ambito della modellazione antropica superficiale e profonda: le opere in sotterraneo", Parma, febbraio-maggio 1993

CAMPANA M., 1993, "Il foro pilota quale strumento d'acquisizione di dati geologici, geoidrologici, geostrutturali e geomeccanici", 1° Corso Interdisciplinare di Geoingegneria S.I.GI "Progetto e costruzione integrata nell'ambito della modellazione antropica superficiale e profonda: le opere in sotterraneo", Parma, febbraio-maggio 1993

CAMPANA M., LUNARDI P., PAPINI M., 1993, "Dealing with unexpected geological conditions in underground construction: the pilot tunnel technique", Atti del 6th European Forum on "Cost Engineering" - Università Bocconi, Milano, 13-14 maggio 1993, Vol. 1

CAMPANA M., LUNARDI P., PAPINI M., 1993, "Tipologie dei rilasci e delle deformazioni in "foro pilota" in relazione alle condizioni geostrutturali dell'ammasso roccioso", Le Strade, marzo-aprile 1993

MOLINARI L., PIGNATELLI M., PIGORINI B., 1991, "L'impiego del cunicolo pilota nel consolidamento delle rocce", Atti del Convegno Internazionale su: "Il consolidamento del suolo e delle rocce nelle realizzazioni in sotterraneo", Milano, 19-20 marzo 1991

CASALE R., GAMBA A., LUNARDI P., PIZZAROTTI E.M., 1991, "La galleria "Malborghetto" della linea F.S. Udine-Tarvisio. Esempio di preconsolidamento dell'ammasso roccioso da cunicolo pilota", Atti del Convegno Internazionale su: "Il consolidamento del suolo e delle rocce nelle realizzazioni in sotterraneo", Milano, 19-20 marzo 1991

CEPPI G., DE PAOLI B., LOTTI A., PEDEMONTE S., 1986, "La galleria di Monte Olimpino 2. Scavo con tecniche speciali", Atti del Convegno Internazionale su "Grandi Opere Sotterranee", Firenze 8-11 giugno 1986

OLIVA L., BONASSO R., BEOMONTE M., 1986, "La galleria di valico di Forca Canapine", Atti del Convegno Internazionale su "Grandi Opere Sotterranee", Firenze 8-11 giugno 1986

CASALE R., COMIN C., 1986, "Evoluzione di alcuni aspetti progettuali relativi ad infrastrutture ferroviarie sviluppatisi prevalentemente in sotterraneo", Atti del Convegno Internazionale su "Grandi Opere Sotterranee", Firenze 8-11 giugno 1986

CUSCANNA R., D'AMATO A., FARIELLO R., PIGORINI B., SERRA A., 1986, "L'impiego del tunnel pilota a mezzo di fresa ad attacco integrale per la realizzazione della galleria autostradale di Tarvisio", Atti del Convegno Internazionale su "Grandi Opere Sotterranee", Firenze 8-11 giugno 1986

MONTANARI L.F., COMIN C., NICOLINI A., PIZZAROTTI E.M., 1986, "Il cunicolo pilota realizzato con fresa: i suoi riflessi nella progettazione ed esecuzione di gallerie ferroviarie", Atti del Convegno Internazionale su "Grandi Opere Sotterranee", Firenze 8-11 giugno 1986

IANNIELLO C., CEPPI G., LUNARDI P., SIOTTO C., 1986, "Vibrazioni indotte dallo scavo della galleria Capo Verde a Sanremo", Atti del Convegno Internazionale su "Grandi Opere Sotterranee", Firenze 8-11 giugno 1986

IANNIELLO C., LEONARDI L., LUNARDI P., PELIZZA S., ZANALDA M., 1986, "Problemi tecnici di costruzione in sotterraneo del nuovo tratto di linea ferroviaria in liguria fra san Lorenzo al mare e ospedaletti", Atti del Convegno Internazionale su "Grandi Opere Sotterranee", Firenze 8-11 giugno 1986

LUNARDI P., 1986, "Lo scavo delle gallerie mediante cunicolo pilota", Politecnico di Torino, Primo ciclo di conferenze di meccanica e ingegneria delle rocce - Torino, 25-26 novembre 1986

BECCHI I., CALISTRI E., LUNARDI P., ORSI S., 1986, "Nuovi orientamenti nella progettazione delle gallerie mediante foro pilota", Atti del Convegno Internazionale su "Grandi Opere Sotterranee", Firenze 8-11 giugno 1986

1988 REINFORCEMENT OF THE ADVANCE-CORE

The Rocksoil technology which revolutionised tunnelling

REINFORCEMENT OF THE CORE-FACE WITH FIBRE GLASS STRUCTURES

SAN VITALE TUNNEL
CASERTA-FOGGIA STATE RAILWAY LINE

∅ = 12.70 M.
GROUND: SCALY CLAY
OVERBURDEN: 130 M .

Reinforcement of the advance core of a tunnel using fibre glass structures, together with vertically and horizontally grouted shells, constitutes the most successful and most widely used of the technologies conceived of by Prof. Ing. Pietro Lunardi and developed by Rocksoil during the course of its first 30 years. It can be said today that, in industrialised countries at least, no tunnel is driven through ground under difficult stress-strain conditions without the use of that technology.
It consists of dry drilling a series of holes into the face sub parallel to the axis of the tunnel evenly distributed over the area in question. Special fibre glass reinforcement is then inserted into the holes

REINFORCEMENT OF THE CORE-FACE WITH FIBRE GLASS REINFORCEMENT

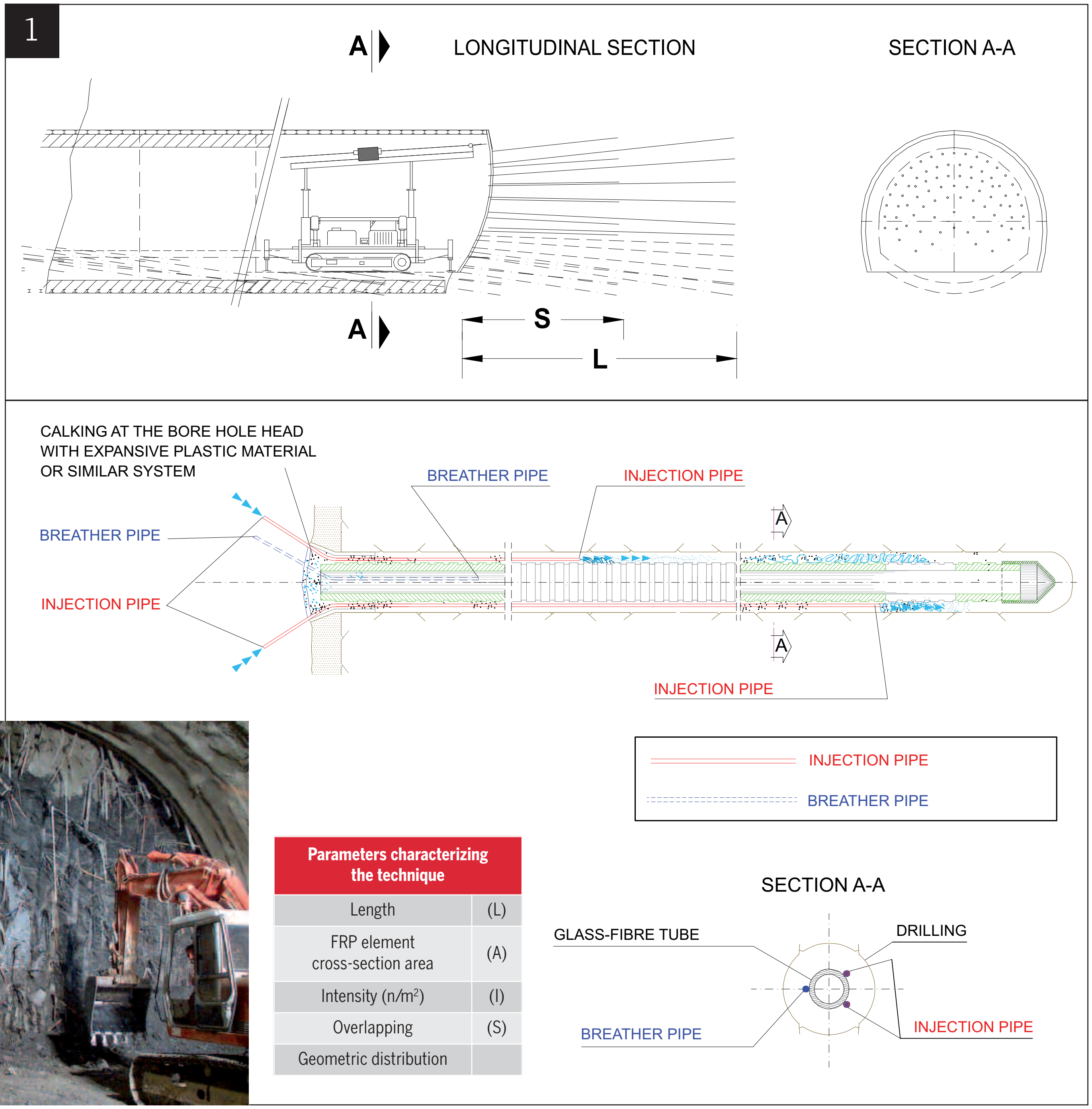
SAN VITALE TUNNEL
CASERTA-FOGGIA STATE RAILWAY LINE

∅ = 12.70 M.
GROUND: SCALY CLAY
OVERBURDEN: 130 M.

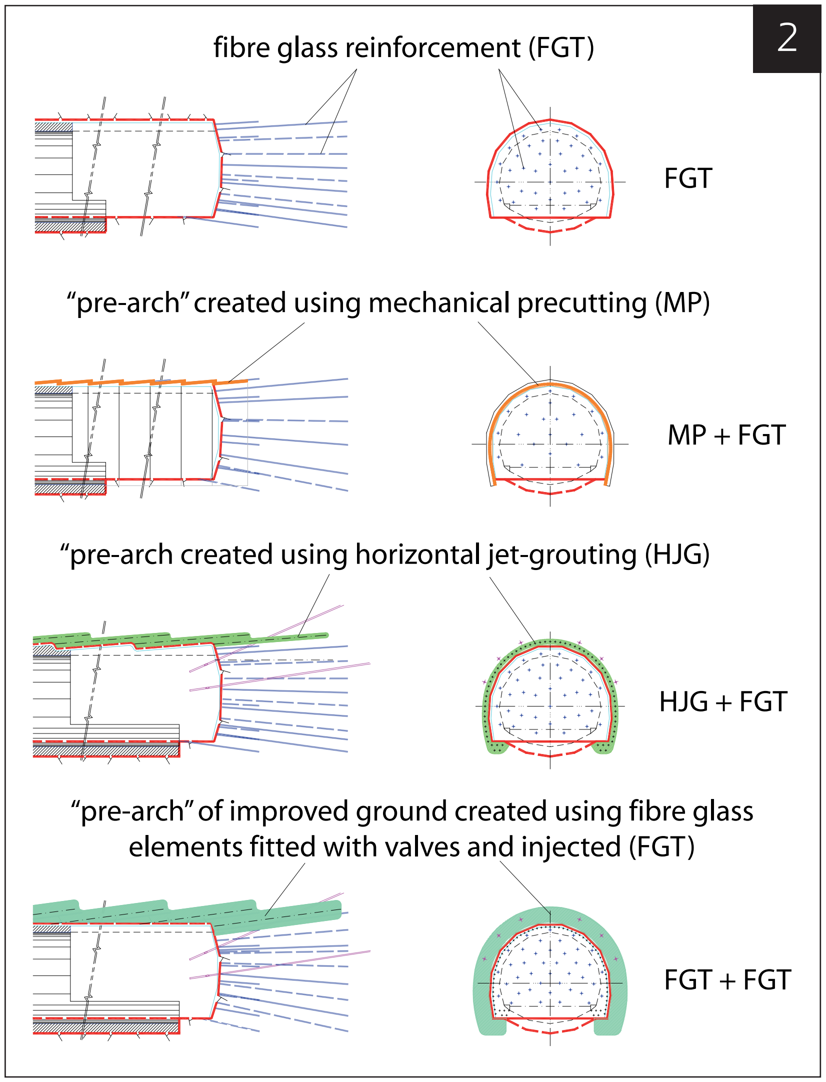
and this is immediately injected with cement mortar (Figure 1). When the remaining length of the reinforcement inserted in the face-core after tunnel advance is no longer sufficient to guarantee pre-confinement of the cavity (a circumstance that can be recognised immediately by careful reading of extrusion measurements) another series is placed.

The length, frequency, overlap, cross section and geometrical distribution of the reinforcement are the parameters that characterise the reinforcement action which can be used in cohesive and semi cohesive soils and, with a few measures taken to ensure the integrity of the drill holes, even in soils with very poor cohesion.

1. REINFORCEMENT OF THE CORE-FACE WITH FIBRE GLASS REINFORCEMENT (FGT)



Parameters characterizing the technique	
Length	(L)
FRP element cross-section area	(A)
Intensity (n/m^2)	(I)
Overlapping	(S)
Geometric distribution	



CAULKING THE MOUTHS OF THE HOLES

SAN VITALE TUNNEL
CASERTA-FOGGIA STATE RAILWAY LINE
∅ = 12.70 M.
GROUND: SCALY CLAY
OVERBURDEN: 130 M.

***2.** TYPES OF REINFORCEMENT OF THE CORE-FACE USING FIBRE GLASS REINFORCEMENT*

Within the framework of the ADECO-RS approach, the technology forms part of cavity preconfinement operations and it can be performed in combination with other ground improvement work ahead of the face and classified into four types, to be selected on the basis of the type of ground and the stress-strain conditions to be tackled (Figure 2):

1. (FGT): simple reinforcement of the advance-core using fibre glass reinforcement (an indirect conservative technique);

2. (MP+FGT): fibre glass reinforcement of the advance-core and protection of the core at the same time by means of pre-vaults of shotcrete created using mechanical precutting (a mixed conservative technique);

3. (HJG+FGT): fibre glass reinforcement of the face-core and protection of the core at the same time by means of pre-vaults of improved ground created using horizontal jet-grouting (a mixed conservative technique);

4. (FGT+FGT): fibre glass reinforcement of the advance-core and protection of the core at the same time by means of pre-vaults of improved ground around the tunnel using glass fibre elements, fitted with valves, placed in advance and injected with grout (a mixed conservative technique).

Each of these types can be associated with the relative project (see Table 1) in which it was either used experimentally for the first time or developed in a particular way.

CONCEPTION AND FIRST EXPERIMENTAL APPLICATION

If reinforcement of the advance core using fibre glass elements is considered in the same way as any other rock-bolting operation to be performed occasionally along short sections of tunnel as a countermeasure against the fall in of ground at

the face, it is difficult to establish with certainty when this technology was used for the first time. However, if it is considered as a construction technology to be applied systematically under medium to extremely difficult stress-strain conditions to achieve full control of deformation (and of consequent surface settlement, when necessary), then it was applied for the first time experimentally in 1985 during the construction of a short water tunnel, 4 m. in diameter, for a floodway for the Citronia river at Salsomaggiore Terme (Italy).
However, the idea, as proposed by Prof. Ing. Pietro Lunardi, had started to take shape during the studies and research which led to the development of the ADECO-RS approach (which, as is known, is based on the analysis and control of the deformation response of the ground to excavation, performed by regulating the stiffness of the core-face). More specifically, resort was made, during the construction of tunnels through clay for the new Sibari-Cosenza railway line (1984), to rock-bolting in the excavation face to counter spalling phenomena which occurred unfailingly in it when work halted at weekends. This operation consisted simply of inserting a certain number of steel rock bolts, with a diameter of 24 mm. and 4 m. in length, into the face pushing them in using an excavator bucket, on which a small cross had been fitted on one of the ends as a handle. This measure, which was completed with a thin layer of shotcrete, turned out to be very effective and when work started again on Monday, it was sufficient to pull-out the bolts gripping them by the cross to resume excavation without any problems.
We were convinced of the importance of the stiffness of the advance-core for controlling tunnel deformation phenomena. It was a conviction which had grown with positive experimental observations of the stress-strain behaviour of the tunnels already constructed with operations for the pure protection of the core (horizontal jet-grouting on the Campiolo tunnel and mechanical precutting on the Sibari-Cosenza railway line). We therefore started to study the possibility of increasing this stiffness artificially until it reached the desired value. To reinforce the advance-core with special

TABLE 1. *THE RELATIVE PROJECTS*

REINFORCEMENT OF THE CORE-FACE WITH FIBRE GLASS REINFORCEMENT

Relative project		Tunnel length	Type of ground	Max overburden	Ø	Characteristics of core-face improvement work					
Work	Tunnel	[m]		[m]	[m]	Type	Type of elements	Length [m]	Number	Density [n/m^2]	Material
Rome-Florence "Direttissima" railway line [1988]	Talleto	2700	Sandy silts	60	7	MP + FGT	Tubular	15	25	0,35	Glass-fibre
	Caprenne	2700	Sandy silts	60	7	MP + FGT	Tubular	15	25	0,35	Glass-fibre
	Poggio Orlandi	850	Sandy silts	60	13	FGT	Tubular	15	50	0,43	Glass-fibre
	Crepacuore	700	Sandy silts	50	13	FGT	Tubular	15	50	0,43	Glass-fibre
	Tasso	2000	Sandy silts	90	13	FGT	Tubular	15	60	0,51	Glass-fibre
	Terranova Le Ville	2600	Lacustrine deposits	90	13	FGT	Tubular	15	60	0,51	Glass-fibre
Caserta-Foggia railway line [1991]	S. Vitale	2500	Scaly clays	150	12,70	FGT + FGT	Tubular	18	49 + 50	0,41	Glass-fibre
Ancona-Bari railway line [1993]	Vasto	5000	Silty clays	135	12	HJG + FGT	Tubular	18	55	0,45	Glass-fibre
Rome Metro Line A [1997]	Baldo degli ubaldi	120	Clays and sandy silts	22	22	MP + FGT	Plate	25	47	0,37	Glass-fibre
TGV Méditerranée Marseille-Lyon railway line [1998]	Tartaiguille	900	Overconsolidated clays	110	15	FGT	Plate	24	90	0,5	Glass-fibre

systematic bolting, capable of guaranteeing significant increases in the strength and deformability of the cores treated, while not hindering demolition of them during excavation did in fact seem a goal worthwhile pursuing.

Having selected fibre glass as the most suitable material for this purpose from those examined, possible operational methods were studied for putting the technology into practice by testing their effectiveness on the basis of computer implementation of original calculation models (Figure 3).

The opportunity to carry out reliable experiments of the new insights in the field soon arose, as already mentioned, with the construction of a short water tunnel, 4 m. in diameter for the floodway of the Citronia river at Salsomaggiore Terme.

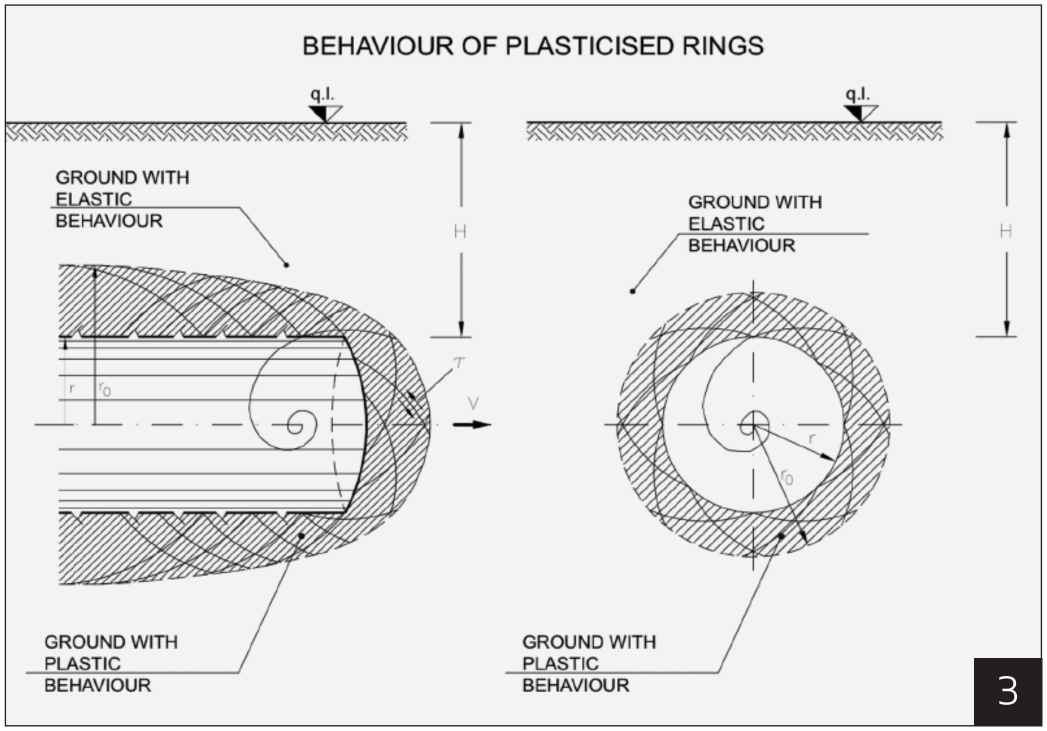
The contractor had been forced to suspend tunnel advance because the great instability of the ground, consisting of scaly clays, made it too dangerous to continue tunnel advance. Prof. Ing. Lunardi was brought in and proposed trying out the new technology, to be implemented by following the procedures that emerged from the calculation models just mentioned.

The procedures resulted in the rapid completion of the tunnel which confirmed the validity of the new technology, which was then ready for further development on larger projects.

The opportunity materialised soon afterwards during the construction of six tunnels between Florence and Arezzo for the new high speed railway line between Rome and Florence.

THE SECOND EXPERIMENT (1985)

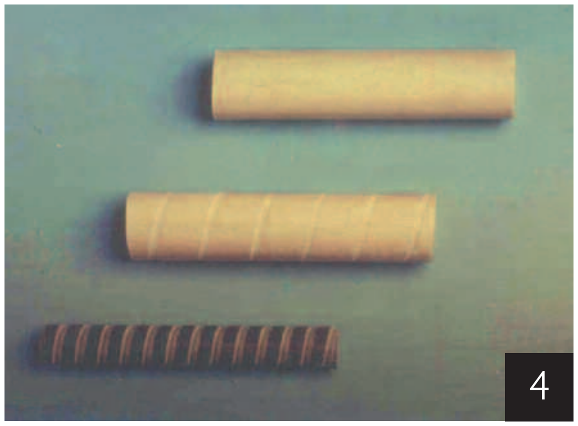
The project for the new high speed Florence-Rome railway line involved the construction of six tunnels (Talleto, Caprenne, Tasso, Terranova Le Ville, Crepacuore and Poggio Orlandi) on the section between Florence and Arezzo. However, during the works, the poor quality of the relative geological formations (sandy/clayey silts and lacustrian deposits, often under the water table) caused considerable problems, requiring work to stop and the redesign of the tunnels.



It was decided, thanks to the enterprise and farsightedness of State Railway Management and the contractors (Ferrocemento S.p.A. and Fondedile S.p.A.), to try out the new technology developed by Rocksoil S.p.A. which actually transformed the whole line under construction between Florence and Arezzo into one large experimental tunnel construction site.

3. *CALCULATION MODEL FOR THE DIMENSIONS OF THE OPERATIONS TO REINFORCE THE CORE-FACE*

DIFFERENT TYPES OF FIBRE GLASS REINFORCEMENT STORED ON SITE



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4. SAMPLES OF SMOOTH AND CORRUGATED FIBRE GLASS TUBES

5. SHEAR, TENSILE AND BURSTING STRENGTH TESTS

It is interesting to recall the main characteristics of these first works and the more significant tests, controls and measurements that were carried out or which started to be developed on that pioneering construction site. There was no choice but to select the fibre glass structural elements from among those available on the market, with account also taken of the transport factor. Those chosen were tubular in shape with an outer diameter of 60 mm., a thickness of 10 mm. and a length of 15 m. Both smooth elements and those with a corrugated outer surface to give better adherence for the cement grout were tried (Figure 4). Figure 5 summarises the results of the strength tests performed at the time, which were indispensible both for calculating the dimensions of the works (shear and tensile strength) and for establishing the operating parameters to use for the grout injections (bursting strength).

CHARACTERISTIC GEOMETRY AND PARAMETERS OF THE CORE–FACE REINFORCEMENT WORKS

The first reinforcement works were carried out using the following characteristic parameters (Figure 1):

- Length of each reinforcement step: L = 15 m.
- Resistent cross section of the fibre glass elements: ∅ = 60/40 mm.
- Intensity of the reinforcement:

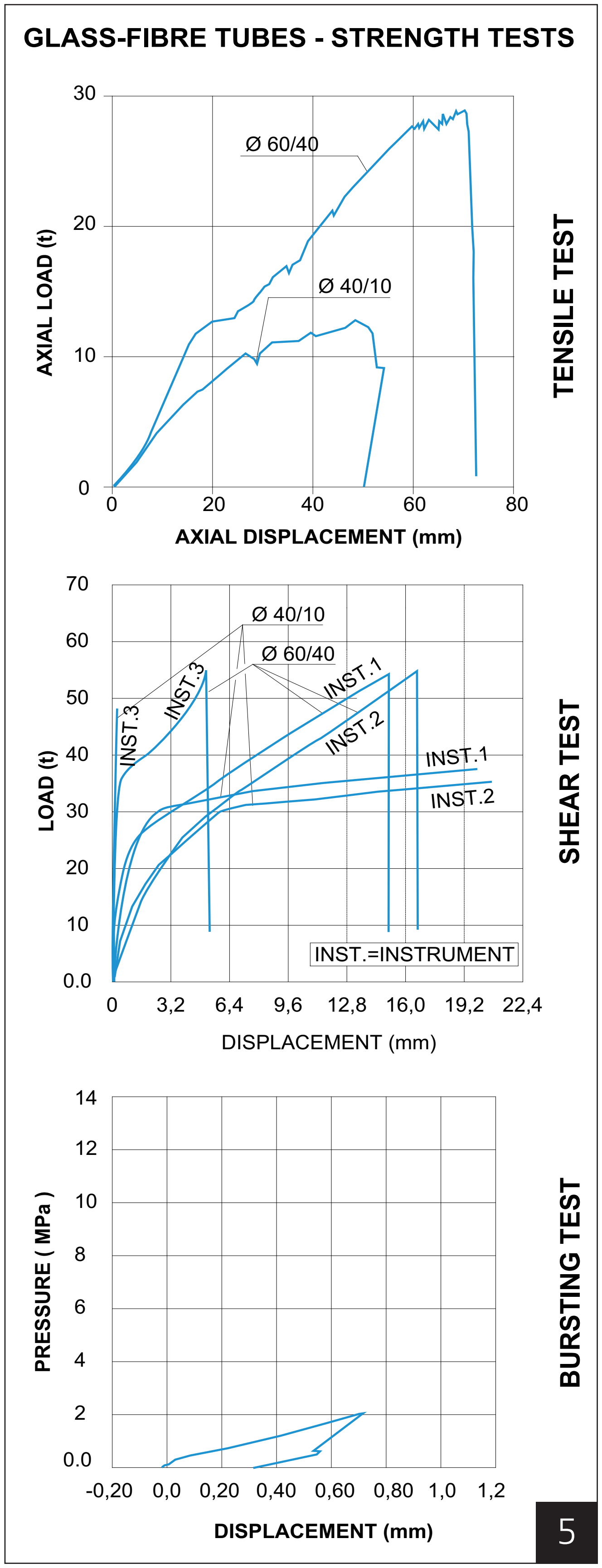
I = 0.35 - 0.51 elements/sq. m.

- Overlap between reinforced sections: S = 5 m.

Excavation was performed full-face and the tunnel invert and side walls were systematically placed at a maximum distance from the face of 1.5 times the diameter of the tunnel.

The face was systematically given a concave shape to favour the formation of a natural longitudinal arch effect.

Figure 6 illustrates a typical operating cycle for full-face tunnel advance following reinforcement of the advance core using fibre glass reinforcement.



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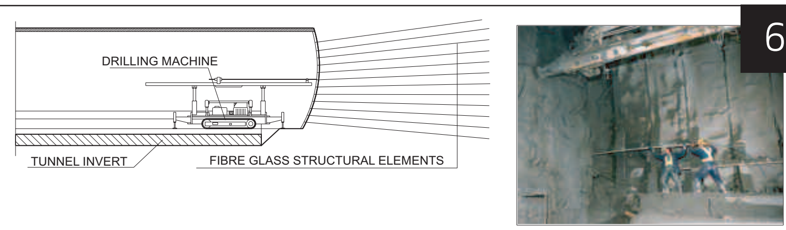
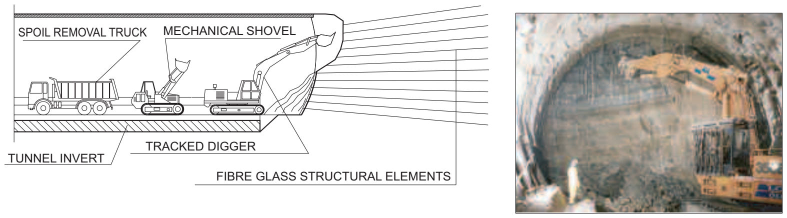
PHASE 1

Reinforcement of the core-face by means of fibre glass structural elements



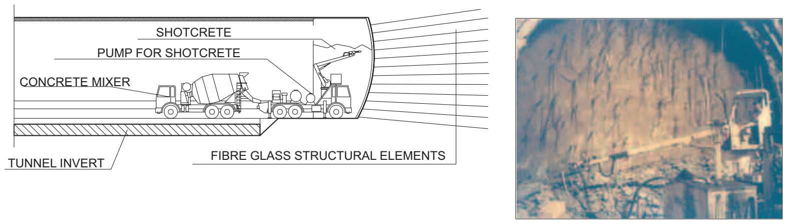
PHASE 2

Excavation step
(m. 0,70 ÷1,00 full face)



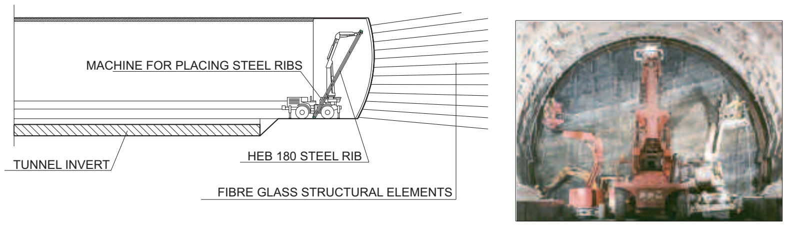
PHASE 3

Placing of shotcrete on the face and surface of the cavity to protect walls of the tunnel



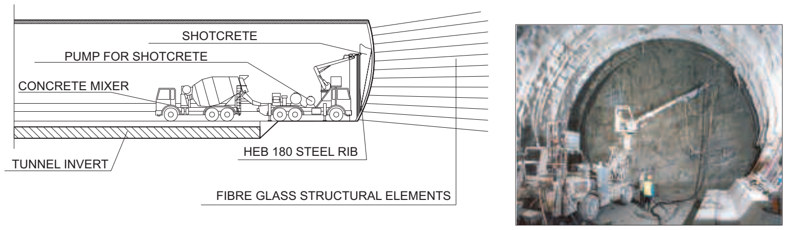
PHASE 4

Placing of the steel rib and connecting reinforcement in the section of tunnel excavated in phase 2



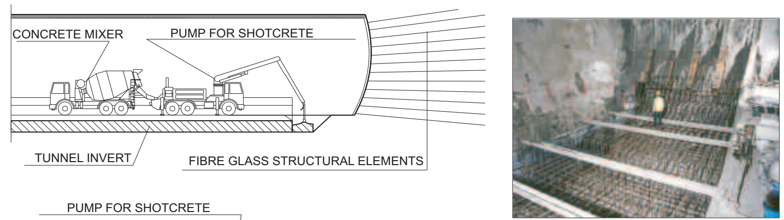
PHASE 5

Placing of the preliminary lining in fibre reinforced shotcrete



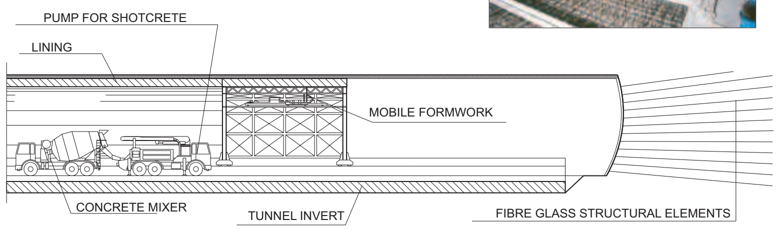
PHASE 6

Excavation and casting of side walls and tunnel invert close to the face

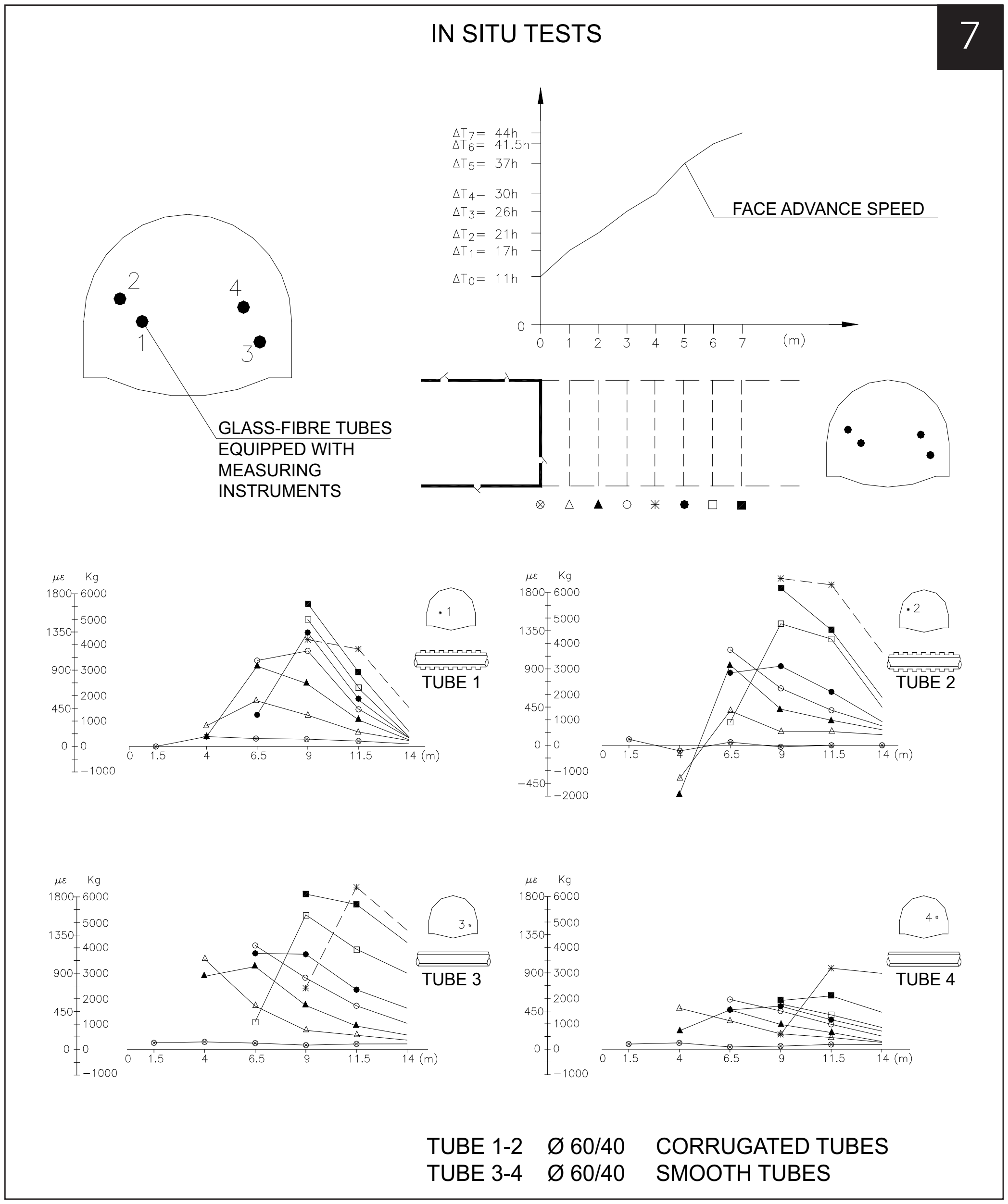


PHASE 7

Casting of the final lining at distance from the face of $\leq 4 \div 5$ Ø



6. *CYCLE OF OPERATIONS FOR TUNNEL ADVANCE FOLLOWING REINFORCEMENT OF THE CORE-FACE USING FIBRE GLASS REINFORCEMENT*



TESTS AND IN SITU MEASUREMENTS

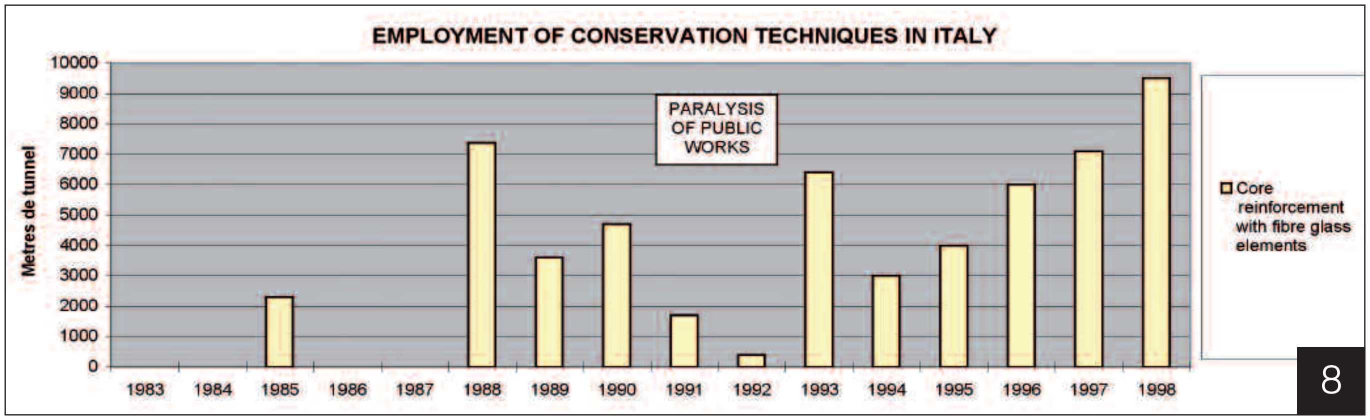
Numerous tests and in situ measurements were performed in order to study in detail, both the nature of the interaction between fibre glass elements and the surrounding ground (deformation and extraction tests, Figure 7) and the effect of the core-face reinforcement on the stress-strain response of the tunnel ahead of the face under different reinforcement conditions. Extrusion measurements were developed for this precise purpose and these have come into widespread use in tunnelling since then, alongside the more conventional convergence measurements. These measurements were made by inserting an incremental subsidence meter 15 m. long horizontally into the face with measurement bases fitted at one metre intervals. The results of the extrusion, preconvergence and convergence measurements taken significantly increased our theoretical knowledge of the stress-strain behaviour of a tunnel at the face and confirmed the effectiveness of the new technology to control deformation behaviour.

SPREAD OF THE TECHNOLOGY

The positive results achieved on the Rome-Florence line during the construction of more than 11 km. of tunnel under objectively difficult conditions, confirmed the complete reliability of the full face advance principles in the presence of a rigid core in soft ground and rapidly established the success of this technology of reinforcing the advance core using fibre glass and, more generally of preconfinement, conservative techniques (Figure 8).

7. DEFORMATION AND PULLING TESTS
POGGIO ORLANDI TUNNEL FLORENCE-ROME STATE RAILWAY TUNNEL

8. ESTABLISHMENT OF THE TECHNOLOGY IN ITALY



EVOLUTION OF THE TECHNOLOGY

Fibre glass reinforcement of the face-core technology has undergone significant development since it was first experimented, which has concerned the design instruments, the materials, the types of implementation and operational technologies. A brief description of the main stages of this evolution is given below by examining the relative projects (Table 1):

- the San Vitale Tunnel (Caserta-Foggia State Railway line) was driven in 1991 through the scaly clays formation where conventional methods (partial-face advance and preliminary stabilisation using ribs, radial bolts and shotcrete) had failed com-

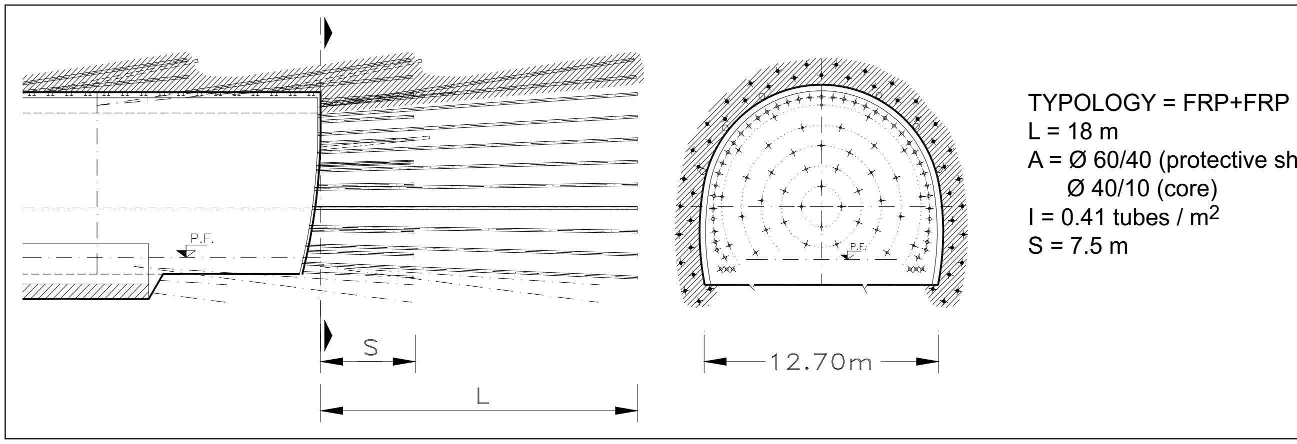
TARTAIGUILLE TUNNEL

TGV MEDITERRANÉE, MARSEILLES-LYON "G.V." LINE

∅ = 15.30 M.
GROUND: SWELLING CLAYS
OVERBURDEN: ~ 110 M.

***9.** KEY PARAMETERS OF REINFORCEMENT OPERATIONS*

***10.** ANALYSIS OF AN EXTRUSION TEST*



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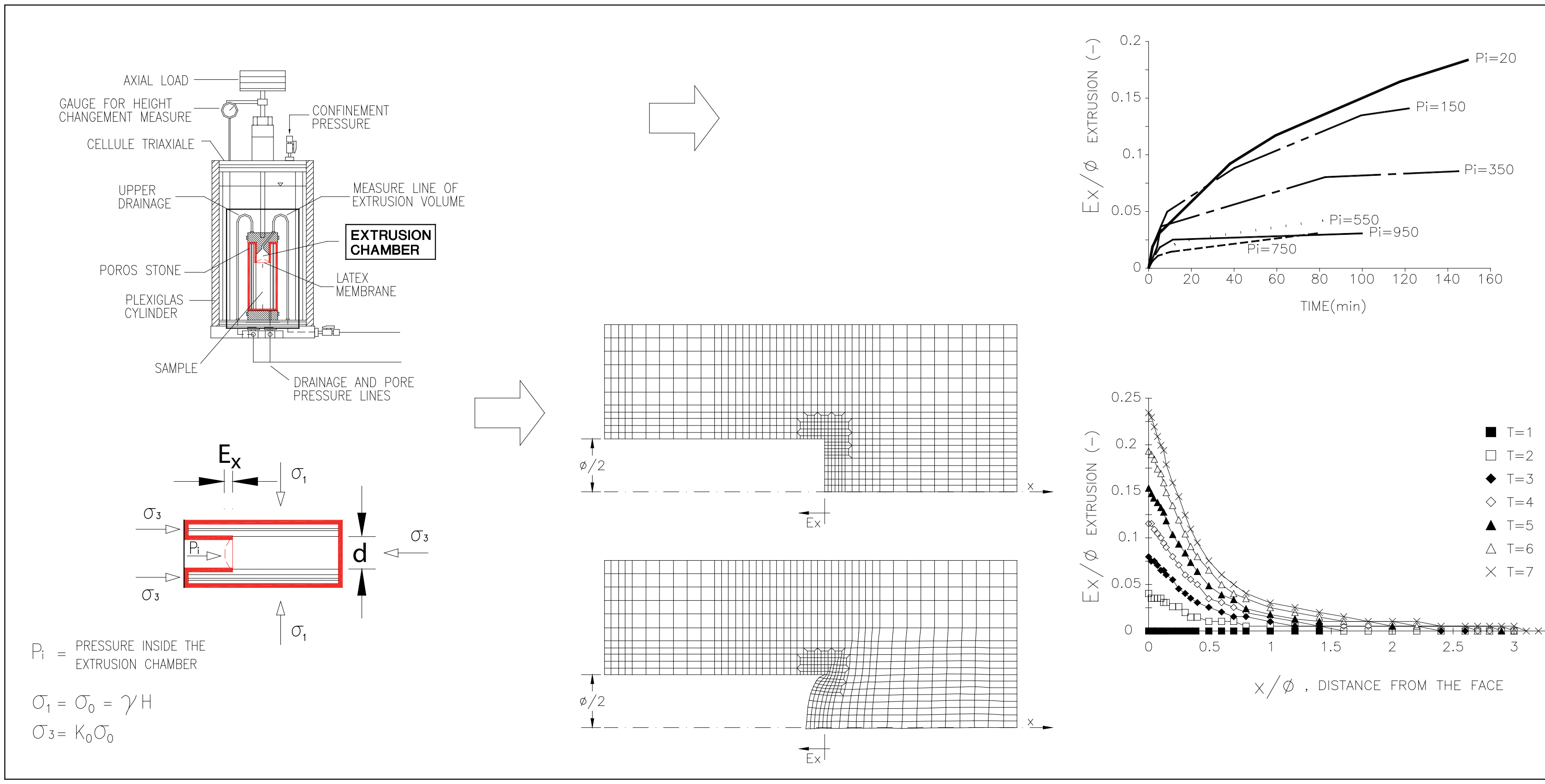
pletely, which resulted in the halt of tunnel advance. The application of advance principles with a rigid core (ADECO-RS) allowed the tunnel to be saved after advance had been abandoned for more than two years and it was completed with advance rates of around 50 m./month.
Figure 9 summarises the parameters for the ground improvement works.

The contractor's difficulties not only constituted a rigorous test and a chance to fully explore the new technology, but it was also an opportunity to develop new types of operating methods, new types of laboratory and in situ measurements and 3D calculation models capable of correctly investigating the effect of intervention in the core-face zone on the stress-strain behaviour of a tunnel and on the size



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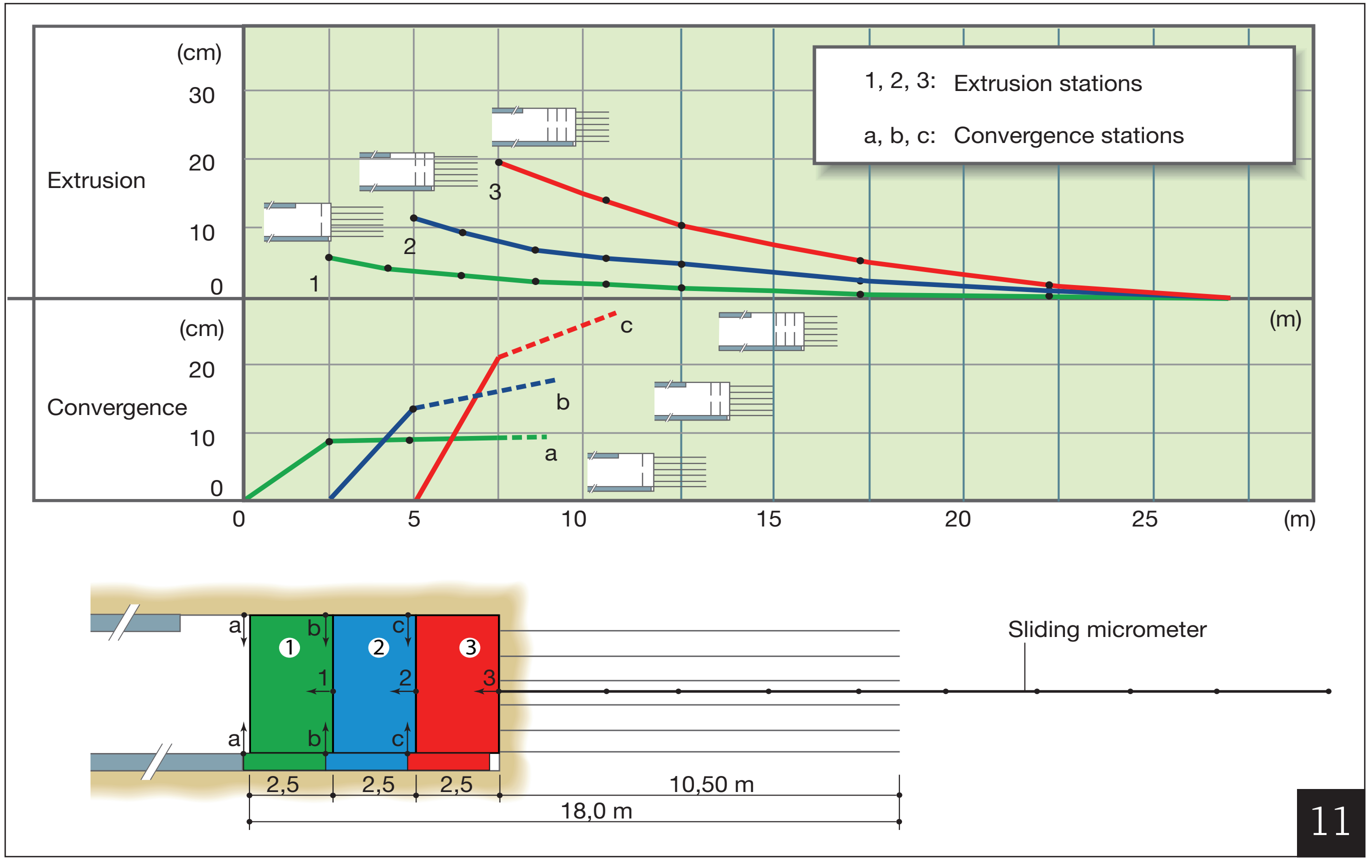

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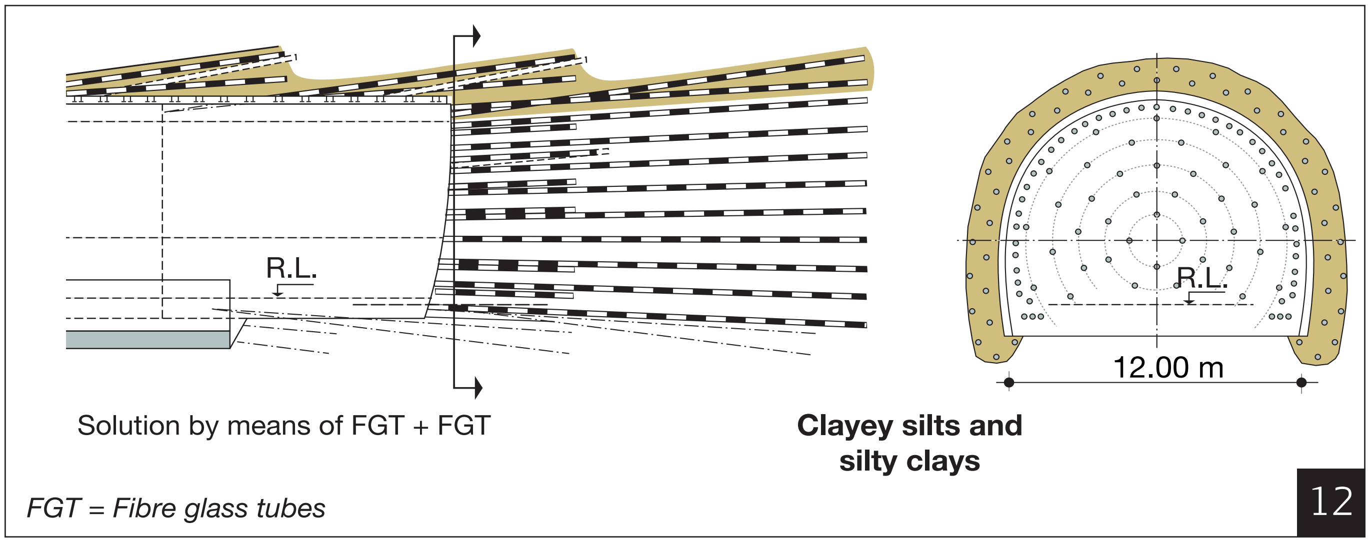
11. COMBINED EXTRUSION AND CONVERGENCE MEASUREMENTS

SAN VITALE TUNNEL



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12. TUNNEL SECTION TYPE

VASTO TUNNEL

of the loads on linings in the long and short term:

• for the former, the VTR + VTR method was introduced (fibre glass reinforcement cemented in the core-face + valved and grouted fibre glass reinforcement ahead of the face around the cavity) to replace PT + VTR, which was not well suited to the ground in question because it was difficult to guarantee the continuity of the precut shell;

• with regard to measurements, new equipment was designed for triaxial cell extrusion tests and for systematic measurement of extrusion, which were very useful. The former were used in the diagnosis stage

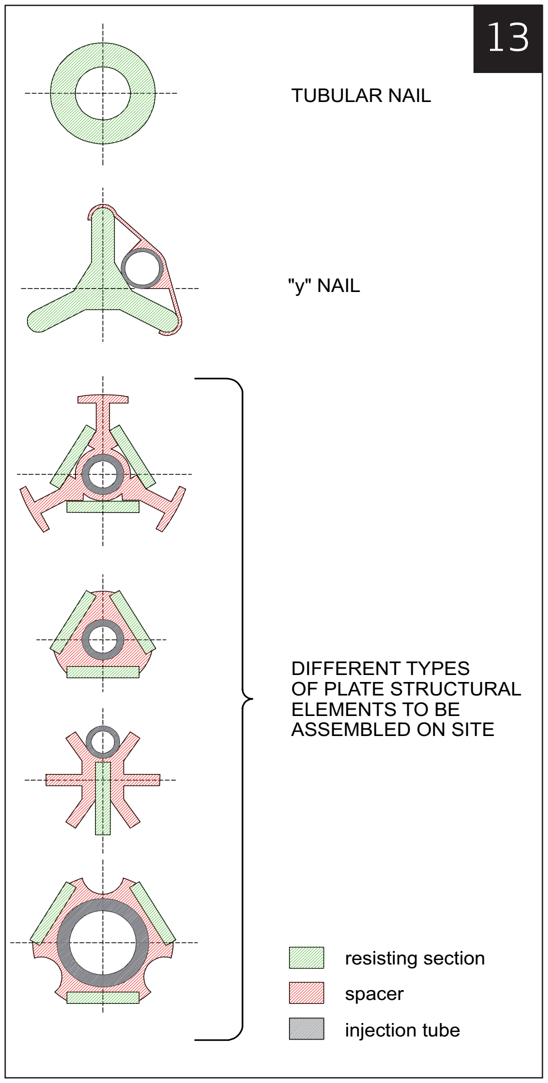
13. *TYPES OF REINFORCEMENT*

14. *MONITORING EXTRUSION OF THE CORE-FACE IN THE SIDE WALL AND CROWN TUNNELS*

"BALDO DEGLI UBALDI" STATION ROME METRO, LINE "A"

15. *TUNNEL SECTION TYPE*

TARTAIGUILLE TUNNEL



to predict behaviour categories and in the therapy stage to calculate the intensity of the reinforcement required to effectively counter extrusion phenomena (Figure 10). The latter were used in the operational stage to calculate the optimum length for reinforcement advance sections and the overlap between them (Figure 11);

• as concerns calculation models, in addition to refining 2D and 3D FEM models, special nomograms were constructed with which the size and distribution of preconvergence ahead of the face could be calculated for the first time;

▸ the "Vasto" tunnel (Ancona-Bari railway line) was driven in 1993 through a heterogeneous formation of silty clays containing substantial aquiferous sandy lentils.

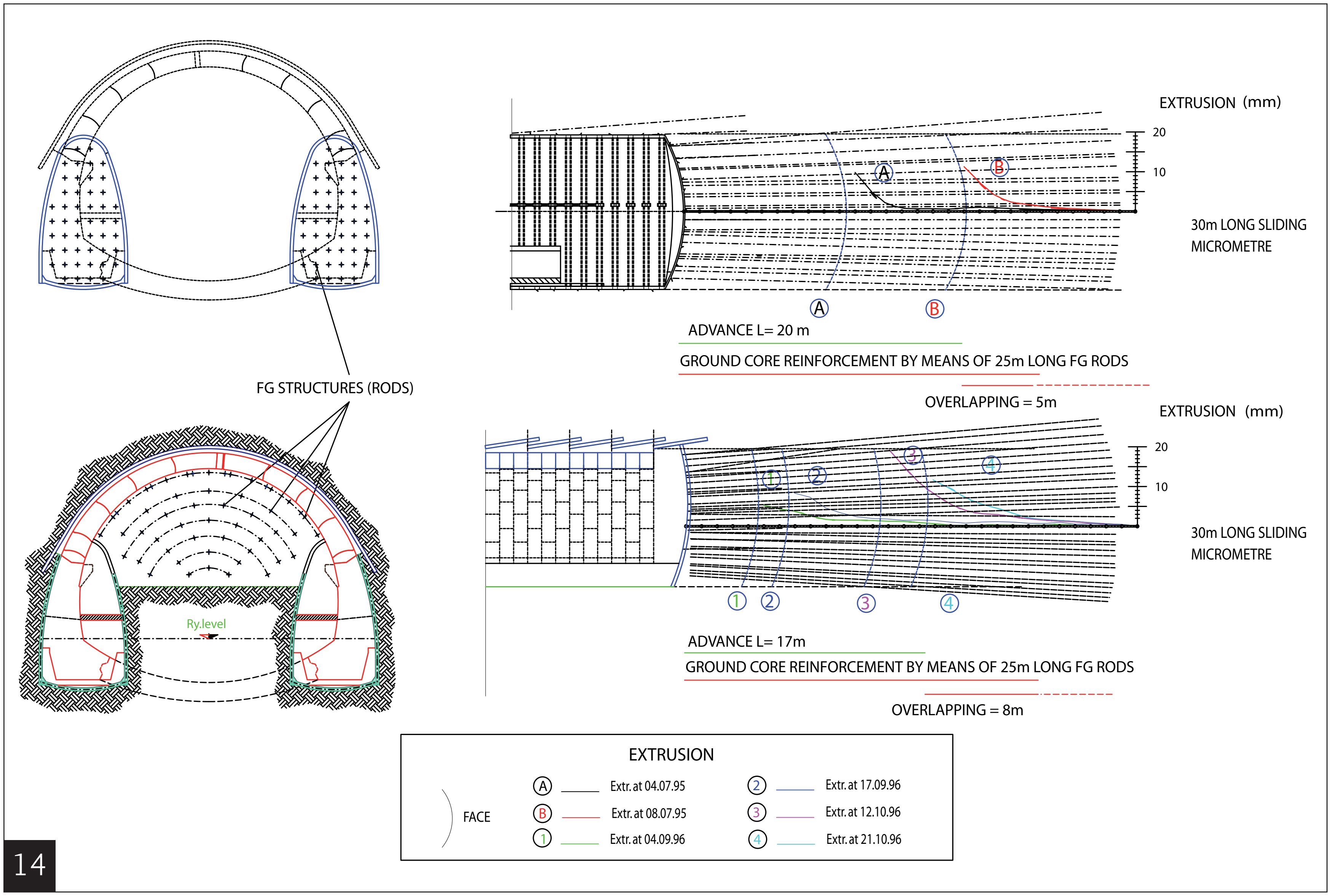
The tunnel passed beneath inhabited houses with a shallow overburden (8 m.) close to the south portal. Here too, the use of normal methods (partial face advance and preliminary stabilisation using steel ribs, radial rock bolts and shotcrete) had failed completely. The use of the ADECO-RS approach made it possible to complete the tunnel at an average speed of approximately 50 m./month.

The HJG+FGT method (fibre glass reinforcement cemented in the core-face + valved and grouted fibre glass reinforcement ahead of the face around the cavity) was introduced for the first time during its construction.

Table 1 on page 236 gives the parameters for the ground reinforcement works.

▸ The tunnel for the "Baldo of the Ubaldi" station on the Rome metro (a span of approximately 22 m. and a cross section of 271 sq. m.) [Lunardi et al, 1998] was driven in 1997 under the centre of the city through clays and sandy silts. Tunnel advance after reinforcement of the core-face was employed for both the side drifts for the tunnel walls and for the subsequent excavation of the crown. Completion of the civil engineering works (excavation and lining of the station tunnel) required just 18 months of work at a cost of approximately 568 euro/cu. m. Surface settlement was negligible. An innovative new type of fibre glass reinforcement element was introduced for this important project, termed a plate element which can not only be assembled in a variety of types (Figure 13), but is also much easier to inject and to transport, making it possible to use reinforcement steps 25 m. in length as opposed to 15 m. + 18 m. achievable previously with tube reinforcement. Table 1 on page 236 gives the parameters for the ground reinforcement operations;

▸ the "Tartaiguille" Tunnel (TGV Mediterranée, on the Marseille-Lyon "G.V." line) [Andre D., Dardard B., Bouvard A., Carmes J., 1999] was driven in 1998 through the "argile du Stampien", a strongly swelling formation (75% montmorillonite content). Faced with the growing difficulties encountered using normal advance methods, the French railways invited major European tunnel designers to present alternative solutions which would allow the remaining 900 m. of tunnel (diameter of 15 m.) to be driven in safety and on time to meet the deadline for the line to start operating. Unanimous opinion was that the tunnel would not be in service by the set deadline. The only exception was that of the solution for full-face advance (180 sq. m.) after reinforcement of the core, based on the ADECO-RS approach, proposed by Prof. Ing. Pietro Lunardi (Figure 15), which forecast completion of the project on time and to budget with daily advance rates of 1.4 m./day guaranteed. The tunnel was actually completed ahead of schedule without problems with average advance rates of approximately 1.5 m./day (during the last five months, with the site operating at full capacity, rates even reached approximately 1.7 m./day) (Figure 16). This exceptional result created quite an impression and aroused some amazement in France. Many in the specialist press there paid tribute to all



EXTRUSION (mm)
20
10
30m LONG SLIDING MICROMETRE
A
B
ADVANCE L= 20 m
GROUND CORE REINFORCEMENT BY MEANS OF 25m LONG FG RODS
OVERLAPPING = 5m
FG STRUCTURES (RODS)
Ry.level
1
2
3
4
ADVANCE L= 17m
GROUND CORE REINFORCEMENT BY MEANS OF 25m LONG FG RODS
OVERLAPPING = 8m
EXTRUSION
FACE
A Extr. at 04.07.95
B Extr. at 08.07.95
1 Extr. at 04.09.96
2 Extr. at 17.09.96
3 Extr. at 12.10.96
4 Extr. at 21.10.96

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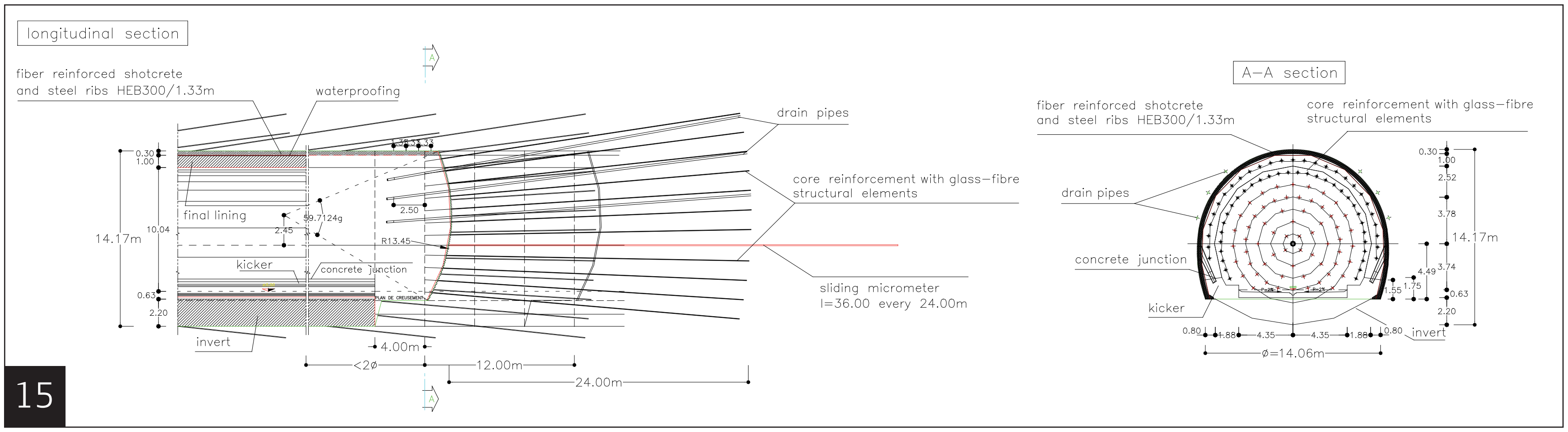
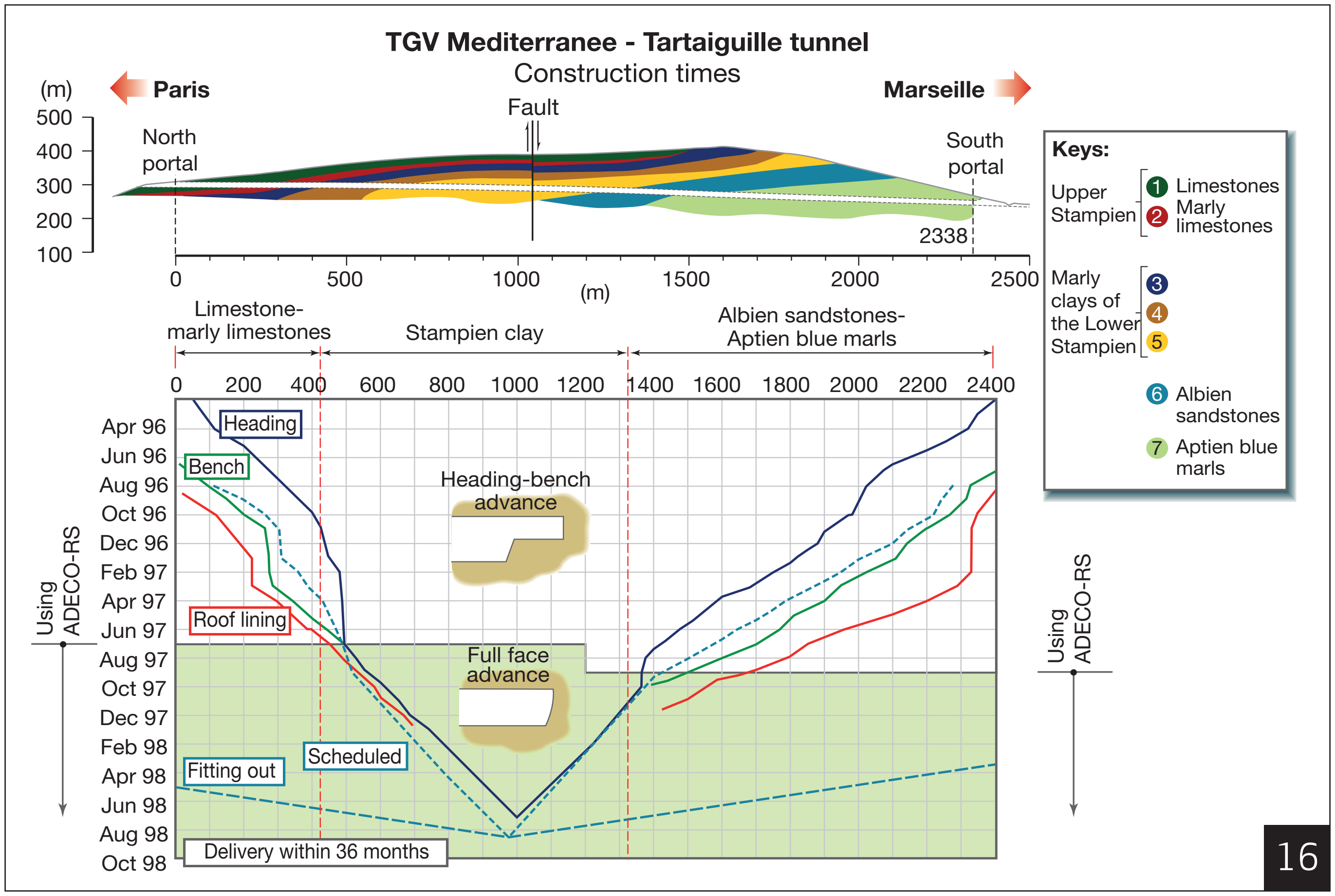


longitudinal section
fiber reinforced shotcrete and steel ribs HEB300/1.33m
waterproofing
drain pipes
core reinforcement with glass–fibre structural elements
final lining
kicker
concrete junction
invert
sliding micrometer l=36.00 every 24.00m
14.17m
4.00m
12.00m
24.00m
A–A section
fiber reinforced shotcrete and steel ribs HEB300/1.33m
core reinforcement with glass–fibre structural elements
drain pipes
concrete junction
kicker
invert
14.17m
ø=14.06m

15

those, the designer, experts and Italian contractors, who had furnished the necessary know-how to complete the project on schedule.
Significant comments included the following: "*Débuté en juillet, le chantier, qui fait travailler 200 personnes, posait principalement des difficultés liées aux pressure exercées par la montagne. Une nouvelle méthode a donc été instaurée, sur l'idée d'un ingénieur italien: le percement en pleine section (plotót qu'en demi-section…*" (Tunnels et ouvrages souterrains, January/February 1998); "*Lorsqu'elle en prend les moyens, l'Italie peut réaliser des travaux à faire pâlir les entreprises françaises…*" (Le Moniteur, 20th February 1998); "*Le creusement du tunnel de Tartaiguille a été très difficile, en raison notamment de convergences inattendues du terrain, qui ont nécessité un changement de méthode en cours de chantier: le professour italien Pietro Lunardi a convaincu la SNCF de travailler à la pelle en pleine section dans les argiles, en boulonnant le front sur 24 m…*" (Le Moniteur, 7th August 1998).
Important studies were conducted during the construction of "Tartaiguille" tunnel on the different types of extrusion and on the importance of the distance at which the tunnel invert is cast during tunnel advance with a stiffened core to minimise the extrusion surface. To date the technology to reinforce the core-face and the framework of the ADECO-RS design approach in which it is set have been employed with excellent results for the construction in Italy alone of around 600 km. of tunnels in a huge variety of different types of ground in many different stress-strain conditions, including those for the new high speed Milan-Rome-Naples railway line between Bologna and Florence.
A list is given below of all the most important projects implemented using Rocksoil designs, which involved the use of fibre glass reinforcement of the advance-core.



16. FACE ADVANCE BEFORE AND AFTER APPLICATION OF THE ADECO-RS APPROACH
TARTAIGUILLE TUNNEL

Start	End	Job	Client	Ground	Max overburden [m]	Span [m]	Length [m]	Special techniques
2009	In progress	Pedemontana Lombarda: Solbiate Olona tunnel / Grandate tunnel / Morazzone tunnel	Pedelombarda S.p.A.	Gravelly Sands, conglomerates / Glaciofluvial deposits, sandstones / Moraine, marly sandstones	15 / 40 / 70	18 / 16 / 16	2x460 / 2x400 / 2x2050	VJG, HJG / HJG, FGT / HJG FGT
2009	In progress	Palermo railway junction - Through tunnel	Sis S.p.A.	Sands and calcarenites	15	8	1700	1700 m HJG, FGT
2009	In progress	State Road No. 212 "Val Fortòre" S. Pietro, M.Leone, Fuciello and Cerzone tunnels	Fortorina S.c.a.r.l.	Flysch	36	14.6	3000	FGT
2008	In progress	State Roads No. 106 "Jonica" and No. 280 modernization (maxilot DG 21/04) - 11 tunnels	CO.MERI S.p.A.	Silty clays	120	13	2x13265	FGT
2008	In progress	Quadrilatero delle Marche - 14 tunnels	C.M.C. GLF, Strabag	Loose deposits, limestones, marly limestones, marls	350	14	30.000	BU, HJG, FGT
2008	In progress	Asti-Cuneo Motorway Link - Alba and Verduno tunnels	Sina S.p.A.	Sandstone and gypsum	95	15	2x3400	VJG, HJG, FGT
2006	In progress	A14 motorway – Widening to 3 lanes for each direction - 4 new tunnels and widening of Monte Domini tunnel by using "Nazzano method"	Spea Ingegneria Europea S.p.A.	Clays	20	16	2x280	Patented method to widen the old tunnel without interrupting traffic
1991	In progress	Milan-Naples A1 motorway modernization: tunnels in the Bologna-Florence section	Toto S.p.A. Todini S.p.A. Baldassini Tognozzi Pontello S.p.A. Impresa S.p.A.	Sands, clays, scaly clays, flysch	400	14.5	2x45000	FGT, HJG, BU
2000	2007	Milan-Naples A1 motorway: Nazzano tunnel widening without interrupting traffic	Autostrade per l'italia S.p.A.	Sands	45	21	1200	Patented method to widen the old tunnel without interrupting traffic
2000	2006	Tunnels along the slip road between A4 and Valtrompia motorway	Autostrada BS VR VI PD S.p.A.	Limestones		12	1700	
1996	2006	New high speed Bologna-Florence railway line: Pianoro tunnel	Maire Engineering	Scaly clays, marls, silts, conglomerates	160	13.5	10706	9126 m FGT, 311 m HJG
1996	2006	New high speed Bologna-Florence railway line: Pianoro large chamber	Maire Engineering	Marls, scaly clays	105	30	418	
1996	2006	New high speed Bologna-Florence railway line: Raticosa tunnel	Maire Engineering	Scaly clays, marls, sandstones	515	13.5	10380	Draining adit in advance, 2x600 m FGT
1995	2006	Aosta-Mont Blanc motorway: Dolonne tunnel	Spea Ingegneria Europea S.p.A.	Granites, schists, calceschists	350	12	2x2850	BU, HJG, FGT
2002	2005	Provincial road No. 169-166: Parscera tunnel	Locatelli			12	1700	
1996	2005	New high speed Bologna-Florence railway line: Borgo Rinzelli tunnel	Maire Engineering	Clays	10	13.5	455	160 m HJG, 295 m MP, 295 m FGT, A.G.O.

Start	End	Job	Client	Ground	Max overburden [m]	Span [m]	Length [m]	Special techniques
1996	2005	New high speed Bologna-Florence railway line: Morticine tunnel	Maire Engineering	Marly-arenaceous siltites	10	13.5	273	193 m FGT, 80 m HJG, A.G.O.
1996	2005	New high speed Bologna-Florence railway line: Mt. Bibele tunnel	Maire Engineering	Marls and sandstones	285	13.5	9118	2327 m FGT, 62 m HJG, BU
1996	2005	New high speed Bologna-Florence railway line: Vaglia tunnel	Maire Engineering	Argillites, marly limestones, sandstones	500	13.5	18647	7342 m FGT, 525 m HJG, BU
2001	2004	Rome Road System: underground connection between Foro Italico Road and Pineta Sacchetti Road	Astaldi	Sands and clayey silts	35	14.7	2500	FGT, HJG
1996	2004	New high speed Bologna-Florence railway line: Sadurano tunnel	Maire Engineering	Conglomerates, silty sandstones	240	13.5	3778	877 m FGT, 65 m HJG, A.G.O.
2002	2003	Alpetunnel: Modane access tunnel	Eiffage	Quartzes, cellular dolomites, micaschists	Var.	10	4000	
2002	2003	Catania-Siracusa motorway: 5 tunnels	Metropolitana Milanese S.p.A.			12	2x6500	
2000	2003	Salerno-Reggio Calabria motorway: Motta tunnel	Toto		70	16	2x600	FGT
1999	2003	Salerno-Reggio Calabria motorway: Vetrano 1 and Vetrano 2 tunnels	Toto	Conglomerates, silty sandstones	30	16	2x1000	
1998	2003	State Road No. 42, Darlo-Edolo section: Capo di Ponte tunnel	Grandi Lavori Fincosit	Sandstones ans siltites	90	12.5	800	
1996	2003	New high speed Bologna-Florence railway line: Firenzuola tunnel	Maire Engineering	Marly-arenaceous Formation	560	13.5	14340	3319 m FGT, 1543 m HJG
1996	2003	New high speed Bologna-Florence railway line: Firenzuola large chamber	Maire Engineering	Marly-arenaceous Formation	120	30	734	
1999	2002	Salerno-Reggio Calabria motorway: Serra Lunga tunnel	Toto	Scaly clays	70	16	2x910	2x910 m FGT
1998	2002	State Road No. 1 "Aurelia": Marinasco tunnel	San Benedetto	Sandstones and argillites	25	12	2x500	
1996	2002	"Aurelia" State Road No. 1: Montenero tunnel	Impregilo	Scaly clay Formation	50	11	2x2150	2x150 m HJG, 2x2350 m FGT

Start	End	Job	Client	Ground	Max overburden [m]	Span [m]	Length [m]	Special techniques
1999	2001	"Aurelia" State Road No. 1 (variant road near Savona): 4 tunnels	Bonifica	Limestones	20	13	6000	
1998	2001	"Aurelia" State Road No. 1 (variant road near La Spezia): 5 tunnels	Bonifica	Argillites	60	13	10000	
1996	2001	Ravone railway yard at Bologna: Ravone tunnel	C.M.C. Adanti	Sands and silty gravels	14	18	2x900	2x900 m HJG
1994	2001	New high speed Rome-Naples railway line: Collatina tunnel	Iricav Uno	Pyroclastites	8	13.5	55	
1994	2001	New high speed Rome-Naples railway line: Massimo tunnel	Iricav Uno	Pyroclastites, lava	34	13.5	1139	120 m FGT
1994	2001	New high speed Rome-Naples railway line: Colli Albani tunnel	Iricav Uno	"Colli Albani" volcanites	75	13.5	6357	353 m HJG, 90 m FGT
1994	2001	New high speed Rome-Naples railway line: Sgurgola tunnel	Iricav Uno	Limestones Formation	114	13.5	2237	FGT
1994	2001	New high speed Rome-Naples railway line: Macchia Piana 1 tunnel	Iricav Uno	"Valle del Sacco" volcanites	43	13.5	970	103 m FGT
1994	2001	New high speed Rome-Naples railway line: Macchia Piana 2 tunnel	Iricav Uno	Pyroclastites	15	13.5	480	480 m FGT
1994	2001	New high speed Rome-Naples railway line: La Botte tunnel	Iricav Uno	Pyroclastites, lava, clays	52	13.5	1185	498 m FGT
1994	2001	New high speed Rome-Naples railway line: Castellona tunnel	Iricav Uno	Clays	60	13.5	469	119 m FGT
1994	2001	New high speed Rome-Naples railway line: S. Arcangelo tunnel	Iricav Uno	Pyroclastites, marls	48	13.5	580	
1994	2001	New high speed Rome-Naples railway line: Selva Piana tunnel	Iricav Uno	Pyroclastites	14	13.5	132	34 m HJG, 98 m FGT
1994	2001	New high speed Rome-Naples railway line: Collevento tunnel	Iricav Uno	Pyroclastites, clays	19	13.5	380	380 m FGT
1994	2001	New high speed Rome-Naples railway line: Selvotta tunnel	Iricav Uno	Clays	11	13.5	163	65 m HJG, 48 FGT
1994	2001	New high speed Rome-Naples railway line: Colle Pece tunnel	Iricav Uno	Scaly clays	33	13.5	873	HJG, FGT
1994	2001	New high speed Rome-Naples railway line: Campo Zillone 1 tunnel	Iricav Uno	"Rocca Monfina" volcanites	48	13.5	2616	FGT, HJG

Start	End	Job	Client	Ground	Max overburden [m]	Span [m]	Length [m]	Special techniques
1994	2001	New high speed Rome-Naples railway line: Campo Zillone 2 tunnel	Iricav Uno	Pyroclastites	25	13.5	350	FGT, 17 m HJG
1994	2001	New high speed Rome-Naples railway line: Piccilli 1 tunnel	Iricav Uno	Pyroclastites	17	13.5	842	288 m FGT, A.G.O.
1994	2001	New high speed Rome-Naples railway line: Piccilli 2 tunnel	Iricav Uno	Pyroclastites	28	13.5	485	FGT, A.G.O.
1994	2001	New high speed Rome-Naples railway line: Lompari tunnel	Iricav Uno	Clays	13	13.5	200	46 m HJG, A.G.O.
1994	2001	New high speed Rome-Naples railway line: Caianello tunnel	Iricav Uno	Pyroclastites	10	13.5	830	42 m HJG, A.G.O.
1994	2001	New high speed Rome-Naples railway line: Briccelle tunnel	Iricav Uno	Marly-Arenaceous Complex, Limestone and Ignimbrite Formations	78	13.5	1033	4 m HJG, 288 m FGT
1994	2001	New high speed Rome-Naples railway line: Castagne tunnel	Iricav Uno	Pyroclastites	8	13.5	289	A.G.O.
1994	2001	New high speed Rome-Naples railway line: Santuario tunnel	Iricav Uno	Pyroclastites	10	13.5	322	A.G.O.
1986	2001	Spriana (Valtellina) landslide protection works: by-pass hydraulic tunnels	Impregilo	Gneiss, granodiorites, micaschists	350	4	2000	Full face TBM, IN, FGT
1998	2000	G.R.A. Roma, Lot 19 - Widening of the road: Appia Antica tunnel	Condotte d'acqua	Pyroclastites	18	20.7	1414	FGT
1998	2000	P.te Mammolo-Via della Bufalotta road link: Capo di Ponte urban tunnel	Comune di Roma					
1997	1999	TGV Méditerranèe, Marseille-Lyon railway line: Tartaiguille tunnel	Tartaiguille	Over-consolidated clays	110	15	900	900 m FGT
1996	1999	Rome Road System: Monte Mario tunnel	Astaldi	Sands and cemented sands, silty sands, clayey silts	270	14.5	2x2300	1200 m MP, 25 m HJG, 350 m FGT 150 m BU
1993	1999	Ancona-Bari railway line: Vasto tunnel	Fioroni	Silty clays	135	12	5000	2260 m HJG, 2600 m MP, 4970 m FGT
1990	1999	State Road No. 38: Valmaggiore and Bolladore tunnels	Secol	Diorites, gneiss, gabbros, phyllites	300	12	2400	150 m HJG, FGT
1985	1999	Sibari-Cosenza railway line: 1, 2, 3 and 4 tunnels	Asfalti Sintex	Pliocenic Calabrian Formation	115	10	7000	1300 m HJG, 2300 m MP, 2300 m FGT
1984	1999	Verona-Brennero railway line: Fleres tunnel	Comer	Dolomites, paragniess, calc-schists	1000	12	7300	BU, FGT
1996	1998	Rome Road System: Principe Amedeo tunnel	Di Penta	Tufs with fluvial-lacustrine intercalations	20	10.5	860	

Start	End	Job	Client	Ground	Max overburden [m]	Span [m]	Length [m]	Special techniques
1989	1998	Sardinian Dorsal railway line: Campeda tunnel	Cofesar	Volcanic grounds	300	12	3800	400 m MP, 400 m FGT
1990	1997	State Road No. 38: Le Prese and Verzedo tunnels	Secol	Diorites, gneiss, gabbros, phyllites	300	12	3100	100 m HJG, FGT
1988	1997	Rome metro - Line A - "Baldo degli Ubaldi" station	Intermetro S.p.A.	Clayey and sandy-silty formations	22	22	120	MP, FGT, AA
1987	1997	Florence-Empoli railway line: S. Vito and Bellosguardo tunnels	Firem	Le Piatre marls, chaotic complex in calcareous-marly facies, M. Modino sandstones	160	12	3510	255 m HJG, 1600 m FGT
1992	1996	E 45 - Orte-Ravenna motorway: Quarto tunnel	Toto	Sandstones, marls and sandy silts	150	11	2x2500	2x100 m HJG, 2x200 m FGT
1991	1996	Caserta-Foggia railway tunnel: San Vitale tunnel	San Vitale Scarl	Scaly clays, limestones, argillites	100	12	2500	300 m MP, 1300 m FGT
1988	1996	State Road No. 38: Tola tunnel	Secol	Diorites, gneiss, gabbros, phyllites	300	12	1500	2x900 m HJG, 2x900 m FGT
1989	1996	Aosta-Mont Blanc motorway: Avise tunnel	R.A.V.	Paragneiss and calc-schists	400	12	2x2700	2500 m HJG, 5500 m MP, 2700 m FGT
1988	1993	State Road No. 38: Cepina tunnel	Secol	Diorites, gneiss, gabbros, phyllites	300	12	2900	70 m HJG, BU
1989	1993	Livorno-Civitavecchia motorway: Rimazzano tunnel	Sotecni	Sands e gravels in silty matrix, pliocenic clays	20	12	2x900	2x900 m HJG, 2x900 m FGT
1988	1993	Nuraxi Figus mining inclined shaft	Torno S.p.A.	Volcanites, "Cixerri" Formation, eocenic series	460	8		150 m HJG, FGT
1988	1991	Rome-L'Aquila-Teramo motorway-Lot 4: Colledara tunnel	Colledara	Marls and lacustrine deposits	50	10	1600	
1987	1991	"Direttissima" Rome-Florence railway line: Talleto and Caprenne tunnels	Fespi	Sandy silts	80	7	2700+2700	2500 m HJG, 5500 m MP, 2700 m FGT
1987	1991	"Direttissima" Rome-Florence railway line: Poggio Orlandi tunnel	Fespi	Sandy silts	50	13.5	1200	250 m HJG, 600 m FGT
1987	1991	"Direttissima" Rome-Florence railway line: Crepacuore tunnel	Fespi	Sandy silts	50	13.5	700	60 m HJG, 120 m FGT
1987	1991	"Direttissima" Rome-Florence railway line: Tasso tunnel	Fespi	Sandy silts	50	13.5	2000	150 m HJG, 1650 m FGT
1987	1991	"Direttissima" Rome-Florence railway line: Terranova Le Ville tunnel	Fespi	Lacustrine deposits	50	13.5	2600	200 m HJG, 1800 m MP, 2200 m FGT

Start	End	Job	Client	Ground	Max overburden [m]	Span [m]	Length [m]	Special techniques
1989	1989	"Sebina Orientale" No. 510 State Road - Lot 7	Secol	"Lombard Cerrucano" limestones, gypsum and anhydrites	150	11	5500	900 m HJG, 400 m FGT
1986	1987	Citronia-Stirpi hydraulic tunnel at Salsomaggiore (Parma)	Magistrato per il Po	Scaly clays		4.5		FGT (first in the world)
1993		Valfabbrica (Perugia) No. 318 State Road: S. Egidio tunnel		Silts, clays e sands	35	12	600	700 m FGT
1991		"Sebina Orientale" No. 510 State Road - Lot 6	Secol	Limestones and dolomites	150	11	5000	900 m HJG, 400 m FGT
1993		Roccella Jonica No. 106 State Road: Lofiri tunnel		Clays, marls	35	12	350	280 m FGT
1993		Roccella Jonica No. 106 State Road: Giulia tunnel		Granitic sands and scaly clays	40	12	780	280 m FGT
1990		State Road No. 38: Mondadizza tunnel	Secol	Diorites, gneiss, gabbros, phyllites	300	12	1400	60 m HJG, FGT
1989	1991	Catanzaro East by-pass: S. Giovanni tunnel	Sincat Scarl	Stratified marly clays and yellowish sands and silts alternate under water-table	40	12	400	400 m MP, 400 m FGT
1989	1994	Bicocca-Siracusa railway line: Targia tunnel	Collini S.p.A.	Hyaloclastites, calcarenites	60	12	3300	850 m MP, 1000 m FGT
1991		State Road No. 237: Sabbio tunnel				11		FGT
1990		Torino-Savona motorway: Montezemolo tunnel	Autostrada TO-SV	Marls and sandstones	140	11	1800	FGT
1990		State Road No. 38: S. Antonio tunnel	Secol	Diorites, gneiss, gabbros, phyllites	300	12	2300	FGT
1988		Roma-L'Aquila-Teramo motorway - Lot 4: Sodera tunnel		Marls and lacustrine deposits		10		125 m HJG, 360 m FGT
1987		Reggio Calabria C.Le-Metaponto railway line: Capo d'Armi tunnel		Limestones	70	12	1000	125 m HJG, 360 m FGT
1987		Doubling of Circumflegrea (Naples) railway line: Varo Pecore, Astroni and Grotta del Sole tunnels		Tufs with fluvial-lacustrine intercalations	30	6	600	600 m MP, 600 m FGT
1986		Sonico-Cedegolo hydraulic tunnel in Val Camonica	Selm Montedison	Scaly clays		4.5		
1985		West Campania waterworks: Cassino tunnel	Cogefar	Limestones, dolomites		4.5	3200	
						Total length	**~ 580 Km**	

APPIA ANTICA TUNNEL

ROME MOTORWAY RING ROAD

∅ = 20.65 M.

GROUND: GRANULAR PYROCLASTITES

OVERBURDEN: ~ 4 M.

BIBLIOGRAPHY

LUNARDI P., CASSANI G., GATTI M., 2008. Design Aspects of the construction of the new Apennines crossing on the A1 Milan-Naples motorway: the Base Tunnel, Proceedings of the International Congress on "Le Souterrain: espace d'avenir", Monaco, Principato di Monaco, 6-8 ottobre 2008

LUNARDI P., 2008. Design and construction of tunnels - Analysis of Controlled Deformation in Rock and Soils (ADECO-RS), Ed. SPRINGER, 576 pagine, marzo 2008

LUNARDI P., BINDI R., CASSANI G., 2007. From the ADECO-RS approach to the tunnelling industrialisation, Atti del Simposio Internazionale su "Tunnelling for urban development", Pattaya City (Thailand), 14-15 dicembre 2007

LUNARDI P., 2007. Progetto e costruzione di gallerie - Analisi delle deformazioni controllate nelle rocce e nei suoli (ADECO-RS), Ed. HOEPLI, 575 pagine, agosto 2007

LUNARDI P., BINDI R., CASSANI G., 2006. Prime evidenze e risultati dell'impiego dell'approccio ADECO-RS per la realizzazione di oltre 73 Km di gallerie di linea, Strade e Autostrade, 1-2006

LUNARDI P., BINDI R., 2004. The evolution of reinforcement of the advance core using fibre glass elements for short and long term stability of tunnels under difficult stress-strain conditions: design, technologies and operating methods

LUNARDI P. E ALTRI, 2001. Aspetti progettuali e costruttivi affrontati nell'ambito dei lavori per il raddoppio della linea ferroviaria Bicocca-Siracusa, tratto Targia-Siracusa, Atti del Congresso Internazionale su "Progress in Tunnelling after 2000", Milano, 10-13 giugno 2001

LUNARDI P., BINDI R., 2001. The evolution of reinforcement of the advance core using fibre glass elements for short and long term stability of tunnels under difficult stress-strain conditions: design, technologies and operating methods, Atti del Congresso Internazionale su "Progress in Tunnelling after 2000", Milano, 10-13 giugno 2001

LUNARDI P., 2001. Approccio ADECO-RS: risoluzione delle problematiche relative al sottoattraversamento dell'autodromo internazionale del Mugello in condizioni di ridotta copertura, Quarry and Construction, maggio 2001

LUNARDI P., 2000. Progetto e costruzione di gallerie – Approccio ADECO-RS, Quarry and Construction, maggio 2001

LUNARDI P., 2000. Tunnelling under the Mugello Racing Circuit incorporating the ADECO-RS approach, Tunnel, n. 8 (December)

KOVARI K., LUNARDI P., 2000. On the observational method in tunnelling, Atti del Convegno Internazionale su "Geotechnical and Geological Engineering", Melbourne, 19-24 novembre 2000

LUNARDI P., PANTALEONE V., 2000. Problematiche affrontate nello scavo delle gallerie Bellosguardo e San Vito, Quarry and Construction, October 2000.

LUNARDI P., 2000. Design & constructing tunnels – ADECO-RS approach, T&T International special supplement, May 2000

LUNARDI P., 2000, Tunnelling under the Via Appia Antica in Rome, Tunnels & Tunnelling International, April 2000

LUNARDI P., MINENNA M., 2000. La galleria di sottopasso della strada consolare romana "Appia Antica", Strade e autostrade, n. 2 anno 2000

LUNARDI P., 1999. The influence of the rigidity of the advance core on the stability of tunnel excavations, Gallerie e grandi opere sotterranee, 59

LUNARDI P., 1999. Il preconsolidamento del terreno nelle opere in sotterraneo, Quaderno della 3° Sessione delle Conferenze Permanenti SIG sull'Alta Velocità, Bologna, 24 giugno 1999

ANDRE D., DARDARD B., BOUVARD A., CARMES J., 1999. "La traversée des argiles du tunnel de Tartaiguille", Tunnels et ouvrages en souterrains, n°. 153, May-June 1999

LUNARDI P., 1999. La galleria "Tartaiguille", ovvero l'applicazione dell'approccio ADECO-RS per la realizzazione di un tunnel "impossibile", Gallerie e grandi opere sotterranee, n. 58 anno 1999

LUNARDI P., 1998. Convergence-confinement ou extrusion-préconfinement ?, Colloque "Mécanique et Géotechnique", Laboratoire de Mécanique des Solides - École Polytechnique, Paris 19 May 1998

LUNARDI P. E ALTRI, 1998. Design and construction of a station on the Rome metro, T&T International, March 1998

LUNARDI P. E ALTRI, 1998. Stazione "Baldo degli Ubaldi", Quarry and Construction, marzo 1998

LUNARDI P., 1997. L'influenza della rigidezza del nucleo d'avanzamento sulla sicurezza degli scavi in galleria, Gallerie e grandi opere sotterranee, n. 52 anno 1997

LUNARDI P. E ALTRI, 1997. Aspetti progettuali e costruttivi della galleria "Vasto", Quarry and construction, August 1997

LUNARDI P., 1995. Fibre-glass tubes to stabilize the face of tunnels in difficult cohesive soils, Materials Engineering, 1995, vol 6, n. 1-2, pagg. 107-165

LUNARDI P. E ALTRI, 1994. Pianificazione tecnico-economica delle opere in sotterraneo mediante l'approccio basato sull'analisi delle deformazioni controllate nelle rocce e nei suoli, Costruzioni, Settembre 1994

LUNARDI P., 1993. La stabilité du front de taille dans les ouvrages souterraines en terrain meuble: études et experiences sur le renforcement du noyau d'avancement, Symposio Internazionale su "Renforcement des sols: experimentations en vraie grandeur des annes 80", Paris, 18 November 1993

LUNARDI P., 1993. Fibre-glass tubes to stabilize the face of tunnels in difficult cohesive soils, SAIE : Seminar on "The application of fiber Reinforced Plastics (FRP) in civil structural engineering" - Bologna, 22 Ottobre 1993

LUNARDI P., 1993. Evolution des technologies d'excavation en souterrain, dans des terrains meubles, Colloque chez le "Comité Marocain des Grands Barrages" - Rabat, 30 settembre 1993

LUNARDI P. E ALTRI, 1993. Technical and financial planning of underground works: the "ADECO-RS" method, VI Forum Europeo di Ingegneria Economica - Università Bocconi - Milano, 13-14 Maggio 1993

LUNARDI P., 1993. Aspetti progettuali e costruttivi nella realizzazione di gallerie in situazioni difficili. Interventi di precontenimento del cavo, Le strade, marzo-aprile 1993

LUNARDI P. E ALTRI, 1993. Tunnelling in soil: design, construction materials and monitoring, Seminario Russo-Italiano - Central Institute for Advanced Training of Construction Engineers (TSMIPKS), Mosca, 11 Marzo 1993

LUNARDI P. E ALTRI, 1992. Tunnel face reinforcement in soft ground: design and controls during excavation, Convegno Internazionale su "Towards New Worlds in Tunnelling" - Acapulco 16-20 May 1992

LUNARDI P., 1991. Aspetti progettuali e costruttivi nella realizzazione di gallerie in situazioni difficili: interventi di precontenimento del cavo, Convegno Internazionale su "Il consolidamento del suolo e delle rocce nelle realizzazioni in sotterraneo" - Milano 18-20 Marzo 1991

LUNARDI P. E ALTRI, 1990. Soft ground tunnelling in the Milan Metro and Milan Railway Link. Case histories, Soft Ground Tunnelling Course - Institution of Civil Engineers - Londra 10-12 Luglio 1990

LUNARDI P. E ALTRI, 1989. Nouvelles orientations pour le projet et la construction des tunnels dans des terrains meubles. Etudes et experiences sur le preconfinement de la cavité et la préconsolidation du noyau au front, Colloquio Internazionale su "Tunnels et micro-tunnels en terrain meuble - Paris 7-10 February 1989

1990

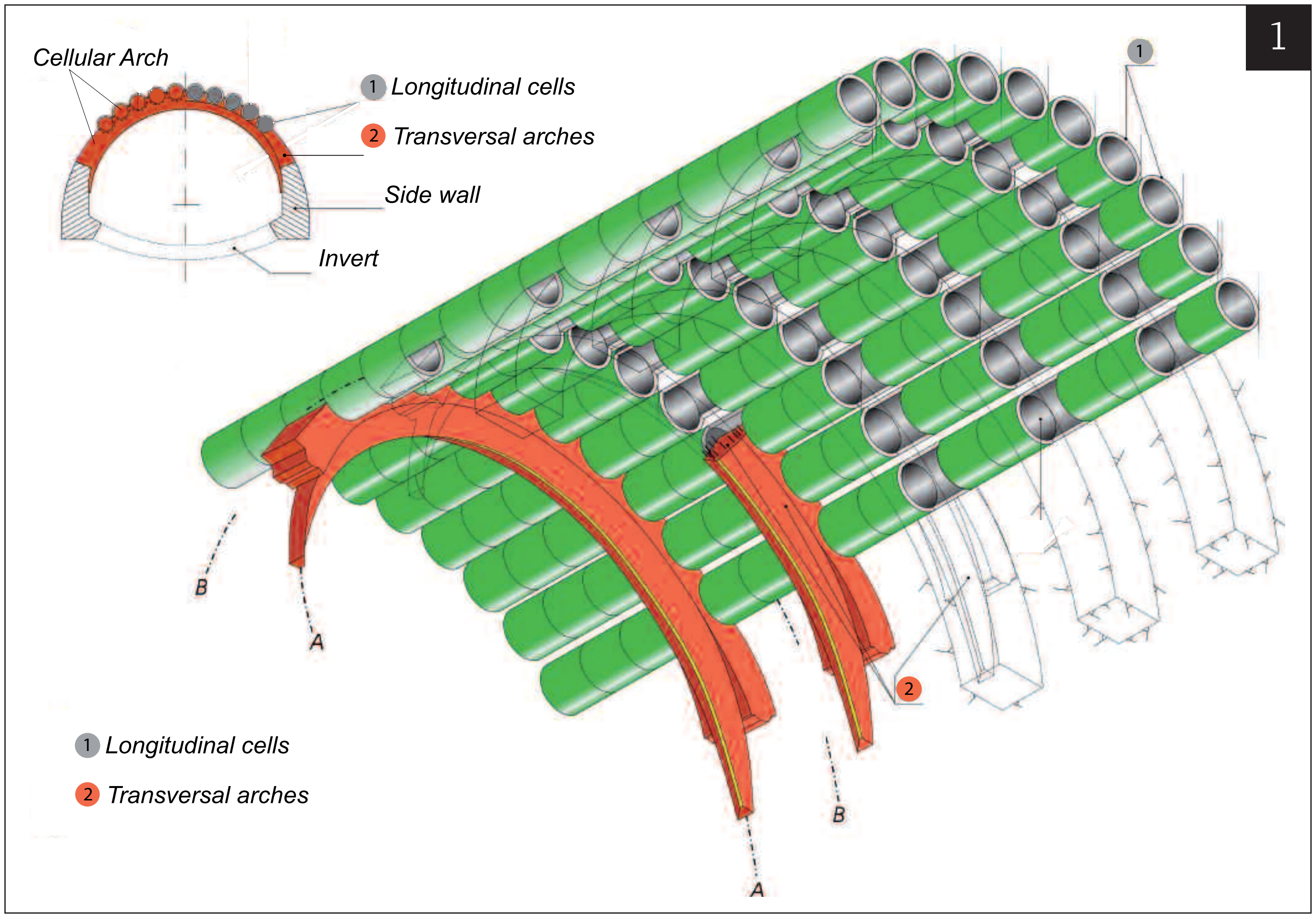
CELLULAR ARCH

Large underground spans even in urban areas

Cellular arch technology was studied and developed by Rocksoil in 1986 on the basis of a shrewd insight by Prof. Ing. Pietro Lunardi, for which he received recognition from the United States journal Engineering News Record which each year nominates a "Man of the Year" in the construction field.

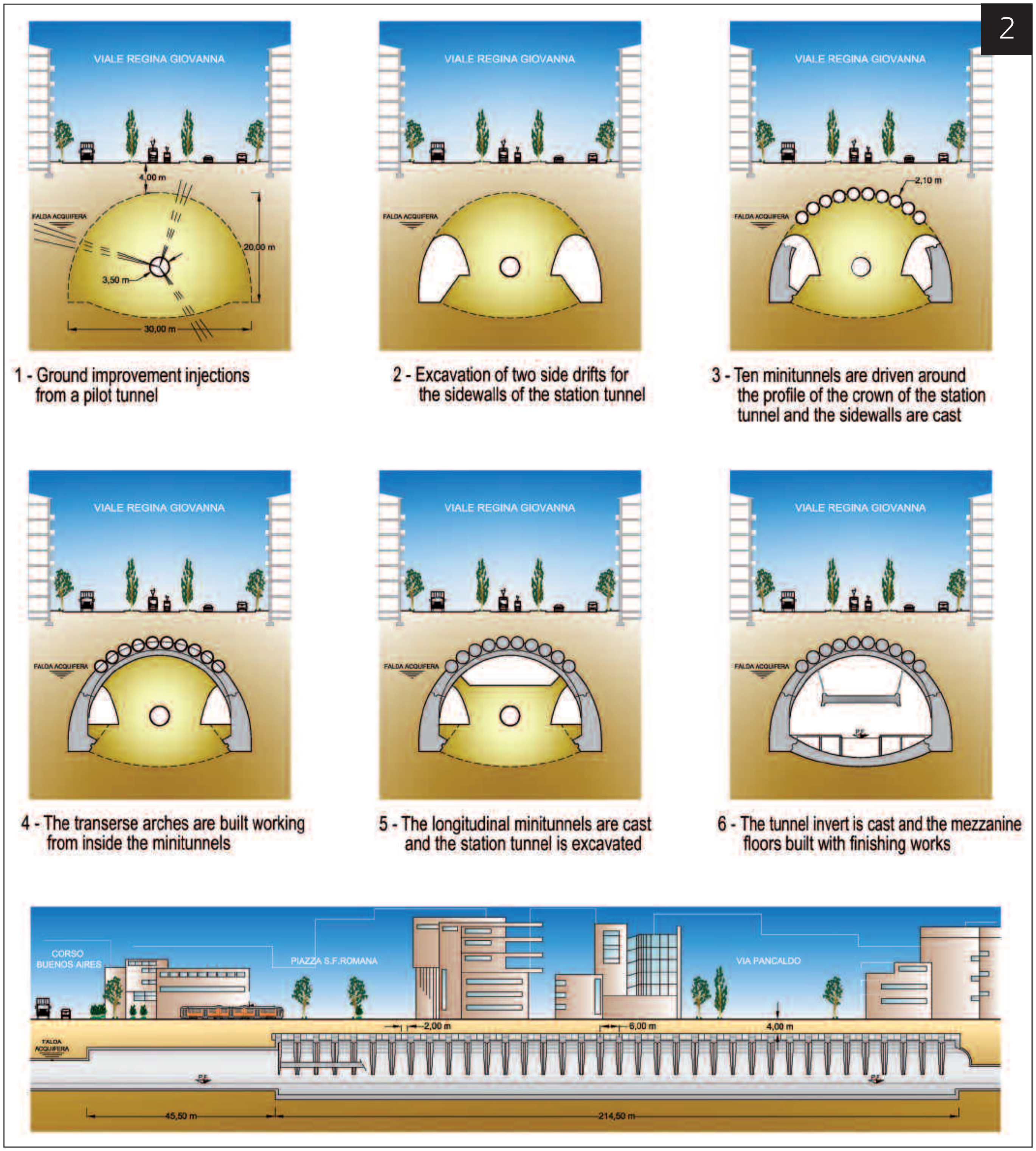
Implemented with great success for the underground construction in loose ground of the Venezia Station on the Milan Urban Railway Link (30 m. in diameter of excavation with an overburden of just 4 m. under ancient eighteenth century buildings), it was considered sensational in the tunnelling world and in the

1. CONSTRUCTION STAGES FOR THE "CELLULAR ARCH"



THE STRUCTURE OF THE CELLULAR ARCHED VAULT (VIEW FROM THE MEZZANINE)
MILAN URBAN RAILWAY LINK "VENEZIA" STATION

near future it could be applied as a solution for similar projects both in Italy and in other countries where it is considered a serious proposition. The cellular arch is in fact a construction technology particularly appropriate for the construction of large span tunnels in urban environments when the geotechnical and stress-strain situations, shallow overburdens and the requirement for construction



2. *BUILDING THE CELLULAR ARCH STRUCTURE FOR VENEZIA STATION*

THE MILAN URBAN LINK RAILWAY LINE

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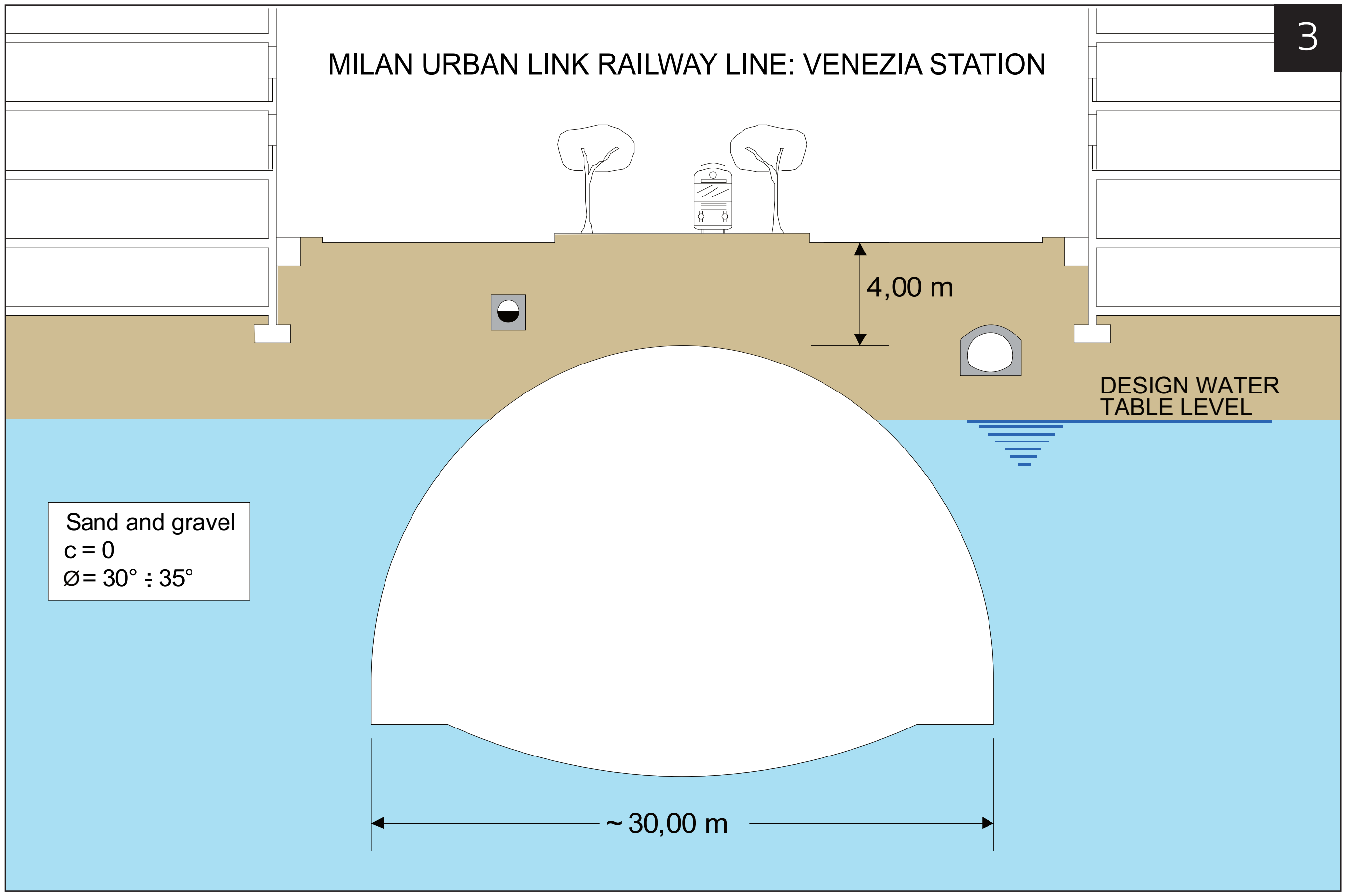
work to have negligible effects on surface constructions and activities, are either not compatible with conventional tunnelling methods, or make them less reliable and competitive.
It consists of a composite structure (Figure 1) similar to a lattice framework with a semi-circular cross section where the longitudinal elements (cells), which consist of tubes filled with r.c., are made to act together by a series of large transverse ribs (arches).
Studies performed to find the limits to its use suggest that it is possible to employ it to build shallow overburden, underground cavities with a span of more than 60 m. even in loose soils and under the water table, without causing any appreciable surface settlement. The feature which gives the cellular arch design the edge over conventional construction techniques is the way in which the passage is managed from the initial equilibrium of the still undisturbed ground to that of its final equilibrium when the tunnel is finished. It is performed in a manner which prevents the material from decompressing and as a consequence also surface settlement from occurring.
Excavation of the final tunnel is in fact not carried out until the load bearing structure, which is very rigid, has already been fully constructed and is able to furnish the ground with the indispensible confinement required, without it deforming to any appreciable extent. Because of that characteristic, the cellular arch method is classified, within the framework of the ADECO-RS approach, among those techniques used to protect the core-face, as a conservative, cavity preconfinement, technology.
The technology involves the use of the following main stages, in order to construct the entire "cellular arch" structure in the ground before starting to excavate a tunnel (Figure 2):

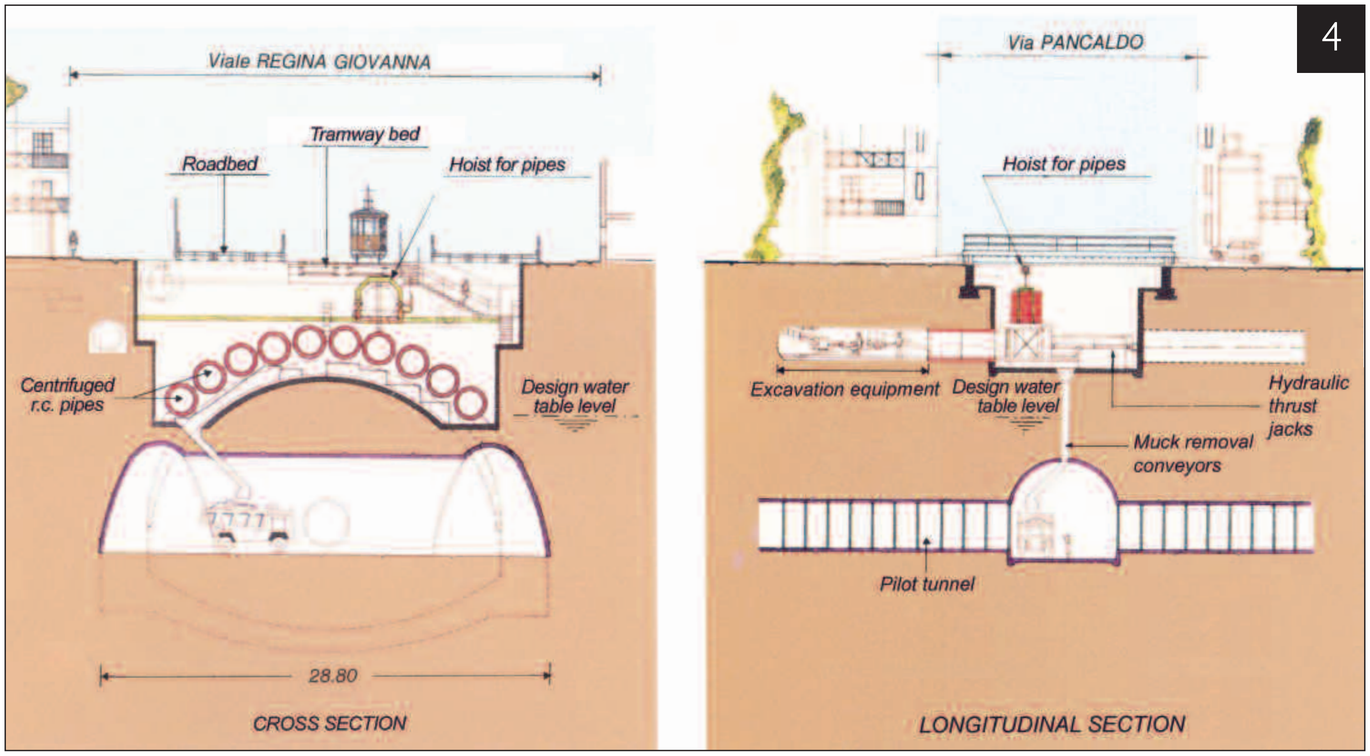


3. THE CONSTRUCTION OF THE "VENEZIA" STATION
WITH A NET SPAN OF 25 M. AND A LENGTH OF 215 M. IT WAS THE LARGEST UNDERGROUND CONSTRUCTION ON THE ENTIRE LOMBARD TRANSPORT NETWORK. THE SLENDER SIZE OF THE OVERBURDEN MADE IT IMPOSSIBLE TO CONSTRUCT USING NORMAL METHODS, BECAUSE THERE WAS NO WAY OF CREATING A BAND OF TREATED GROUND AROUND IT OF ADEQUATE THICKNESS

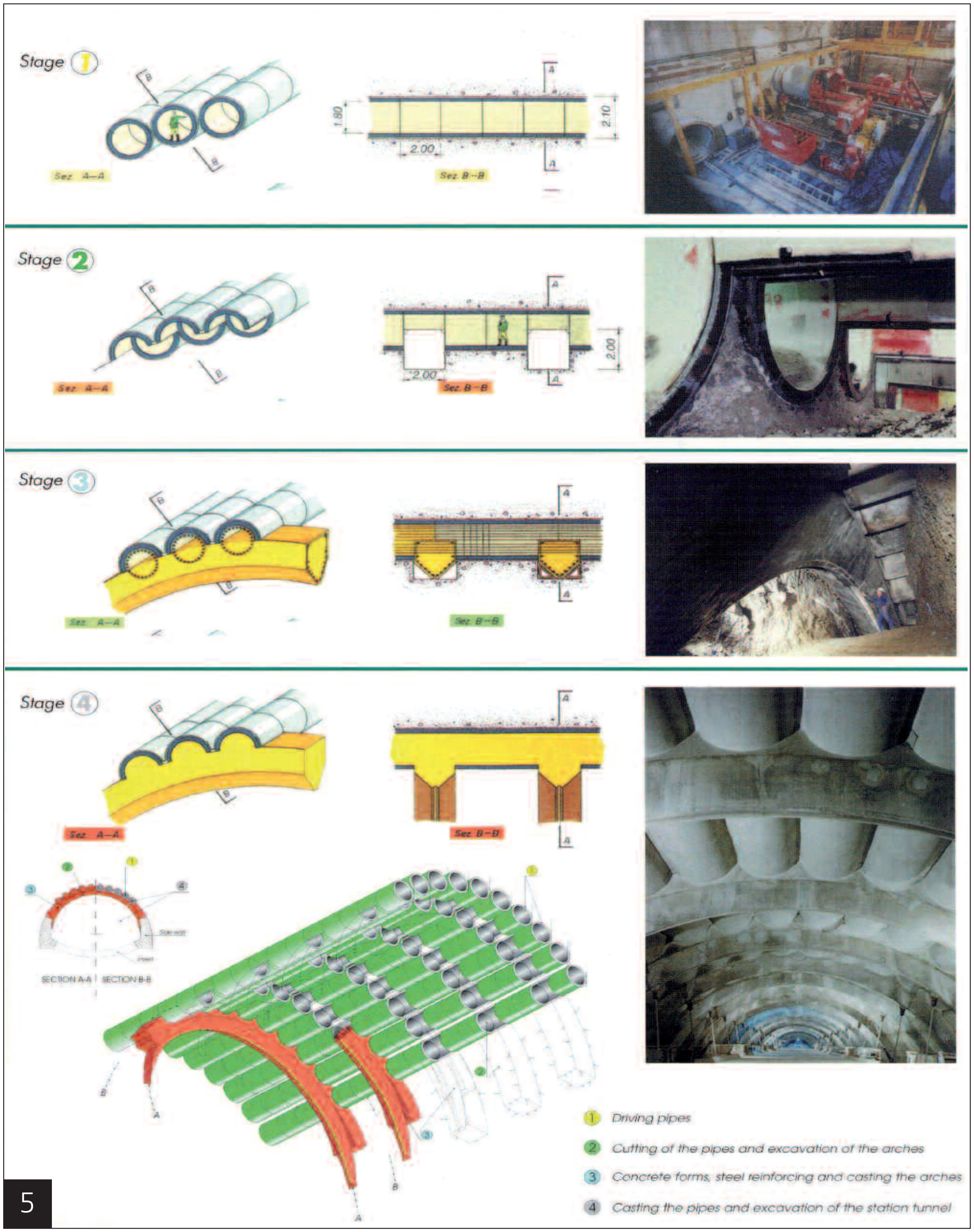
1. improvement of the ground around the future station tunnel, by working from a pilot tunnel and using conventional grouting methods;
2. half section excavation of the tunnel side walls and complete improvement of the ground around the final tunnel by working from them;
3. completion of the excavation of the drifts for the side walls of the tunnel and subsequent casting of the side walls of the station tunnel, while, on a completely independent construction site above, a series of r.c. pipes (minitunnels) are jacked into the ground side by side (diameter 2.10 m.) around the profile of the crown of the future tunnel;
4. excavation, from the minitunnels, of transverse tunnels which will be used as the formwork (the walls of which are formed by the ground itself) for casting the connecting arches in r.c.; the reinforcement for the arches and cells is then placed and they are cast;
5. casting of the longitudinal minitunnels in the crown and excavation of the ground inside the section of the station tunnel under the protection of the cellular arch which is already practically active;
6. casting of the tunnel invert in steps.

CONSTRUCTION OF VENEZIA STATION USING CELLULAR ARCH TECHNOLOGY

As already mentioned the cellular arch technology was conceived of by Prof. Ing. Pietro Lunardi in 1986 to solve the problem of the construction of the Venezia Station on the Milan Urban Link Line. Strategically located in the central business district of the city, with a net span of 25 m. and a length of 215 m., this is the largest underground construction on the entire regional transport network. The 440 sq. m. excavated cross section of the station tunnel was six times larger than that of a normal twin track metropolitan railway running tunnel and almost twice the size of the second largest tunnel built in Milan. Given, amongst other things, the huge dimensions of the excavation required, its construction immediately presented many problems. The numerous underground services present, including Line One of the metro, meant that the new construction had to be positioned close to the surface. Also the requirement not to disturb traffic flow above on viale Regina Giovanna meant work had to be performed underground and therefore cut and cover was not an option.



4. THE THRUST PIT OR CHAMBER
THE MILAN URBAN LINK RAILWAY LINE VENEZIA STATION



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5. *CELLULAR ARCH*
CONSTRUCTION OF THE CROWN
THE MILAN URBAN LINK RAILWAY LINE
VENEZIA STATION

THE THRUST CHAMBER: THE PREFABRICATED PIPES AFTER BEING DRIVEN

THE MILAN URBAN LINK RAILWAY LINE VENEZIA STATION

At the design stage, the extremely shallow overburden (approximately 4 m.) generated huge uncertainties, right from the outset, over the results that could be achieved by using the conventional construction method used in Milan for similar projects, based on removing the ground in steps after improving it around the future tunnel, by using grout injections and then immediately placing steel ribs and shotcrete. In fact the lack of cover would have made it impossible to treat a sufficient thickness of ground in advance that would be adequate for the huge dimensions of the cross section to be excavated.
The numerical analyses performed using finite element software soon confirmed these doubts, clearly indicating that a conventional confinement structure, consisting of steel ribs and shotcrete would have deformed too much and would not have been able, even at the preliminary stage, to contain surface settlement within the limits required to safeguard nearby structures and underground services. The only solution was to find a way of constructing the entire load bearing structure in the ground before excavation commenced and the cellular arch was therefore the result.
For brevity's sake no detailed illustration is given here of the complex stage of study and development that was needed to translate such an innovative idea into a feasible design both from a technical and a financial viewpoint, while guaranteeing the highest levels of safety at all construction stages. Consequently we will move on here directly to an illustration of the construction of Venezia Station.

VIEW AFTER HALF FACE EXCAVATION
MILAN URBAN RAILWAY LINK
"VENEZIA" STATION

THE CONSTRUCTION OF THE CELLULAR ARCH FOR VENEZIA STATION

From an operational viewpoint, two independent construction sites were set up to construct the cellular arch for Venezia Station: one for the side walls and the other for the crown of the tunnel. The drifts for the sidewalls with a 60 sq. m. cross section (7.6 m. in width, 11.0 m. in height) and a length the same as that of the future station tunnel, were constructed in two stages:

1. excavation of 40 sq. m. down to the water table;
2. ground improvement injections under the water table below the future side walls and the tunnel invert and subsequent deepening of the excavation down to the foot of the side walls.

INSTALLATION OF REINFORCEMENT INSIDE A TRANSVERSAL MINITUNNEL
MILAN URBAN RAILWAY LINK
"VENEZIA" STATION

The lining for them consisted of steel ribs, steel mesh reinforcement and shotcrete.
Once completed, the side walls of the future station tunnel were then cast inside them. If excavation times are also included, the 430 metres of the side walls (215 m. on each side) were completed in around eleven months, which was approximately the same time required for pipe jacking during the same period on the other independent construction site.
In this respect the design for Venezia Station involved driving ten minitunnels around the profile of the crown using pipe jacking technology. This involved driving approximately 1,080 pipes for a total length of around 2,160 m. The pipes were prefabricated, manufactured using the radial prestress system and high strength cement mix and had an outer diameter of 2,100 mm, an internal diameter of 1,800 mm. and a length of 2 m.
They were pipe jacked into the ground from a thrust pit (Figure 4). The excavation equipment consisted of a cylindrical metal shield with a diameter of 2,100 mm. and a length of 7.7 m., divided into two parts. The front part for cutting was jointed to allow the operator to guide the vertical and horizontal movement and it was fitted with a computer operated hydraulic cutter and a conveyor belt for mucking out, while the rear part, 3.50 m. in length, contained the motors, pumps and reservoirs for the hydraulic fluid. The thrust equipment included two hydraulic long stroke jacks, the indispensible load distribution structures and a hydraulic pump operating at a pressure of 600 bar.
Two sets of equipment were employed to obtain daily pipe advance rates of approximately 8-9 m. per day. Topographic monitoring carried out during and after pipe jacking ensured and then confirmed that it was performed accurately with negligible deviations in direction and depth. Once the side walls had been cast and all the pipes driven into place, construction of the load bearing cross members of the arch to confine the crown of the future station tunnel was performed, construction of which undoubtedly constituted the most characteristic part of the cellular arch technique.

VIEW OF VENEZIA STATION AFTER COMPLETION OF EXCAVATION UNDER THE PROTECTION OF THE CELLULAR ARCH CROWN

THE MILAN URBAN LINK RAILWAY LINE

The cross members consisted of intermediate arches placed at intervals of 6.00 m. plus the two end pieces. Construction was performed as follows (Figure 5):
1. cutting and removal of the parts of the pipe intersected by the arches and excavation of the arches, mainly by hand, down to the tunnel wall side drifts;
2. assembly of the prefabricated steel forms inside the excavation, placing of the reinforcement for the cells (minitunnels) and the arches and casting of the latter on the side walls already in place;
3. casting of the pipes.
Excavation in steps of the top heading in the crown and the bench of the station tunnel was then able to start in complete safety under the already active load bearing cellular arch structure. The lining was finally completed with an invert varying from 1.5 to 2 m. It was cast in 5 m. steps for a total length of 92 m. and a total average cross section of 38 cu. m./m. Each 5 m. advance took a seven day working week to complete on average and work was co-ordinated so that the ring of the tunnel lining was left unclosed for only three days.

Finally, it is interesting to consider that the average overall production rate for the civil engineering works of Venezia Station, constructed using the cellular arch technique, was 57 cu. m. per day, for a final cost of approximately 516 euro/cu. m, comparable to, if not less than, the current price of an ordinary automobile garage in the centre of the city.

THE MONITORING SYSTEM

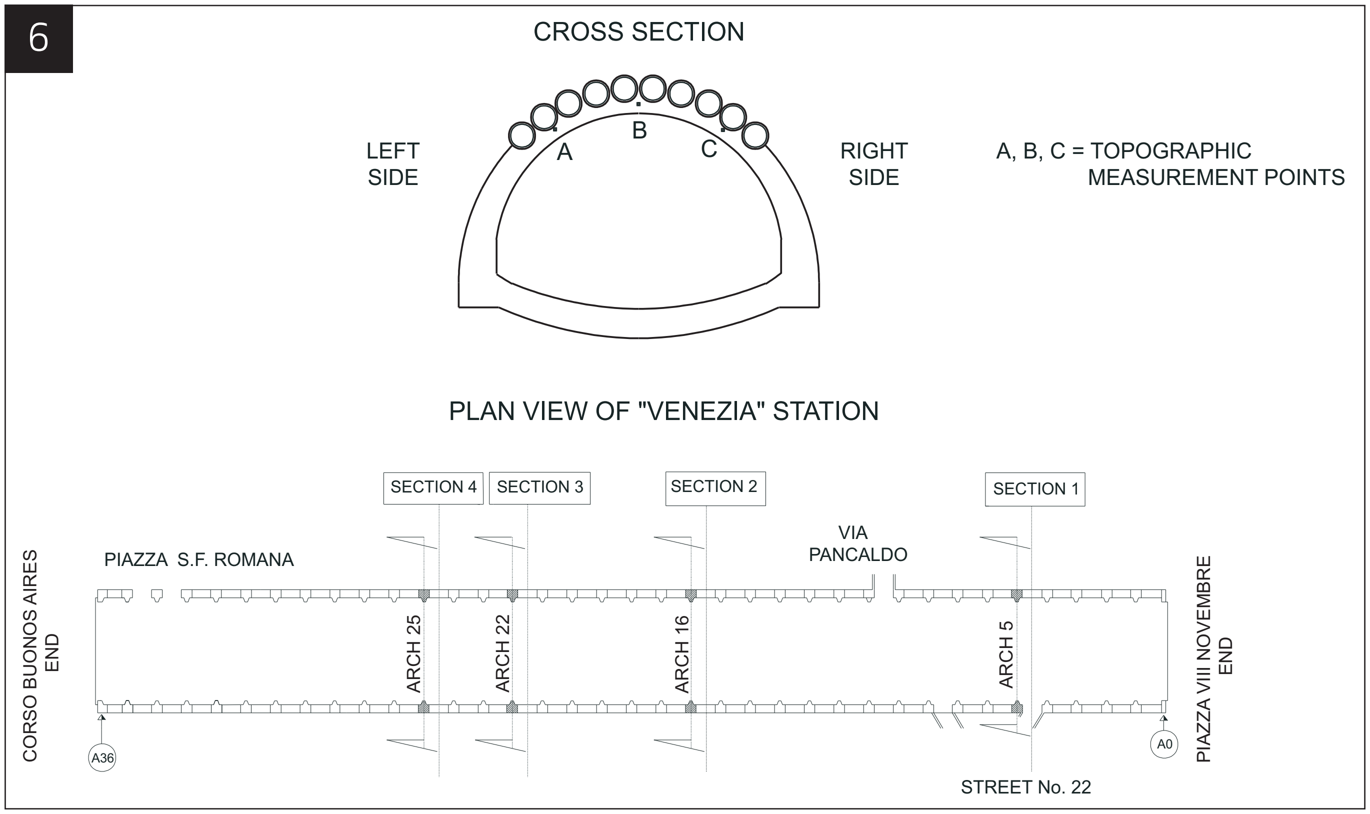
The large dimensions of the cavity, the completely new construction method and the delicate surface constraints meant that a vast monitoring programme had to be designed and implemented to measure:

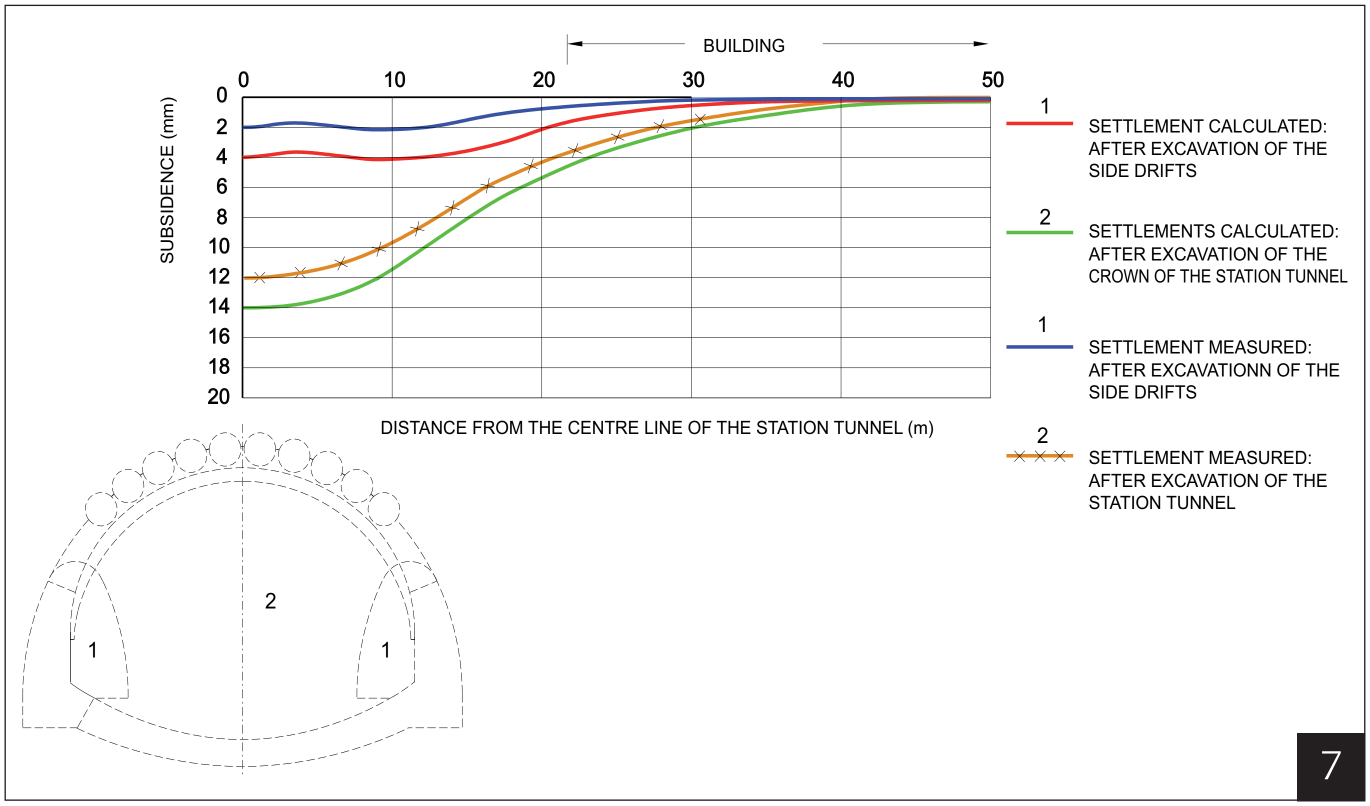
- surface settlements, especially at ground level in front buildings, during all stages of the works;
- deformation of the ground around the tunnels;
- stresses and strains in the final linings of the tunnel.

The programme included:

- topographical measurements (Figure 6);

6. TOPOGRAPHICAL MONITORING
THE MILAN URBAN RAILWAY LINK LINE VENEZIA STATION





7. *CALCULATED AND MEASURED VERTICAL SETTLEMENT AT THE SURFACE*

THE MILAN URBAN RAILWAY LINK LINE VENEZIA STATION

8. *MAXIMUM SUBSIDENCE AT DIFFERENT DEPTHS*

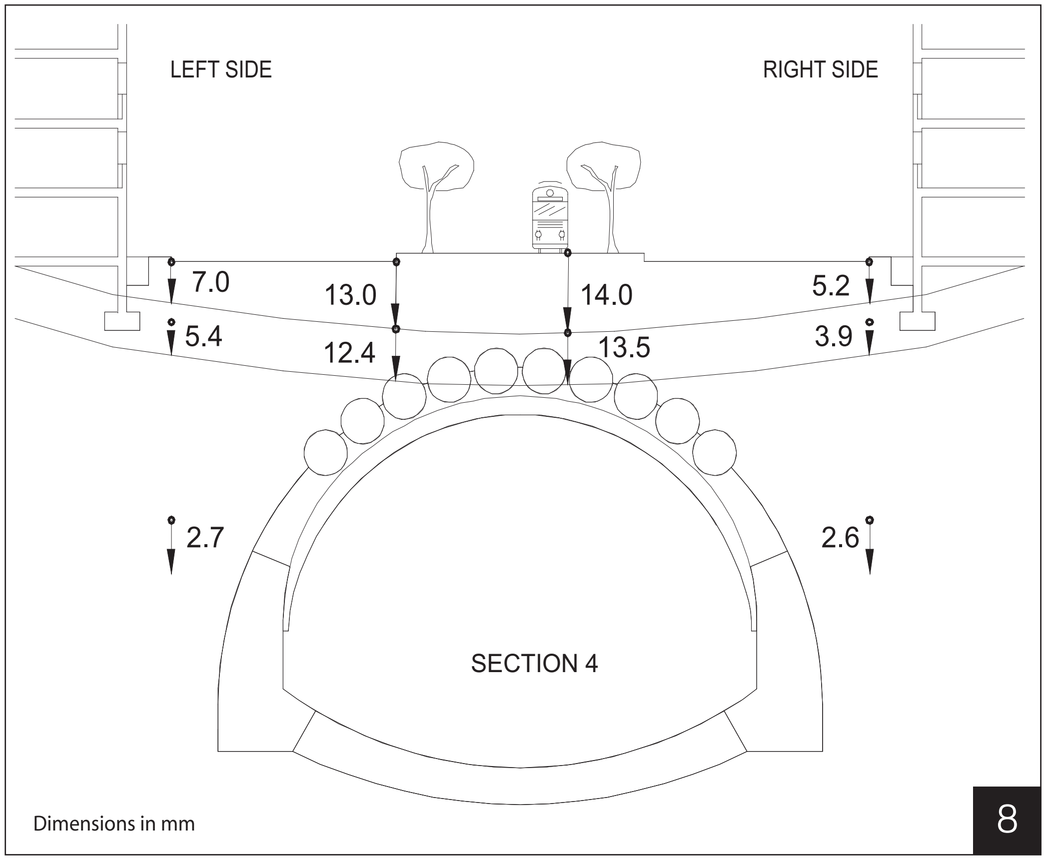
FOR THE ADVANCE IMPROVEMENT STAGE USING INJECTIONS OF CEMENT MIXTURES WHEN THE TUNNEL WAS COMPLETE

▸ levelling, deflectometer and inclinometer measurements to monitor the development of deformation on existing buildings;
▸ convergence measurements in tunnels;
▸ pressure and deformation measurements on the lining structures.

Continuous recording and processing of the various measurements taken enabled the stress-strain conditions of the ground and the lining to be kept under control during the various stages of construction to provide a useful and constant comparison with both the design forecasts and the threshold limits within which existing structures will maintain their integrity.

As shown in Figure 7, surface settlements always remained below the calculated values.

Of course the greatest settlements were observed during excavation of the crown. The increase in deformation, slow at first and then more rapid as soon as the face passed the cross section measured, gradually died down as the face moved further away.

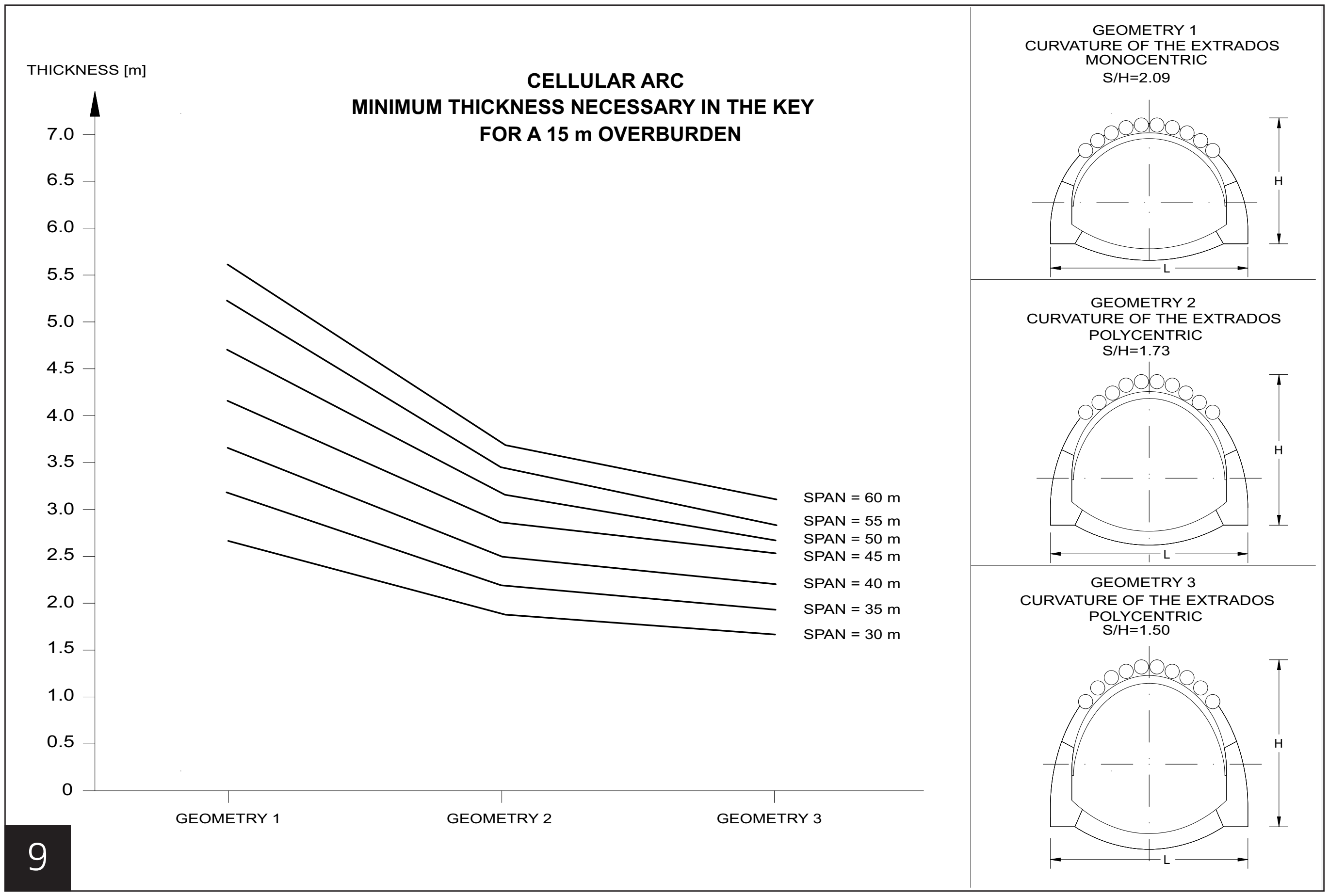


This behaviour shown from the datum points at street level above the route of the tunnel was confirmed, although to a lesser extent, by those located on buildings for which maximum settlement during the passage of the face did not exceed 1-2 mm. (Figure 8).
This monitoring system guaranteed constant surveillance over actual conditions, demonstrating the compatibility between the efficiency of the construction system and the urban environment, to provide a reassuring overall picture.

POSSIBLE DEVELOPMENTS OF THE SYSTEM

As already mentioned, studies were performed to establish the limits of the "Cellular Arch" method for the construction of wide span tunnels in cohesionless soils with shallow overburdens, under the water table. Once a basic outline of the problem had been constructed and the variables determined, approximate dimensions of the main construction elements were calculated using a one dimensional finite element model to simulate the behaviour of the structure and its interaction with the ground. Three different geometries were considered with S:H ratios of 2.09, 1.73 and 1.5, with the span S variable up to 60 m. (Figure 9).
The results of the calculation led to the production of tables giving the minimum thickness of the structural elements and the surface settlement as a function of the geometry, the outer diameter, the depth of the water table and the size of the overburden (see example in Figure 9).
It would appear from those results that the "Cellular Arch" method could be employed with success on the underground construction of shallow overburden tunnels with a span of over 60 m. in cohesionless ground under the water table without any significant surface settlement.

9. MINIMUM THICKNESS IN THE TOP OF THE CROWN AS A FUNCTION OF THE GEOMETRY AND OF THE SPAN FOR A 15 METRE OVERBURDEN



BIBLIOGRAPHY

LUNARDI P., 2008. Design and construction of tunnels - Analysis of Controlled Deformation in Rock and Soils (ADECO-RS), Ed. SPRINGER, 576 pages, March 2008

LUNARDI P., 2007. Progetto e costruzione di gallerie - Analisi delle deformazioni controllate nelle rocce e nei suoli (ADECO-RS), Ed. HOEPLI, 575 pages, August 2007

LUNARDI P., 2000. The construction of large-span stations for underground railways, Tunnel, n. 8 (December)

LUNARDI P., 1999. Construction des stations de grandes portées pour métro, ETH-Tunnelbau-Symposium 99, Zurigo, 18th March 1999

LUNARDI P., 1997. Utilisation du sous-sol urbain en Italie: comparison de deux cas exemplaires: Ligne ferroviaire souterraine de Milan: Stazione Venezia (technique de l'Arc Cellulaire) - Métro de Rome: Stazione Baldo degli Ubaldi (technique du prédécoupage), Tunnels et ouvrages souterrains, n. 141, maggio-giugno 1997

LUNARDI P., 1997. Fermata "Venezia": il progetto e la realizzazione della struttura ad "Arco cellulare", Atti del Convegno "Il Passante Ferroviario di Milano: Infrastrutture e Tecnologie" - C.I.F.I., Milano, 22 maggio 1997

LUNARDI P., 1997. Usages du sous-sol urbain en Italie: confrontation de deux cas significatifs: Ligne ferroviaire souterraine de Milan: Stazione Venezia (technique de l'arc cellulaire) - Métro de Rome: Stazione Baldo degli Ubaldi (technique du prédécoupage mecanique), Atti delle Journées d'étude "La ville souterraine 2" - Paris, 19-20 marzo 1997

LUNARDI P., 1996. Construction Technologies for Wide Span Tunnels. A comparison of Methods, 45th Geomechanics Colloquium - Salisburgo, 10-11 ottobre 1996

LUNARDI P. E ALTRI, 1994. The Venezia Station of Milan Railway Link carried out by the Cellular Arch Method: water proofing, fire proofing and safety, ventilation, International Conference "Underground openings for public use" - Oslo, 14-17 Giugno 1994

LUNARDI P., 1993. Evolution des technologies d'excavation en souterrain, dans des terrains meubles, Colloque chez le "Comité Marocain des Grands Barrages" - Rabat, 30 settembre 1993

LUNARDI P. E ALTRI, 1993. Performance observations during construction of the large span Milan metro (Venezia) station, International Congress "Option for tunnelling" - Amsterdam, 19-22 Aprile 1993

LUNARDI P., 1993. Aspetti progettuali e costruttivi nella realizzazione di gallerie in situazioni difficili. Interventi di precontenimento del cavo, Le strade, marzo-aprile 1993

LUNARDI P. E ALTRI, 1993. Tunnelling in soil: design, construction materials and monitoring, Seminario Russo-Italiano - Central Institute for Advanced Training of Construction Engineers (TSMIPKS), Mosca, 11th March 1993

LUNARDI P., 1992. Cellular Arch technique for large span station cavern, Tunnels & Tunnelling, novembre 1991

LUNARDI P. E ALTRI, 1991. Un metodo costruttivo conservativo e di precontenimento del cavo per grandi cavità in terreni difficili, Convegno Internazionale su "Il consolidamento del suolo e delle rocce nelle realizzazioni in sotterraneo" - Milano 18-20 Marzo 1991

LUNARDI P., 1991. Aspetti progettuali e costruttivi nella realizzazione di gallerie in situazioni difficili: interventi di precontenimento del cavo, Convegno Internazionale su "Il consolidamento del suolo e delle rocce nelle realizzazioni in sotterraneo" - Milano 18-20 Marzo 1991

LUNARDI P., 1990. The Cellular Arch Method: technical solution for the construction of the Milan Railway's Venezia Station, Tunnelling and Underground Space Technology, Settembre 1990

LUNARDI P. E ALTRI, 1990. Soft ground tunnelling in the Milan Metro and Milan Railway Link. Case histories, Soft Ground Tunnelling Course - Institution of Civil Engineers - Londra 10-12 Luglio 1990

LUNARDI P., 1990. Un noveau systeme constructif pour la realisation de tunnel de grande portee dans terrains non coherents: l'"Arc Cellulaire", Colloque International "Les fondations des grands ouvrages" - UNESCO - Parigi 27-29 Marzo 1990

LUNARDI P., 1990. Un nuovo sistema costruttivo per la realizzazione di gallerie di grande luce in terreni sciolti: l'"Arco Cellulare", Quarry and Construction, n. 2 anno 1990

COLOMBO A., LUNARDI P., LUONGO E., 1989. Innovative techniques for the realization of large diameter tunnels in loose land, Congresso Internazionale su "Progress and Innovation in Tunnelling" - Toronto 9-14 Settembre 1989

LUNARDI P., 1989. Un nuovo sistema costruttivo per la realizzazione di gallerie di grande luce in terreni sciolti: l'"Arco Cellulare", Gallerie e grandi opere sotterranee, n. 29 anno 1989

LUNARDI P. E ALTRI, 1989. L'"Arc Cellulaire" pour la Station "Venezia" du "Passante Ferroviario Metropolitano" de Milan, Colloque International "Tunnels et micro-tunnels en terrain meuble - Parigi 7-10 Febbraio 1989

LUNARDI P. E ALTRI, 1988. L'impiego dell'"Arco Cellulare" nella realizzazione della stazione Venezia del Passante Ferroviario Metropolitano di Milano, Costruzioni, dicembre 1988

LUNARDI P., 1988. Progresso tecnologico nell'utilizzo degli spazi sotterranei, Atti del Convegno Nazionale su "Lasciamo all'uomo la luce del sole, sprofondiamo i servizi" - Milano, 8-9 Novembre 1988

LUNARDI P. E ALTRI, 1988. L'emploi de l'"Arc Cellulaire" dans la realisation de la Station "Venezia" du "Passante Ferroviario Metropo-litano" de Milan, Atti del Congresso Internazionale su "Le gallerie e l'Acqua" - Madrid, 12-15 Giugno 1988

LUNARDI P., 1988. L'emploi de microtunnels pour la realisation d'ouvrage souterrains de grande portee: l'"Arc Cellulaire", Atti del Congresso Internazionale su "Le gallerie e l'Acqua" - Madrid, 12-15 Giugno 1988

1995

ARTIFICIAL GROUND OVERBURDENS

For underground excavation even in the absence of an overburden

THE FACE OF THE BORGO RINZELLI TUNNEL WHILE ADVANCING UNDER THE ARTIFICIAL GROUND

BOLOGNA–FLORENCE HIGH SPEED STATE RAILWAY LINE
∅ = 13.5 M.
GROUND: CLAY
OVERBURDEN: NIL

"Artificial ground" technology was proposed by Prof. Ing. Pietro Lunardi and developed by Rocksoil S.p.A. in 1995, in order to industrialise the excavation of tunnels under low or nil overburden conditions.
The conventional solution for artificial ground involves making a deep cut into slopes to be crossed with consequent serious problems of:

- stability of the slopes themselves;
- what to do with the huge volumes of material excavated;
- solving the problem of any interference there may be with existing surface structures;

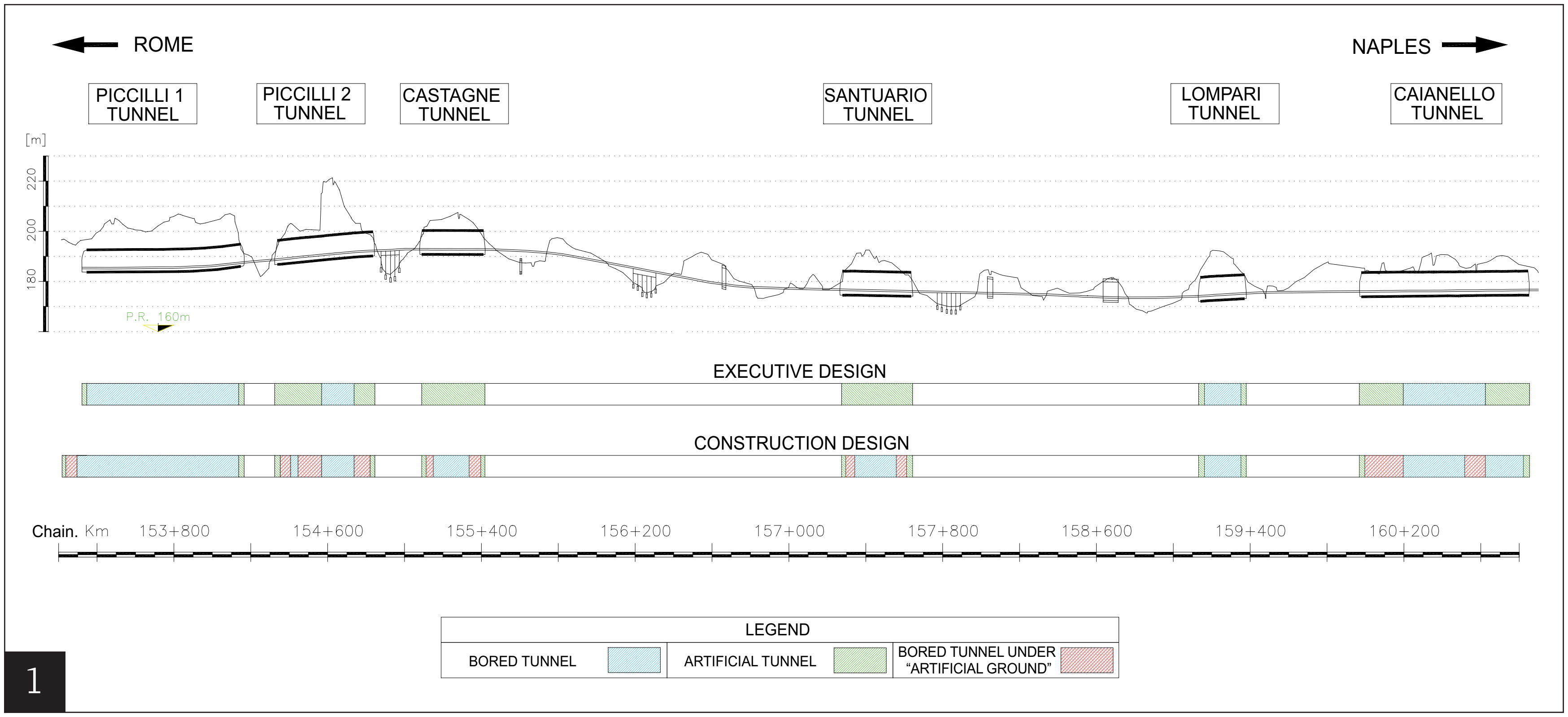
- vulnerability of the tunnel to seismic activity;
- environmental and landscape impact.

And that is without mentioning the costs resulting from the organisation of the relative construction site when low overburdens involve just short sections of a tunnel to be driven mainly fully underground. On the other hand, the alternative of improving an appropriate band of ground around the future excavation, in order to confer upon it the indispensable properties of strength and deformability to be able to then proceed to tunnel underground in safety is often not possible because the indispensable minimum thickness of ground over the crown is lacking.
In order to overcome those problems, in 1995, after accepting an engagement to design a series of tunnels with sections characterised by very shallow overburdens on the new high speed/high capacity Rome-Naples line, Rocksoil started to study the possibilities of an alternative solution, which in some way would enable tunnels to be constructed entirely fully underground, thereby by-passing all the problems that would have been encountered in the implementation without changes of the original conventional final design.
The idea of "artificial ground' began to form.

ARTIFICIAL GROUND

The "artificial ground" method is used to replace the ground located over the crown of a tunnel with structural "artificial" elements obtained by using the ground in the overburden after treating it adequately first. Figure 2 illustrates the various operational stages of the methodology.
After first having removed the surface layer of the ground over the tunnel to be constructed, by following the profile of the crown with a tolerance of 10 cm. down to the springline level and having backfilled (where necessary) according to the geometry shown in the figure, a 10 cm. layer of steel mesh (6 ∅ 15 x 15 cm.) reinforced shotcrete is then sprayed on the surface of the excavation shaped in this way with the function of:

- shaping the future tunnel;
- distributing the loads that will weigh on the crown.

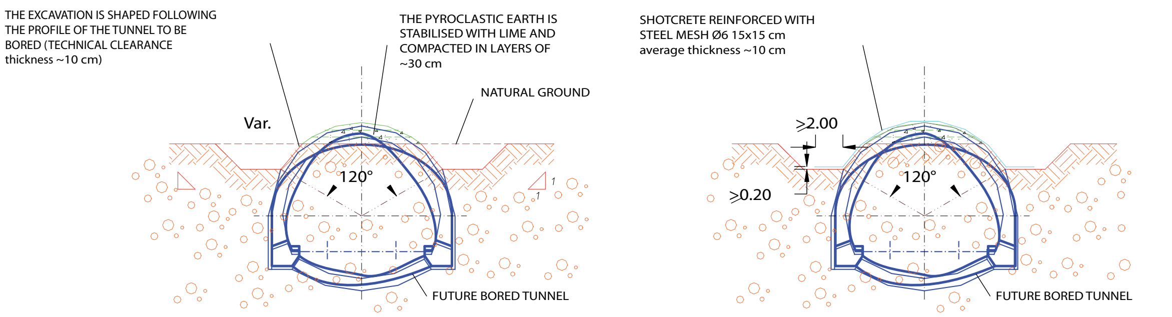


1. ADAPTATION OF THE CONSTRUCTION DESIGN FOLLOWING THE ADOPTION OF THE "ARTIFICIAL GROUND" METHOD

HIGH SPEED STATE RAILWAY LINE ROME-NAPLES

2. THE DIFFERENT OPERATIONAL STAGES OF THE METHODOLOGY

BY EXERTING CLEAR PREVENTATIVE ACTION TO PROTECT THE CORE-FACE OF THE FUTURE TUNNEL, THE ARTIFICIAL GROUND METHOD IS CLEARLY CLASSIFIABLE IN TERMS OF THE ADECO-RS APPROACH AMONG INDIRECT CONSERVATIVE TECHNIQUES FOR CAVITY PRECONFINEMENT

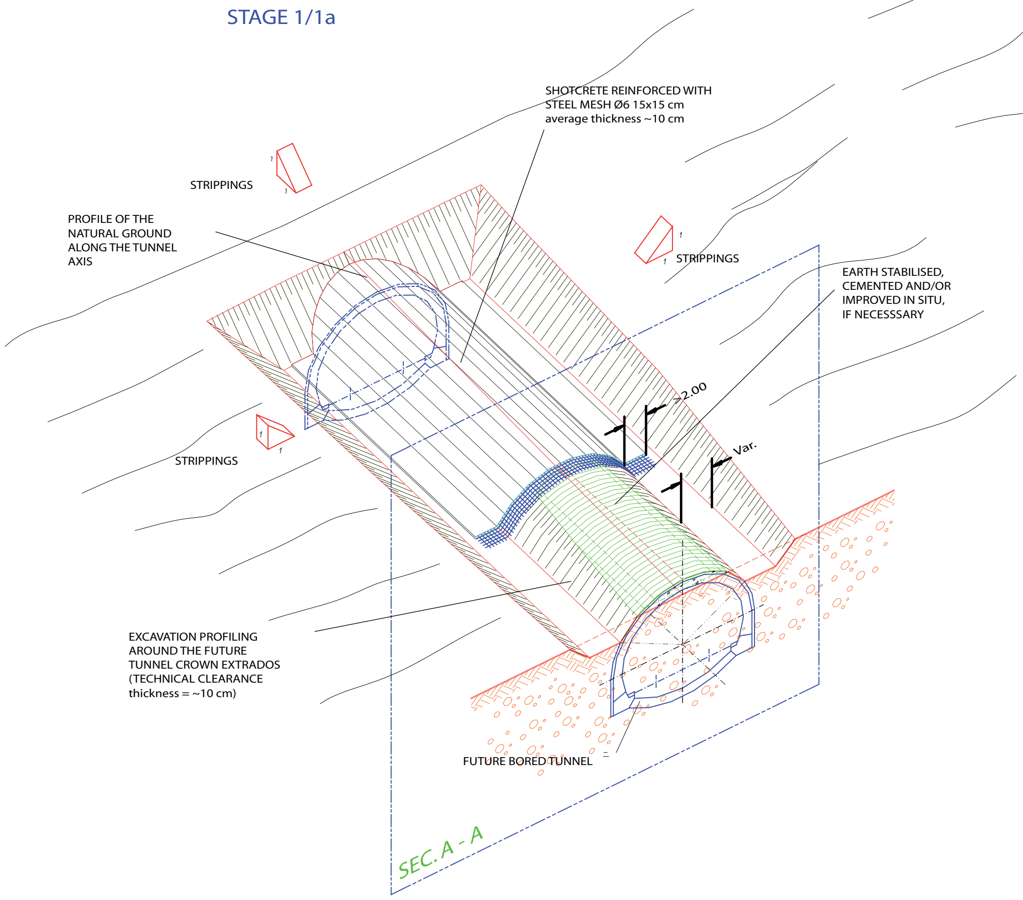


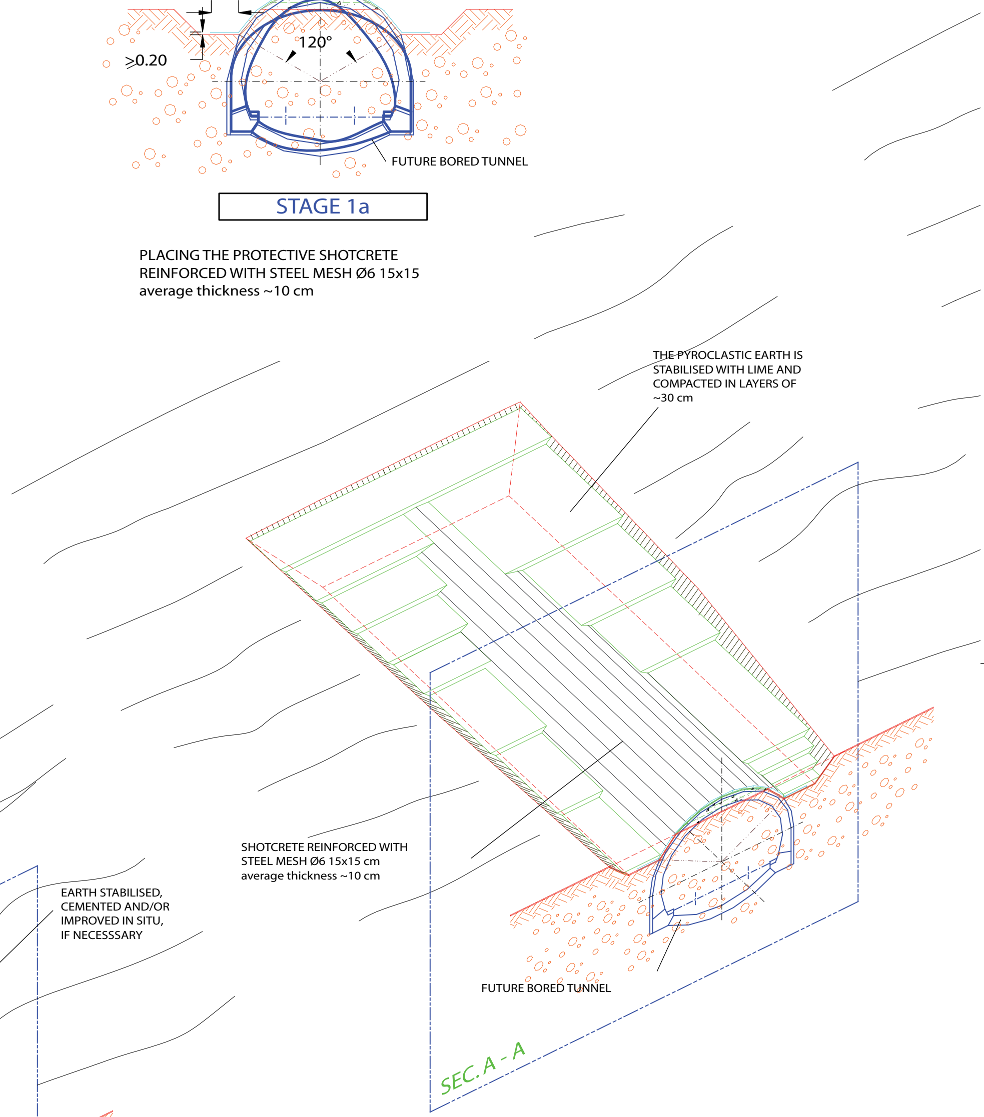
EMBANKMENT AND SHAPING OF THE EXCAVATION FOLLOWING THE PROFILE OF THE TUNNEL TO BE BORED (TECHNICAL CLEARANCE thikness ~10 cm)

PLACING THE PROTECTIVE SHOTCRETE REINFORCED WITH STEEL MESH Ø6 15x15 average thickness ~10 cm

AXONOMETRIC VIEW

STAGE 1/1a

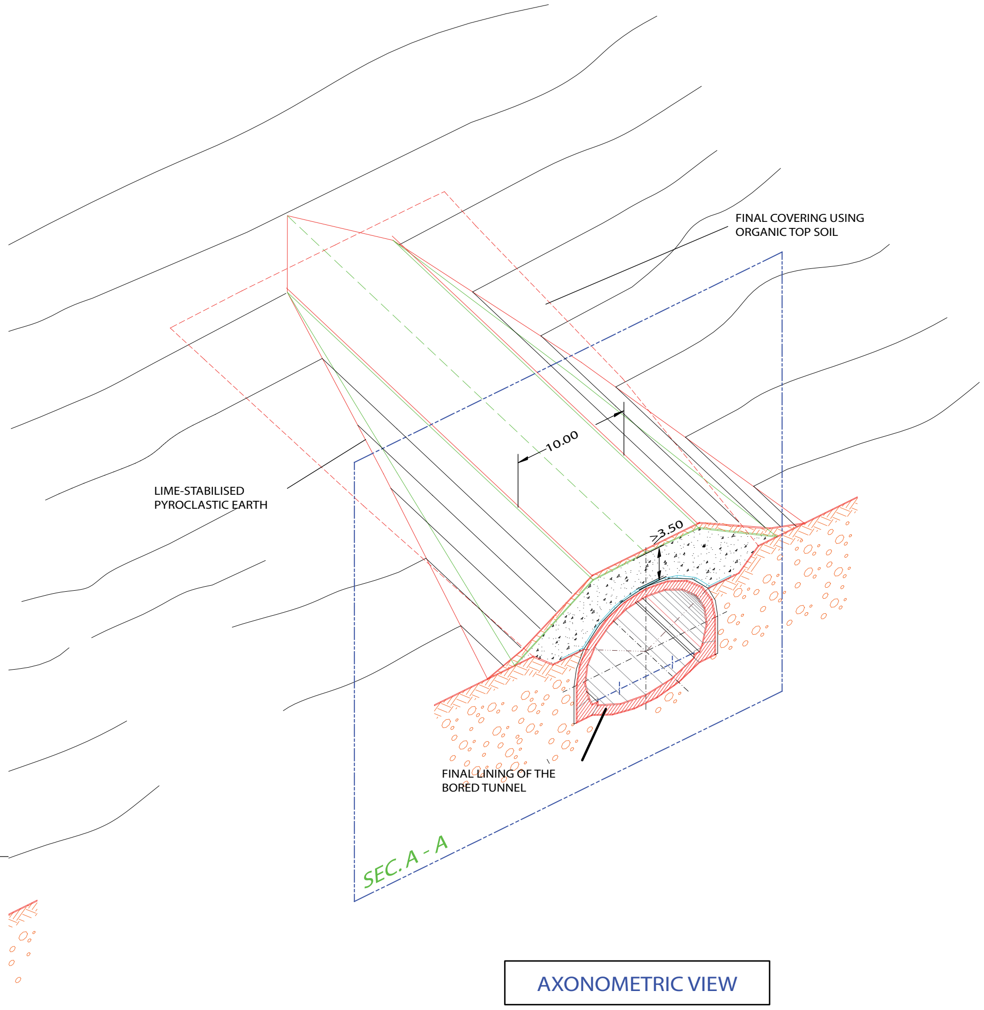




AXONOMETRIC VIEW

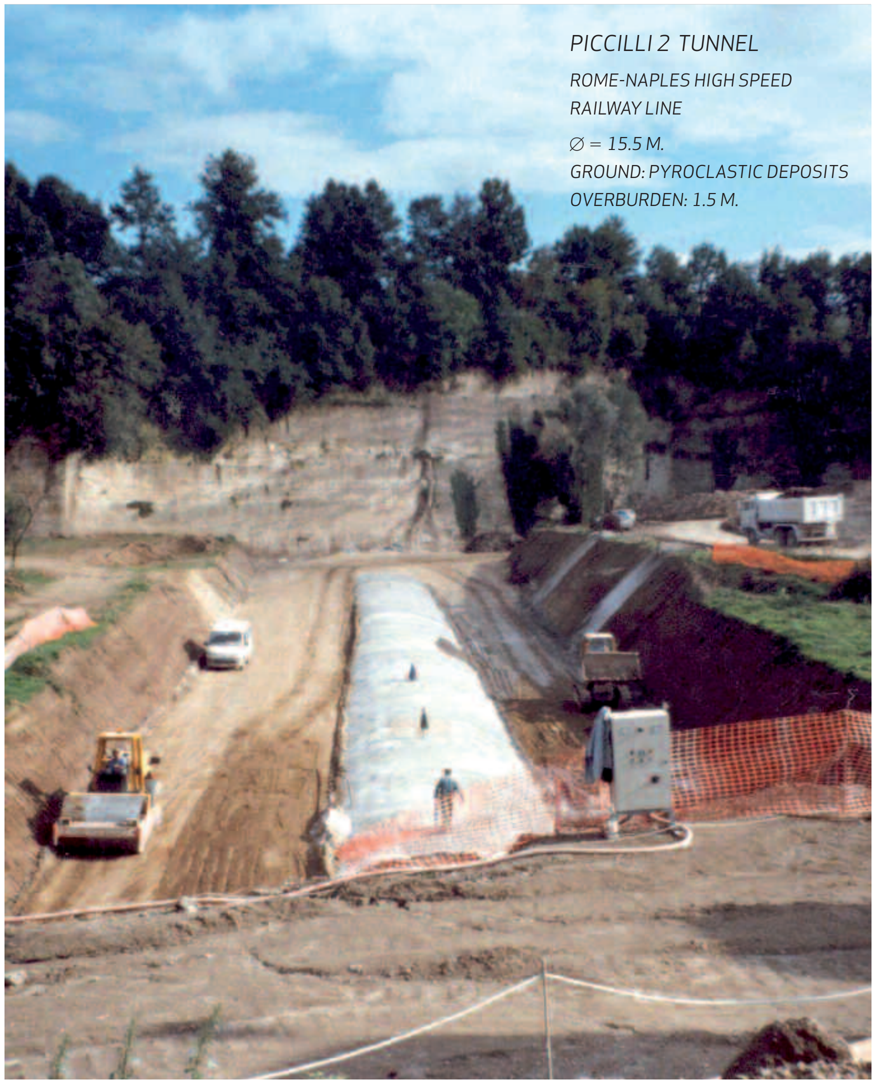
STAGE 2

2

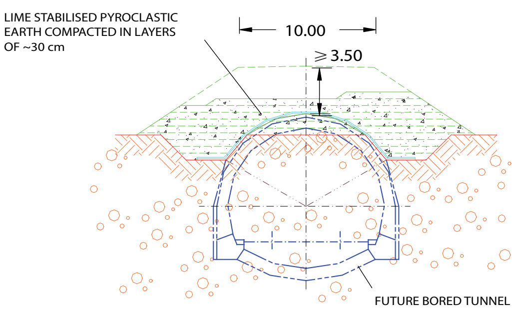


AXONOMETRIC VIEW

STAGE 3/4



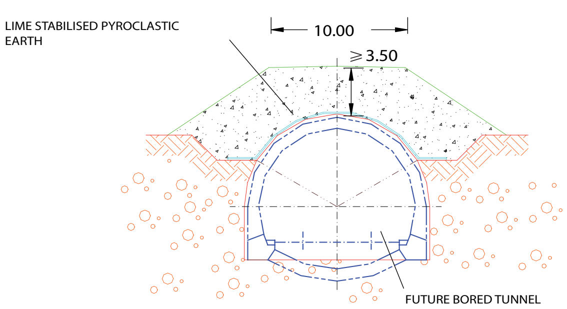
SECTION A-A



STAGE 2

BACKFILL USING A MIX OF PYROCLASTIC EARTH STABILISED WITH LIME AND COMPACTED IN LAYERS OF OF ~30 cm TO FORM A THICKNESS OVER THE CROWN OF 3.50 m (Min.)

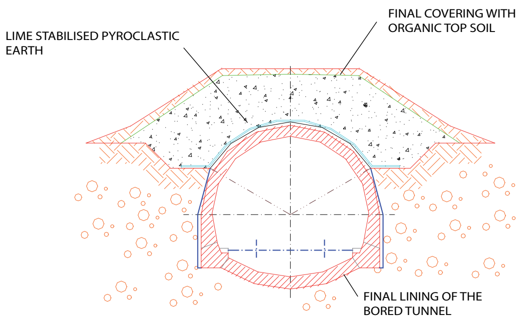
SECTION A-A



STAGE 3

THE TUNNEL IS BORED AND A PRELIMINARY LINING OF SHOTCRETE (thickness ~20 cm) AND STEEL RIBS ARE PLACED

SECTION A-A



STAGE 4

THE FINAL LINING OF THE BORED TUNNEL IS PLACED AND THE FINAL COVERING WITH ORGANIC TOP SOIL IS PLACED

At this point the area is filled in and embanked with the same ground previously removed, after treating it appropriately to increase its strength and rigidity as required, until the crown of the future tunnel is covered in individually compacted 30 cm. layers to a total thickness of at least 3.5 m.

When this is completed, underground excavation under the crown can begin.

By exerting clear preventative action to protect the core-face of the future tunnel, the "artificial ground" method is clearly classifiable among indirect conservative techniques for cavity preconfinement.

THE APPLICATION OF THE METHOD ON THE TUNNELS OF THE NEW ROME-NAPLES RAILWAY LINE

The application of new methods of driving tunnels always requires very thorough preliminary studies to assess whether it is genuinely feasible and compatible with the specific local conditions.

In the case of "artificial ground", these studies primarily regarded the soils found in situ in which construction would have to take place, in order to select the most appropriate type of ground treatment and to test the effects in advance.

3. *SHAPING THE CROWN UNDER THE ARTIFICIAL GROUND*

4. *LABORATORY STRENGTH TESTS ON A SAMPLE OF STABILISED GROUND*

THE GEOLOGY AND THE SURVEY CAMPAIGN

From a geological viewpoint, the route of the tunnels on the high speed/high capacity Rome-Naples railway line borders on the Roccamonfina volcano system and only marginally intersects the lava deposits, running almost entirely through eruption material.

The various lithotypes consist of successions of stratified pyroclastites (tuffs) sub-horizontally positioned with granulometry varying from sandy silt with inclusions of gravel to coarse sand, to give a morphology characterised by undulating hills with valley depressions originated by seasonal streams.

A geological survey campaign was conducted consisting of continuous core bore sampling and exploration trenches (one every 7 – 10 m.) to assess the quality of the weathered material to be improved before placing the "artificial ground". The main geophysical characteristics of the material were measured in the laboratory: weight by volume, granulometry and consistency limits, in view of the subsequent study on the treatment with lime, and modified ASSHTO compaction tests were also performed.

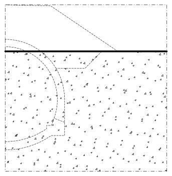
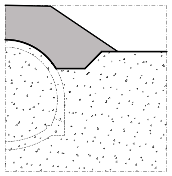
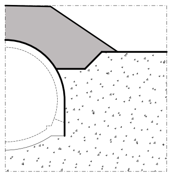
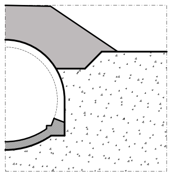
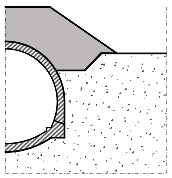
Samples of the ground then underwent strength tests. They were first mixed with lime in percent-

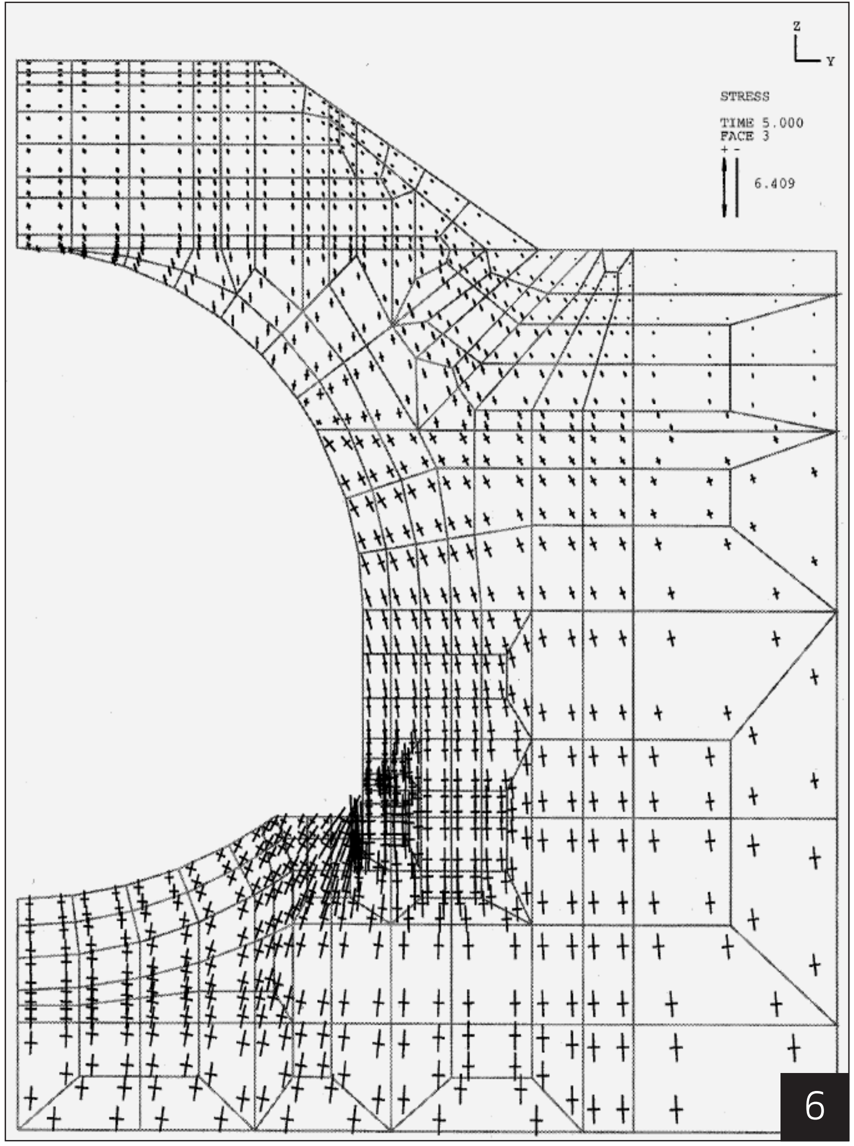
5. *STEPS IN THE FINITE ELEMENT CALCULATION*

6. *GRAPH OUTPUT OF THE F.E.M. CALCULATIONS:*

THE PRINCIPAL STRESSES IN THE GROUND AROUND THE TUNNEL CALCULATED AT TIME FIVE

5

TIME	MODEL	DESCRIPTION
1		CALCULATION OF THE GEOSTATICS
2		IMPROVEMENT AND BACKFILL WITH PYROCLASTITES TREATED WITH LIME
3		THE TUNNEL IS DRIVEN FULL FACE AND THE PRELIMINARY LINING OF SHOTCRETE AND STEEL RIBS IS PLACED RELAXATION OF THE CORE 70%
4		THE TUNNEL INVERT AND KICKERS ARE CAST RELAXATION OF THE CORE: 80%
5	"	CURING OF THE TUNNEL INVERT RELAXATION OF THE CORE: 90%
6		THE TUNNEL ARCH IS CAST AND THE GEOMECHANICAL CHARACTERISTICS OF THE GROUND TREATED WITH LIME FALL RELAXATION OF THE CORE: 100%
7	"	SEISMIC ACTION



6

BACKFILL WITH STABILISED GROUND AND COMPACTION

PICCILLI 2 TUNNEL, ROME-NAPLES HIGH SPEED RAILWAY LINE ages varying from 3% to 5%, prepared using the optimum humidity found with the modified AASHTO test. They were then left to harden in a saturated water vapour environment. The findings of the experiments showed that percentages of 3-4% of lime were the best to use. Finally the laboratory tests were verified by means of field tests, which produced results in line with expectations.

STATICS TESTS

Finally the feasibility of the new solution and the reliability of the statics were tested by means of finite element numerical analysis conducted in the non linear field using version 6.0 of the ADINA software application.

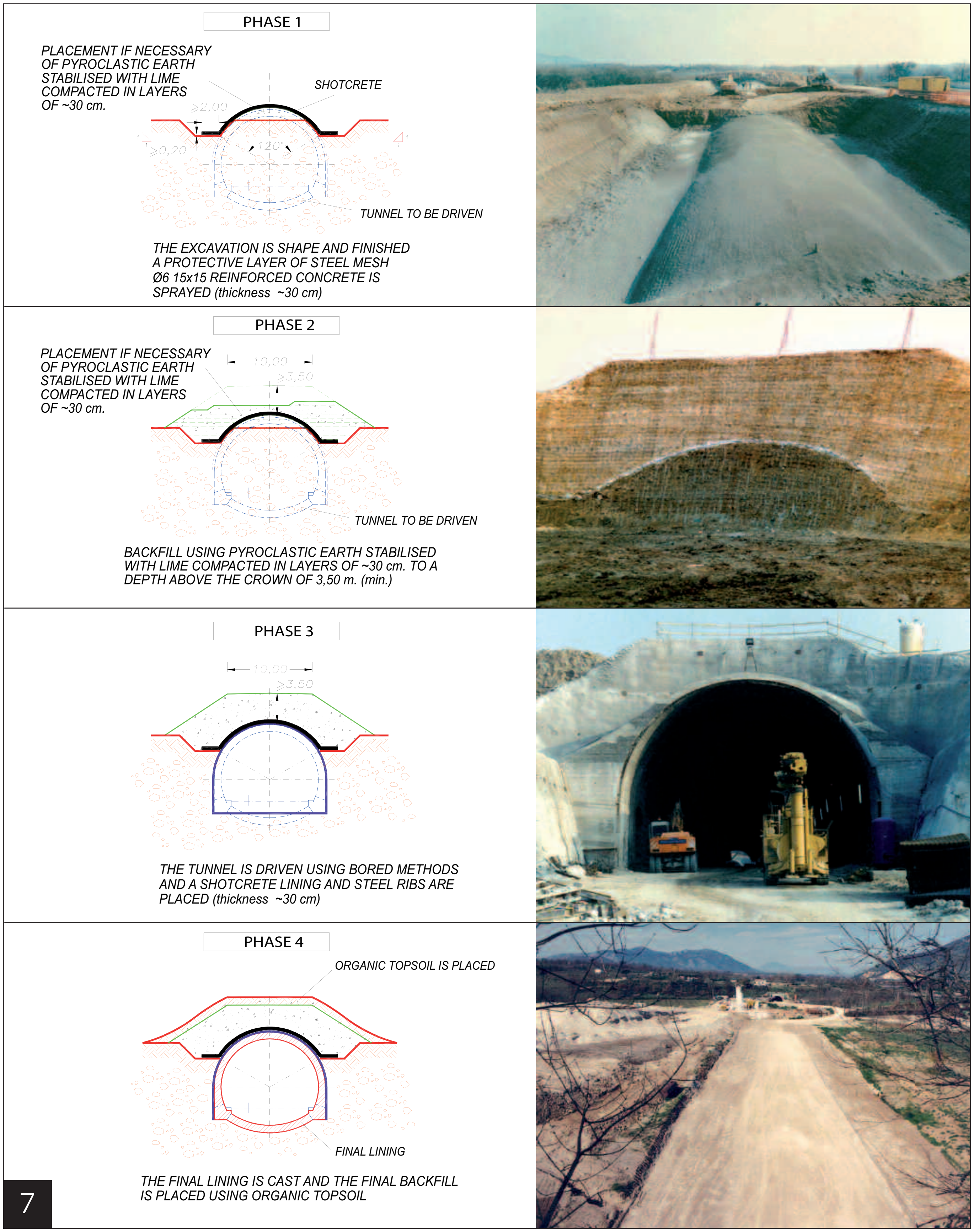
Because the analysis was intended to study the stress-strain behaviour of the entire ground-structure system at the different stages of construction and when in service, seven calculation 'times' were performed (Figure 5) to model the succession of those stages and the implementation of stabilisation intervention during construction work as realistically as possible.

The results of the calculation confirmed that full-face underground tunnel advance was possible under the protection of the "artificial ground" treated with lime, by employing the geometry and construction methods illustrated. From the viewpoint of deformation, they gave very low values for convergence (less than 2 mm.), while the role played by the treatment in

SHAPING THE EXTRADOS OF THE FUTURE TUNNEL

PICCILLI TUNNEL 1
ROME-NAPLES HIGH SPEED STATE RAILWAY LINE

7. *I ILLUSTRATED SUMMARY OF THE CONSTRUCTION STAGES*



the crown of the tunnel before excavation started was very important from a stress viewpoint. Thanks to its arched shape, the improved ground was subject to contained compressive stress action only, which was appropriately channelled around the tunnel and transmitted to the natural ground on the sides of the tunnel (Figure 6).
Finally, the preliminary and final linings were thoroughly tested.

CONSTRUCTION

Once it had been ascertained that the solution studied was feasible and that the statics were reliable, experimental implementation began following the procedures and stages already described and illustrated in Figure 2.

Work commenced from the Picilli 2 tunnel where underground tunnel advance was performed under the protection of the "artificial ground" for more than 250 m.
The photographs in Figure 7 illustrate some of the different stages of the work.
A series of measurements were taken systematically during construction to ensure that it complied with the design predictions.
The measurements were as follows:

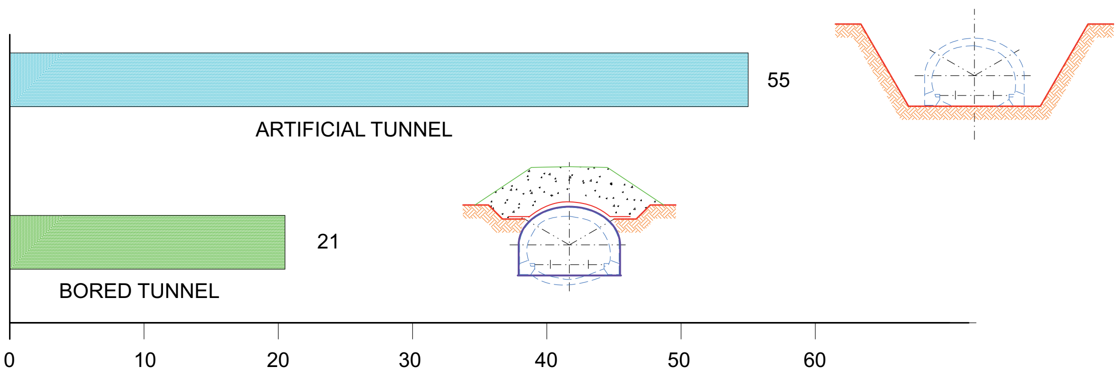
- for the pozzolana-lime mix: the average compressive strength after 7 days of hardening and the density reached in situ compared to the maximum density achieved in the laboratory;
- for the shotcrete: average compressive strength after 48 hours and after 28 days.

COMPARISON OF CONSTRUCTION COSTS AND TIMES FOR BORED TUNNELS DRIVEN THROUGH ARTIFICIAL GROUND OVERBURDEN (A.G.O.) AND FOR THE SAME TUNNELS CONSTRUCTED ARTIFICIALLY

WORK *	ARTIFICIAL TUNNEL €/m	BORED TUNNEL UNDER A.G.O. €/m	DIFFERENCE
EARTH REMOVAL, GROUND IMPROVEMENT, TUNNEL EXCAVATION	2,464.00	4,450.00	-1,986.00
BACKFILL	915.00	0.00	915.00
LINING OF CROWN, KICKERS AND SIDEWALLS	3,421.00	2,383.00	1,038.00
TUNNEL INVERT	1,664.00	790.00	874.00
TOTALS	8,464.00	7,623.00	841.00

*ONLY THOSE ITEMS OF THE TWO CONSTRUCTION METHODS WHICH ARE DIFFERENT

TIME EMPLOYED FOR THE CAIANELLO TUNNEL (FOR 365 m) WITH THE SAME MEANS EMPLOYED



8

8. *THE "ARTIFICIAL GROUND" METHODOLOGY WAS FOUND TO BE EXTREMELY PRACTICAL, SAFE AND ADVANTAGEOUS*

BOTH FROM A CONSTRUCTION AND A FINANCIAL VIEWPOINT AS WELL AS IN TERMS OF THE ENVIRONMENT AND THE LANDSCAPE

Deformation phenomena were also monitored systematically and the results were very close to those predicted by the FEM calculations.
Since the work was performed smoothly without problems of any type, use of the solution was extended to the other tunnels with similar problems.

SPREAD OF THE METHODOLOGY

The "artificial ground" methodology studied and experimented was found to be extremely practical, safe and advantageous when applied in the field, both from a construction and a financial viewpoint (Figure 8) as well as in terms of the environment and the landscape.
Consequently it has been used increasingly more frequently in preference to conventional methods for tunnels with little or no overburden.
Table 1 summaries the data on the use of "artificial ground" in Italy, from the first experimentation until today: a total of 1,346 metres of tunnel have been profitably constructed using this system.

__TABLE 1.__ THE USE OF "ARTIFICIAL GROUND" IN ITALY
FROM THE FIRST EXPERIMENTATION UNTIL TODAY

Tunnel	Tunnel total length [m]	Length of tunnel driven under "artificial ground" [m]
Piccilli 1 (Rome-Naples High Speed Railway Line)	907	58
Piccilli 2 (Rome-Naples High Speed Railway Line)	485	251
Castagne (Rome-Naples High Speed Railway Line)	289	73
Santuario (Rome-Naples High Speed Railway Line)	322	80
Caianello (Rome-Naples High Speed Railway Line)	832	363
Sadurano (Bologna-Florence High Speed Railway Line)	3767	68
Borgo Rinzelli (Bologna-Florence High Speed Railway Line)	528	73
Morticine (Bologna-Florence High Speed Railway Line)	654	380
Total length driven under "artificial ground"		1346

BORGO RINZELLI TUNNEL
BOLOGNA–FLORENCE HIGH SPEED STATE RAILWAY LINE
∅ = 13.5 M.
GROUND: CLAY
OVERBURDEN: 1 M.

VIEW OF THE SPRAYED CROWN OF THE FUTURE BORED TUNNEL

SADURANO TUNNEL BOLOGNA–FLORENCE HIGH SPEED STATE RAILWAY LINE

BIBLIOGRAPHY

LUNARDI P., 2008. Design and construction of tunnels - Analysis of Controlled Deformation in Rock and Soils (ADECO-RS), Ed. SPRINGER, 576 pagine, marzo 2008

LUNARDI P., 2007. Progetto e costruzione di gallerie - Analisi delle deformazioni controllate nelle rocce e nei suoli (ADECO-RS), Ed. HOEPLI, 575 pagine, agosto 2007

LUNARDI P., BELLOCCHIO A., ROMANO G., 2003. Galleria Borgo Rinzelli: studi per l'applicazione del sistema della protesi strutturale in terreni limo-argillosi, Gallerie e grandi opere sotterranee, n. 71, anno 2003

LUNARDI P., 2000. The ADECO-RS approach in the design and construction of the underground works of Rome to Naples High Speed Railway Line: a comparison between final design specification, construction design and "as built", Atti del Congresso Internazionale su "Progress in Tunnelling after 2000", Milano, 10 - 13 giugno 2001

LUNARDI P. E ALTRI, 1997. Una soluzione innovativa per la realizzazione di gallerie naturali senza copertura, Quarry and Construction, maggio 1997

2007

THE NAZZANO METHOD

It is no longer necessary to interrupt service to widen a tunnel

The "Nazzano Method" is a surprising construction invention, which enables the capacity of an infrastructure to be increased where tunnels are involved, while traffic is flowing, without the need to construct new tunnels and without diminishing the quality of services for users.

The idea originated from a shrewd insight by Prof. Ing. Pietro Lunardi in 1999 and was developed by Rocksoil in the years that followed to be then tried successfully for the first time in the world at Nazzano (Rome), to widen a major motorway tunnel without interrupting traffic.

1. THE "NAZZANO METHOD" MAKES IT POSSIBLE FOR THE FIRST TIME TO WIDEN ROAD, MOTORWAY AND RAIL TUNNELS IN SERVICE WITHOUT INTERRUPTING TRAFFIC DURING THE WORKS

THE STEEL TRAFFIC
PROTECTION SHELL

"NAZZANO" TUNNEL, 2007
MILAN-ROME A1 MOTORWAY

∅ = 21 M.
GROUND: SANDS
OVERBURDEN: ~ 45 M.

2. THE "NAZZANO METHOD" MAKES IT POSSIBLE FOR THE FIRST TIME TO WIDEN ROAD, MOTORWAY AND RAIL TUNNELS IN SERVICE WITHOUT INTERRUPTING TRAFFIC DURING THE WORKS

It was without doubt the most exciting news seen in the world of tunnelling for around ten years in these parts, a real achievement which provided public administrations and operators in the sector with an extraordinarily effective solution to give a concrete answer to growing demands for intervention in the transport infrastructure sector. With constantly increasing traffic the need to expand existing road, motorway and rail infrastructures is arising ever more frequently. To satisfy this need, however, is anything but simple when routes run through tunnels, because it is indispensible, in the absence of appropriate technologies, to resort to costly new routes to drive new bores in addition to the existing tunnels.
On the other hand, to widen a tunnel while it is in service is only possible if one is able to:
- guarantee the necessary safety of users, while keeping the inconvenience within acceptable limits;
- solve the technical problems associated with advance of the widening face in a ground that has already been disturbed by the previous excavation;
- create strong structures as existing structures are gradually demolished, by treating all stress-strain conditions, including unexpected conditions, which might arise during construction works in a manner which will guarantee safety for tunnel users and also for human activities that might be performed on the surface.

The "Nazzano Method" satisfies all these requirements and for the first time makes it possible to widen road, motorway and rail tunnels in service without interrupting traffic during works (Figures 1 and 2).

THE HISTORY OF THE TECHNOLOGY

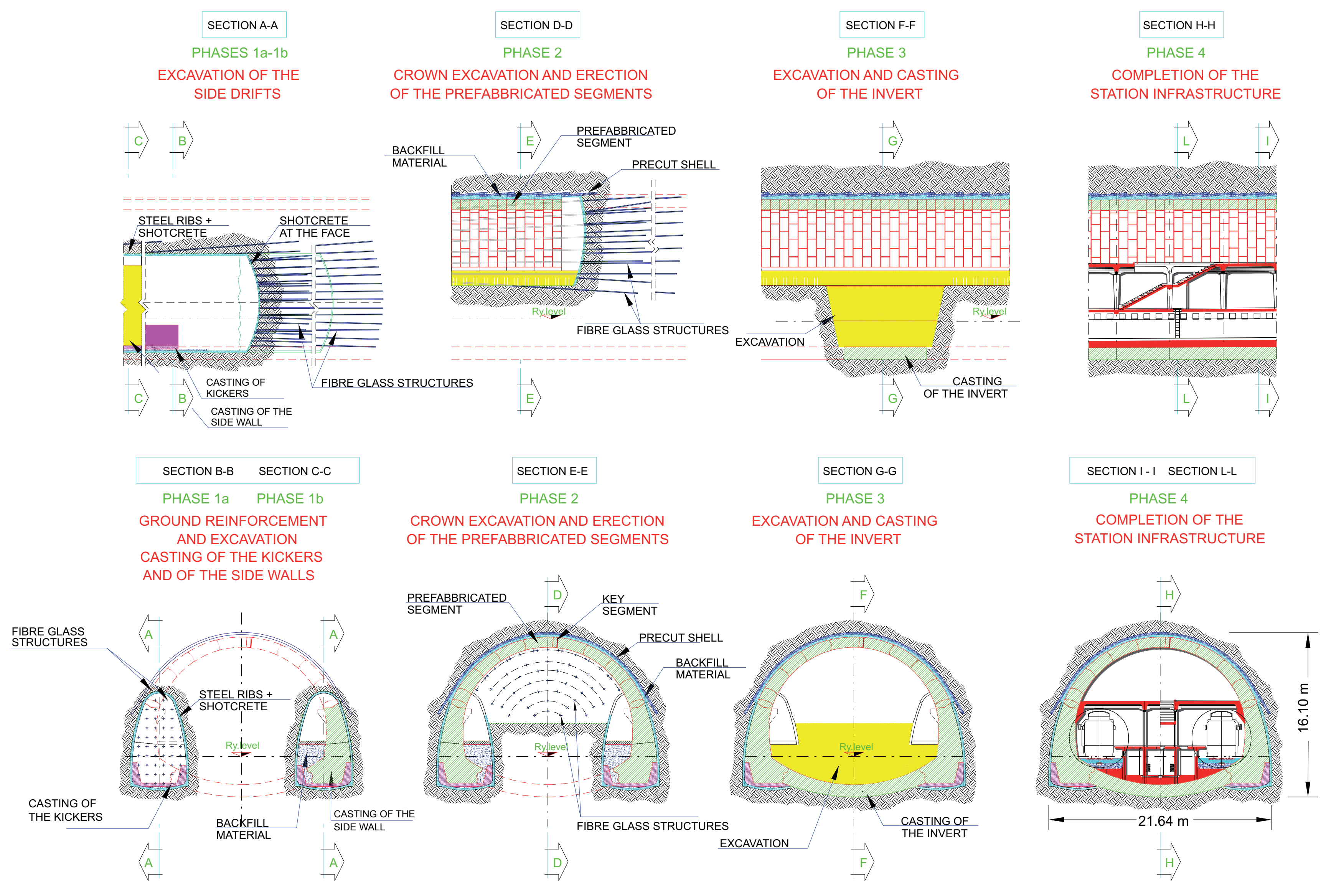
The conviction that it was possible to develop a construction technology to achieve the aim of widening existing tunnels without interfering with traffic began to take shape in the mind of Prof. Ing. Pietro Lunardi at the time of the construction of the "Baldo degli Ubaldi" underground station on the Rome metro. The large dimensions (21.5 m

span and 16 m. in height), the type of ground (Pliocene clays under the water table) and the severe constraints on admissible surface settlements in an urban context required the design of an innovative construction method.
The tunnel for the station was being constructed in four main stages (Figure 3):
1a. two side tunnels, 5 m. wide and 9 m. high, to house the future side walls of the station tunnel were excavated from two access shafts, after first reinforcing the ground ahead of the face (the core-face) with fibre glass structural elements and lining the cavity with fibre reinforced shotcrete and steel ribs equipped with invert struts;
1b. casting of the side walls with reinforced concrete;
2. excavation of the crown of the station tunnel (21.5 m span, 8.5 m. high with a cross section of 125 sq. m.), after first reinforcing the core-face with fibre glass structural elements and protecting it with a strong shell created using the mechanical

__3.__ CONSTRUCTION STAGES FOR THE "BALDO DEGLI UBALDI" STATION TUNNEL
ROME METRO - LINE "A"

3

STATION TUNNEL CONSTRUCTION SEQUENCE



"BALDO DEGLI UBALDI" STATION TUNNEL

ROME METRO - LINE "A"

∅ = 21.50 M.
GROUND: CLAY
OVERBURDEN: ~ 18 M.

precutting method and then immediately placing the lining in the crown with an "active arch" of pre-fabricated concrete segments;

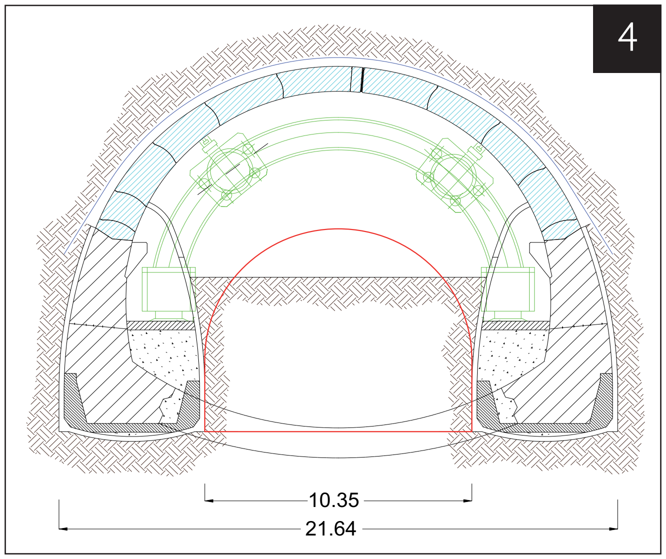
3. excavation downwards of the station tunnel (90 sq. m. cross section) and immediate casting of the tunnel inverts in steps after the construction of the crown;

4. completion of the station infrastructures.

The new construction system contained two important new features:

- one was advance reinforcement of the core-face with fibre glass structural elements and the use of mechanical precutting technology (employed for the first time in the world on a span of 21.5 m.) combined with an "active arch" lining;
- the other was the extremely high level of industrialisation of the works achieved with the intense use of machinery.

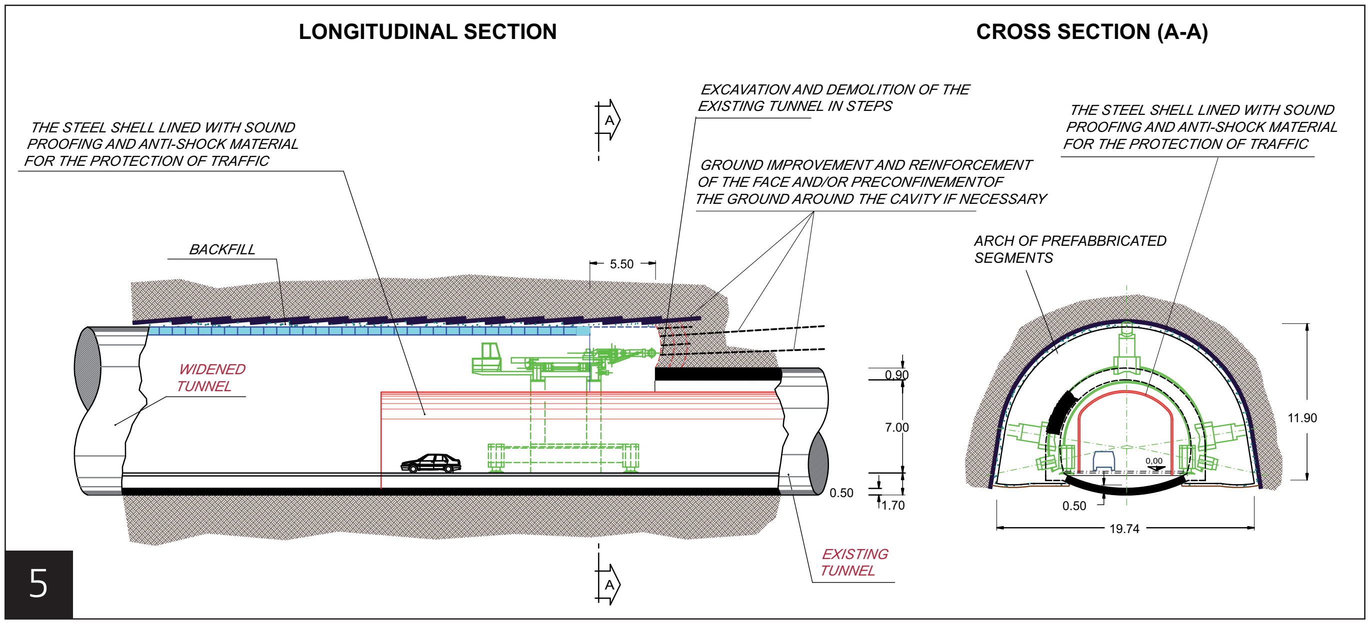
In fact a special machine was designed, developed and constructed jointly with STAC S.p.A. of Mozzate (Como) to combine the technologies employed, all fairly recent, into a single highly efficient construction system. It consisted (photo left) of a large metal portal, with the same geometry as the profile of the crown of the station tunnel, which rested on the inside of the side wall tunnels by means of stabilisers, positioned on its side members, to enable it to travel backwards and forwards.

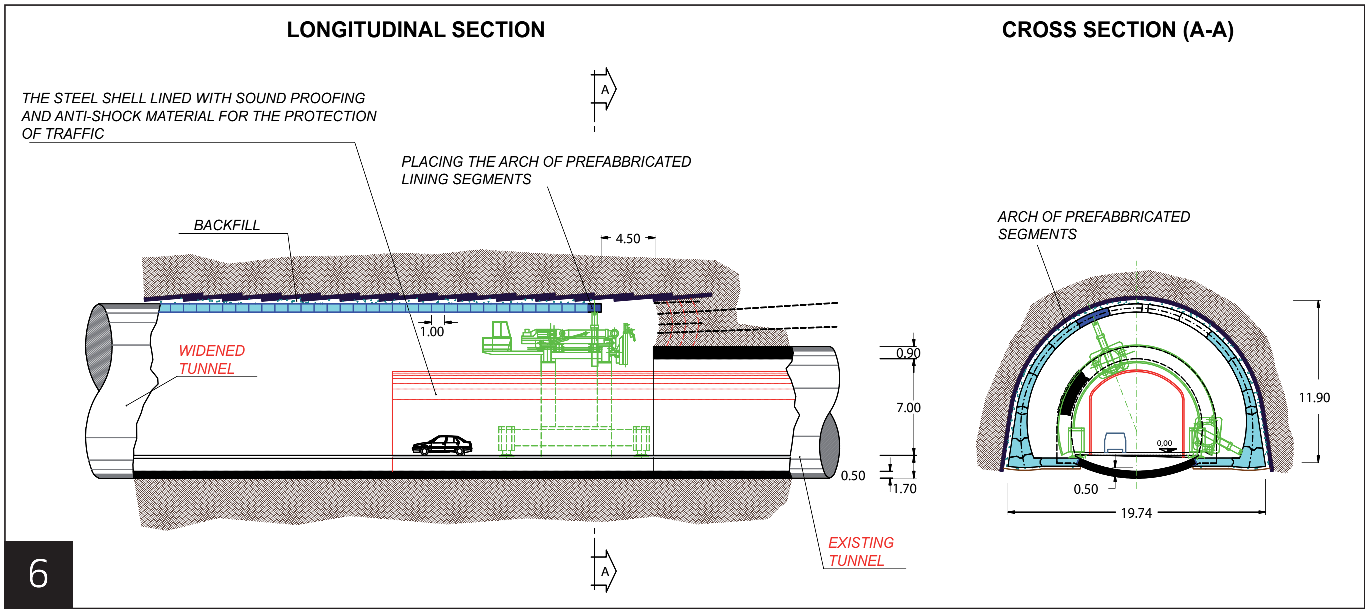


***4.** THE IDEA FOR THE "NAZZANO METHOD" WAS TRIGGERED DURING THE CONSTRUCTION OF "BALDO DEGLI UBALDI" STATION ON THE ROME METRO,*

AFTER NOTICING THAT THE VOLUME OF GROUND BELOW THE SPRINGLINE OF THE TUNNEL, WITH A CROSS SECTION SIMILAR TO THAT OF A NORMAL MOTORWAY OR MAINLINE RAIL TUNNEL WAS NOT AFFECTED TO THE SLIGHTEST EXTENT BY THE CONSTRUCTION OPERATIONS

Not only was the equipment for the mechanical precutting installed on the portal but it also housed that required for handling and erecting the prefabricated concreted segments of the final lining. Once the machine and its accessory equipment were in operation, Prof. Ing. Pietro Lunardi noticed during construction of the crown that the area consisting of the bench of the future station tunnel, with a cross section similar in size to that of a normal motor way or rail tunnel, was not used at all for construction operations (Figure 4). These operations could have been performed in exactly the same way on the extrados of an existing tunnel in order to widen it, without the need to close it to traffic, naturally as long as appropriate safety measures were taken to protect tunnel users. It was, in the final analysis, a question of extending the half cross section system used on the Baldo degli Ubaldi tunnel to the full cross section.





5. *GROUND IMPROVEMENT AND EXCAVATION OF THE GROUND AT THE WIDENING FACE AND DEMOLITION OF THE EXISTING TUNNEL*

6. *PLACING THE ARCH OF PREFABRICATED LINING SEGMENTS, WHICH ALTERNATES WITH EXCAVATION OF THE WIDENING FACE*

That is how the idea of a technique was born, which, by using machinery and equipment based on those used for the Baldo degli Ubaldi station, would be capable of widening an existing tunnel without being obliged to put it out of service.

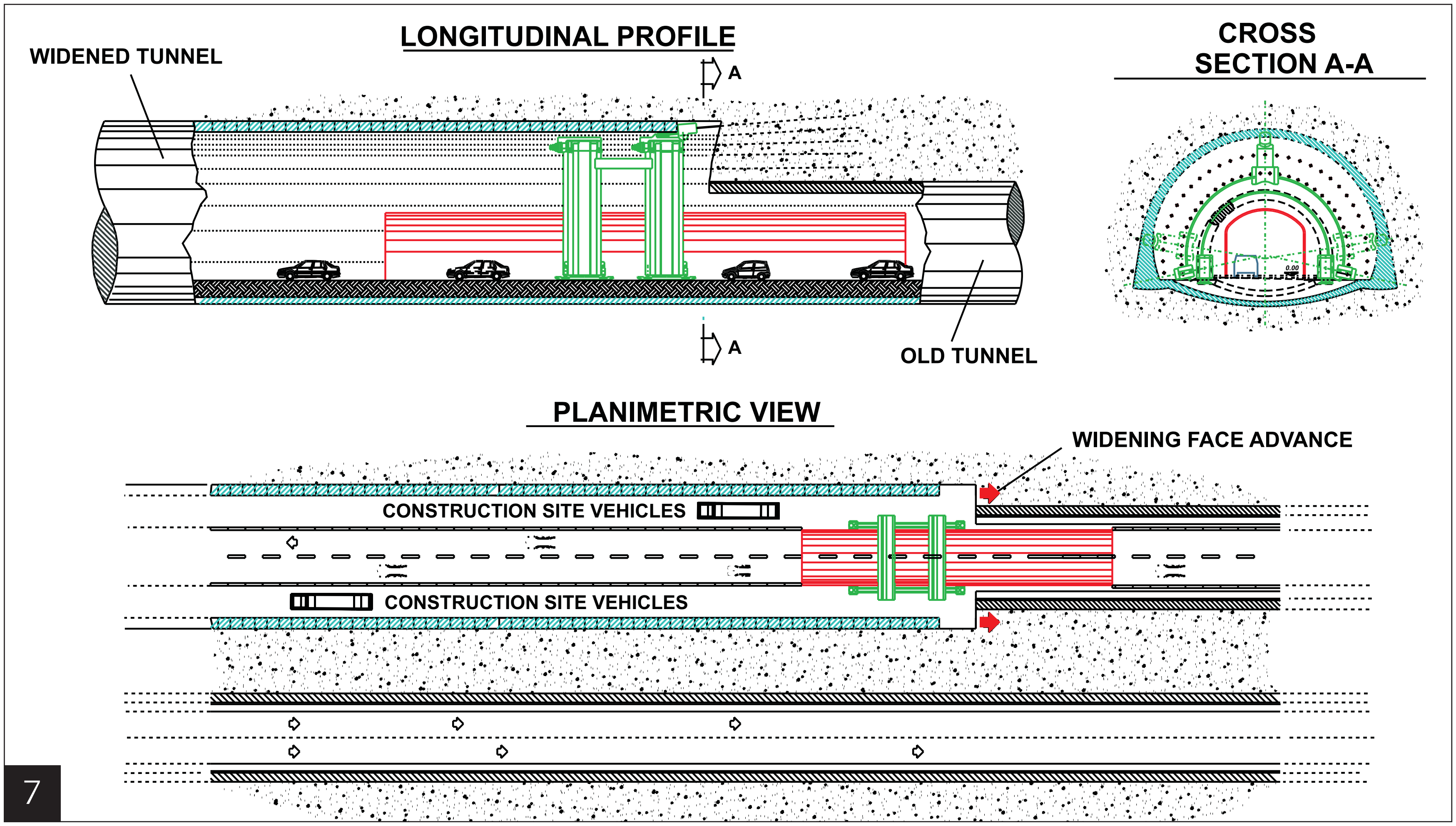
DESCRIPTION OF THE TECHNOLOGY

The "Nazzano Method" involves three basic construction stages (Figures 5 and 6):

- a first stage of "ground improvement and excavation" in steps, during which operations were performed to reinforce the excavation face and/or to preconfine the cavity, as necessary, depending on the geological and geotechnical conditions in question. Then the ground consisting of the "widening" core-face is excavated and the lining of the old tunnel is demolished;
- a second "lining stage", in which the final lining is immediately placed and activated, directly behind the face, by placing one or more arches of prefabricated concrete segments, according to the "active arch" principle;
- a third "foundation stage", during which the foundation of the widened tunnel (tunnel invert) is laid, if necessary. During the first two stages, which should be performed in extremely regular cycles inside the profile of the tunnel to be widened, a steel traffic protection shell is in place and all the machinery used in construction operations moves and works above this (Figure 7). The hollow space between this steel protection and the lining of the existing tunnel is filled with sound proofing and anti-shock material.

The steel shell is at least four times longer than the

7. *DURING THE WIDENING WORK CARRIED OUT INSIDE THE PROFILE OF THE TUNNEL TO BE WIDENED, A "STEEL TRAFFIC PROTECTION SHELL" IS IN PLACE AND ALL THE MACHINERY USED IN CONSTRUCTION OPERATIONS MOVES AND WORKS ABOVE THIS*



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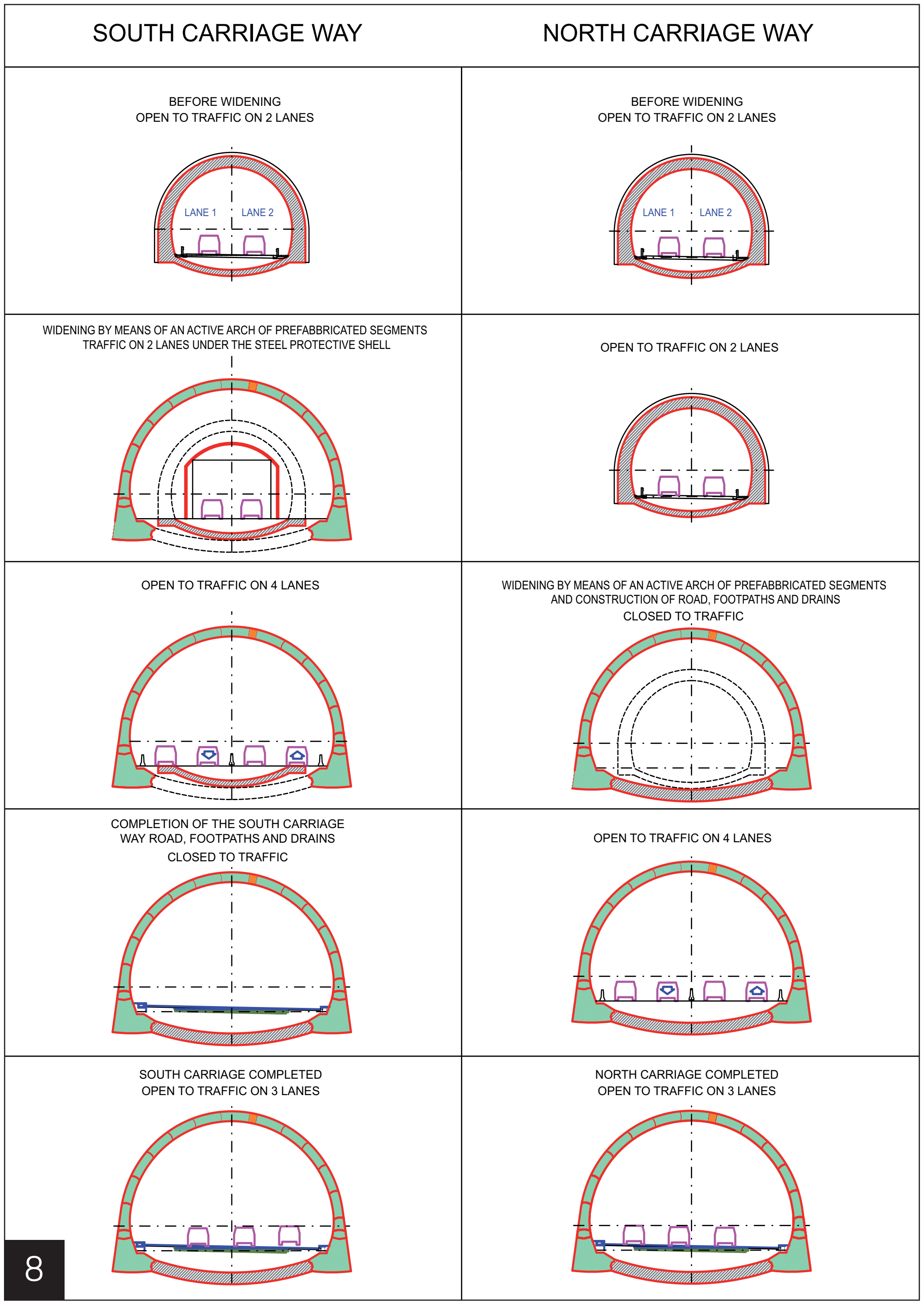


8. THE TRAFFIC REGULATION PLAN DURING WIDENING OF THE CARRIAGE WAY

diameter of the tunnel to be widened and it occupies a relatively small space within it and allows construction work to be performed without interrupting traffic in the existing lanes. When, following face advance, the distance between the face and the front end of the shell is considered to be at the minimum required for safety, the protection shield is moved forwards and the various stages are then repeated in cycles until widening of the entire tunnel is complete.

The "Nazzano Method" is classified, within the framework of the ADECO-RS approach, among conservative techniques to preconfine the cavity by means of protection or reinforcement of the core-face, depending on the ground reinforcement and ground improvement which is chosen on the basis of the geological, geotechnical and stress-strain conditions in question.

DETAILS TO MAINTAIN TRAFFIC FLOWING DURING CONSTRUCTION WORK

While the first two construction stages do not pose any particular problems for the maintenance of traffic flow, since everything takes place above the steel traffic protection shield, with the third stage two distinct cases exist.

Rail tunnels

Once the tunnel has been widened, rail traffic has to be interrupted to change the layout of the tracks in the new situation.

The structure to join the final lining of the widened tunnel with the existing tunnel invert or, alternatively (if the static conditions require it), the casting of a new tunnel invert can be performed in this interval of time.

Road tunnels (single bore) or motorway tunnels (twin bore) with two lanes in each direction

Traffic in single bore road tunnels can always be kept flowing in at least one lane for each direction by appropriate organisation of the works to construct the foundations and widen the road itself.

Similarly, for twin bore motorway tunnels, two lanes in each direction can always be kept open by appropriately switching the works between the two tunnels and deviating traffic flow accordingly onto lanes, according to need, as they become free (as illustrated in the example in Figure 8).

THE APPLICATION OF THE TECHNOLOGY TO WIDEN THE "NAZZANO" TUNNEL

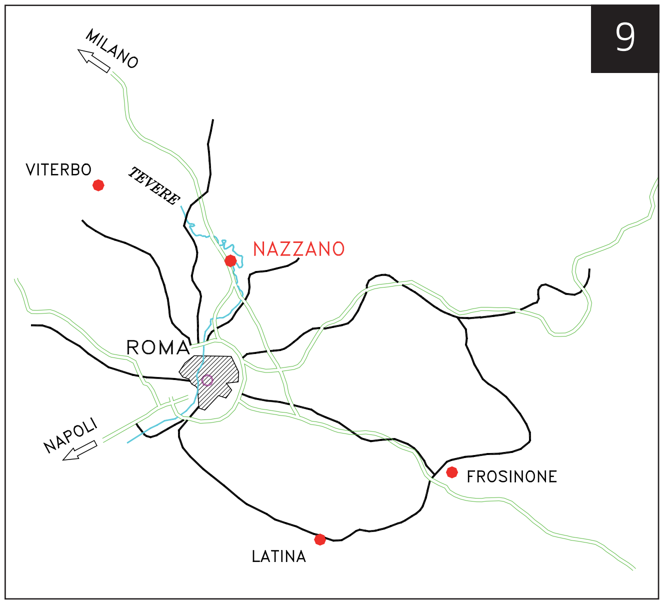
As has already been said, the new technology was applied experimentally for the first time in the world to widen the "Nazzano Tunnel" located on the Milan-Rome motorway between Orte and Fiano Romano, between chainage km. 522+00 and km. 523+200 (Figure 9). The route of this tunnel is completely linear at around 166 m. a.s.l. It is approximately 337 m. long and runs under a maximum overburden of 45 m.

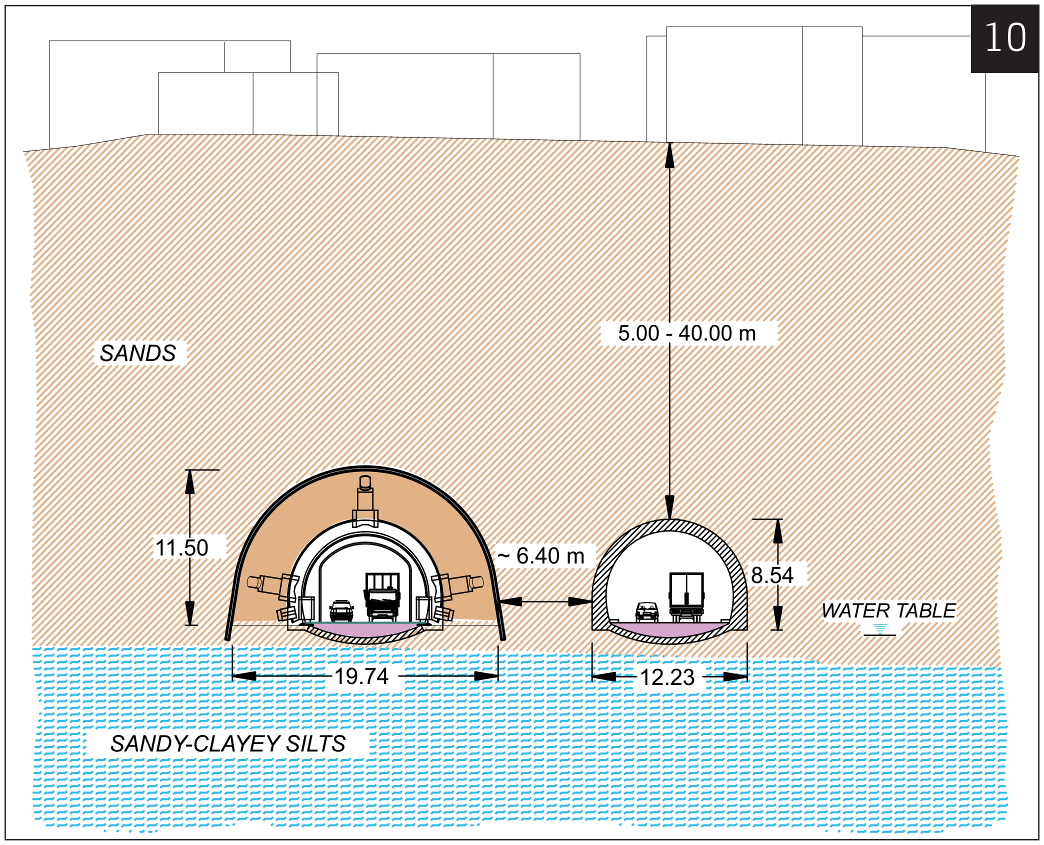
From a geological viewpoint, the alignment runs through sandy and silty-clayey soils of the Plio-Pleistocene series on which the town of Nazzano is located (Figure 10).

In consideration of the type of ground to be tunnelled, the design specified improvement of the ground in advance in the "widening" face, where necessary, by using radial injections of chemical and cement mixtures, performed by working from inside the tunnel during the night.

Subsequently it also specified performing the excavation to widen the tunnel after first creating a shell of fibre reinforced shotcrete around it by means of mechanical precutting technology.

Widening of the tunnel therefore took place according to the following operating cycle (Figure 11):





1. the creation of a mechanically precut shell around the future tunnel (19.74 m. span) 5.5 m. in length and 35 cm. thick and ground reinforcement ahead of the widening face if necessary (Figure 12);

2. demolition of the old lining in steps under the protection of the previously improved ground and excavation of the ground until the design profile of the widened tunnel is reached (Figure 13);

3. immediate erection of the final lining behind the face (4.5 – 6.5 m. max), by placing an arch of prefabricated concrete segments using the "active arch" principle (Figure at pag. 292 and Figure 15).

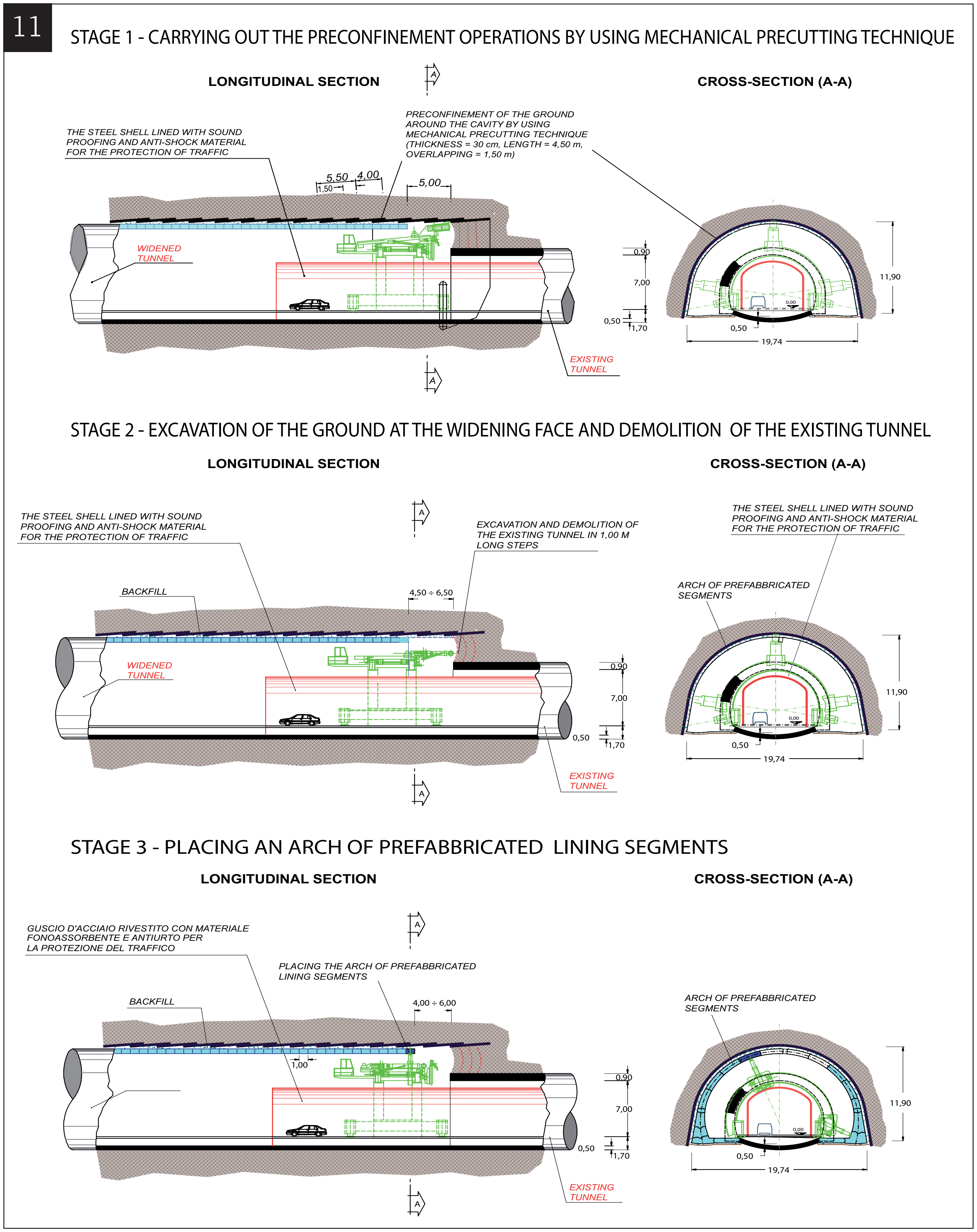
Once the tunnel widening was completed the foundation structure was laid (new tunnel invert). All the work for the first three stages was performed with the roadway protected under the self-propelled steel traffic protection shield under which vehicles continued to flow in safety (Figure 16).

The shield employed measured 60 m. in length and extended for over 40 m. beyond the excavation face.

9. CHOROGRAPHY OF THE NAZZANO ZONE

10. CONSTRUCTION SYSTEM FOR WIDENING THE MOTORWAY TUNNEL IN THE PRESENCE OF TRAFFIC

11. *THE PRINCIPAL CONSTRUCTION STAGES FOR WIDENING THE TUNNEL*



12. *FILLING OF THE PRECUT WITH PUMPED MORTAR*

13. *EXCAVATION AND DEMOLITION OF THE LINING OF THE OLD TUNNEL*

ASSEMBLY OF THE "ACTIVE ARCH" FINAL LINING IN REINFORCED PREFABRICATED CONCRETE SEGMENTS

It consisted of a modular steel structure fitted with runner guides, anchors, motors and sound-proof and anti-shock panels to absorb the impact of falling blocks of material during excavation and demolition of the existing tunnel, including any ground that fell accidentally.
All the machinery for performing the various operations moved and operated above the shield. When, following face advance, the distance between the face and the front end of the shell came close to the minimum required for the safety of motor traffic, the protection shield was moved forwards and the various stages were then repeated in cycles until the whole tunnel had been widened.

THE MACHINE AND ITS EQUIPMENT

The design and development of the machine prototype and its equipment required particular effort because a series of operating functions had to be optimised to work in a very limited space (that between the finished tunnel and the shell): precutting at the face, excavation, placing of the segments, various grouting operations and demolition of the existing tunnel.

The problems were solved by using innovative technologies and the result was a highly computerised, versatile and compact design, capable of performing all the functions required by reducing movements and therefore operating times to a minimum. Basically, it consisted of a robust double arch steel structure (Figure 14) connected at the bottom by telescopic beams, which enabled rapid and precise longitudinal movement both forwards and backwards.

Transverse centring and accurate vertical positioning were performed by means of hydraulic control systems.

A particularly sophisticated carriage is fitted on the arch nearest to the face which holds the cutter for making the precut. The circular movement of the carriage around the arch, obtained by means of gear reduction motors and a rack and pinion mechanism and the single and complex movements of the various parts allow the different operations specified in the design to be performed. A specially positioned dual system is located on the arch to control the tubes used both to fill the precut made by the cutter and the cavity between the concrete segments and the excavation.

The rear arch was designed for placing the concrete segments. A carriage runs on it fitted with an "erector" capable of joining the segments together and setting them in position. The erector is totally electrically and hydraulically powered and is controlled from a mobile switchboard fitted with a display, which provides information on the manoeuvres to be performed and on any errors committed.

Before the key segment is placed and the arch is rendered self-supporting as a consequence, the segments rest on a telescopic framework anchored to the arch itself, which is fitted with sensors that allow the various manoeuvres to be performed in safety.

The structure is fitted with service gangways in different positions to allow personnel to operate with excellent visibility.

The various functions of the equipment are controlled by a PLC (Programmable Logic Controller), which is able to recognise the commands it receives, activate safety locks and send information to the monitors on the various control panels needed for proper and safe control of the equipment.

The table below summaries the main technical data for the machine used.

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14. *THE MACHINE USED TO WIDEN THE "NAZZANO" TUNNEL*

Technical specifications of the machine	
Cutting capacity of the blade	L = 550 cm ; th = 35 cm
Erection of the segments	max load = 7 t at 10,70 m
Rated power	214 KW
Means of power	electricity-hydraulics
Movements	hydraulic controlled by a *Programmable Logic Controller*

15

THE PROGRESS OF THE WORKS AT NAZZANO

Once a series of matters had been resolved attributable exclusively to contractual and financial problems, which delayed the start of widening work several times, work finally commenced on a continuous basis to widen the North carriageway in November 2004 (Figure 17).

After a number of difficulties were resolved related to passing through the low overburdens near the portal, which had been affected to a greater extent by the excavation of the existing tunnel, regular advance rates were finally achieved for tunnel widening in the continuous presence of traffic after a number of fine tuning operations were performed on the system and widening was completed on 17th November 2005. Work to widen the South carriageway started at the end of January 2006 and was completed in March 2007.

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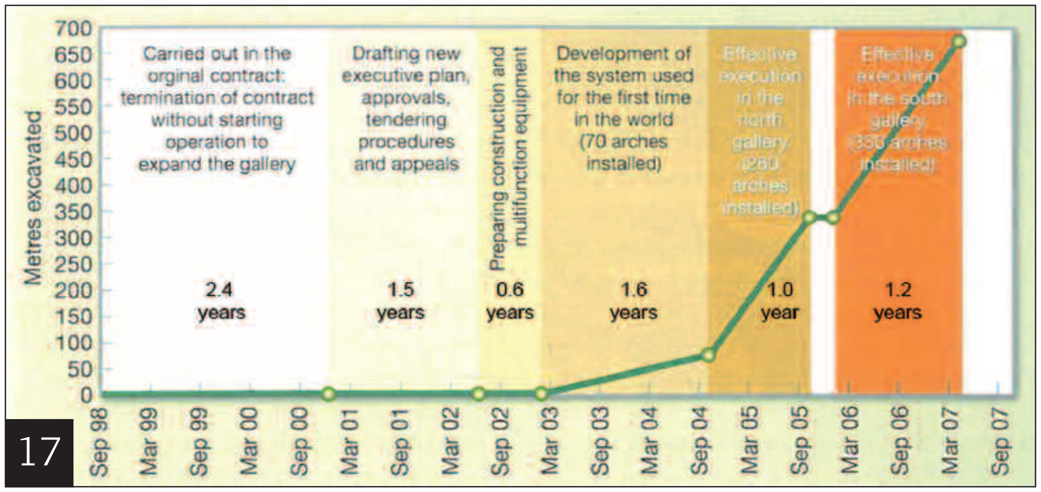
15. *PLACING AN ARCH OF THE PREFABRICATED SEGMENT LINING*

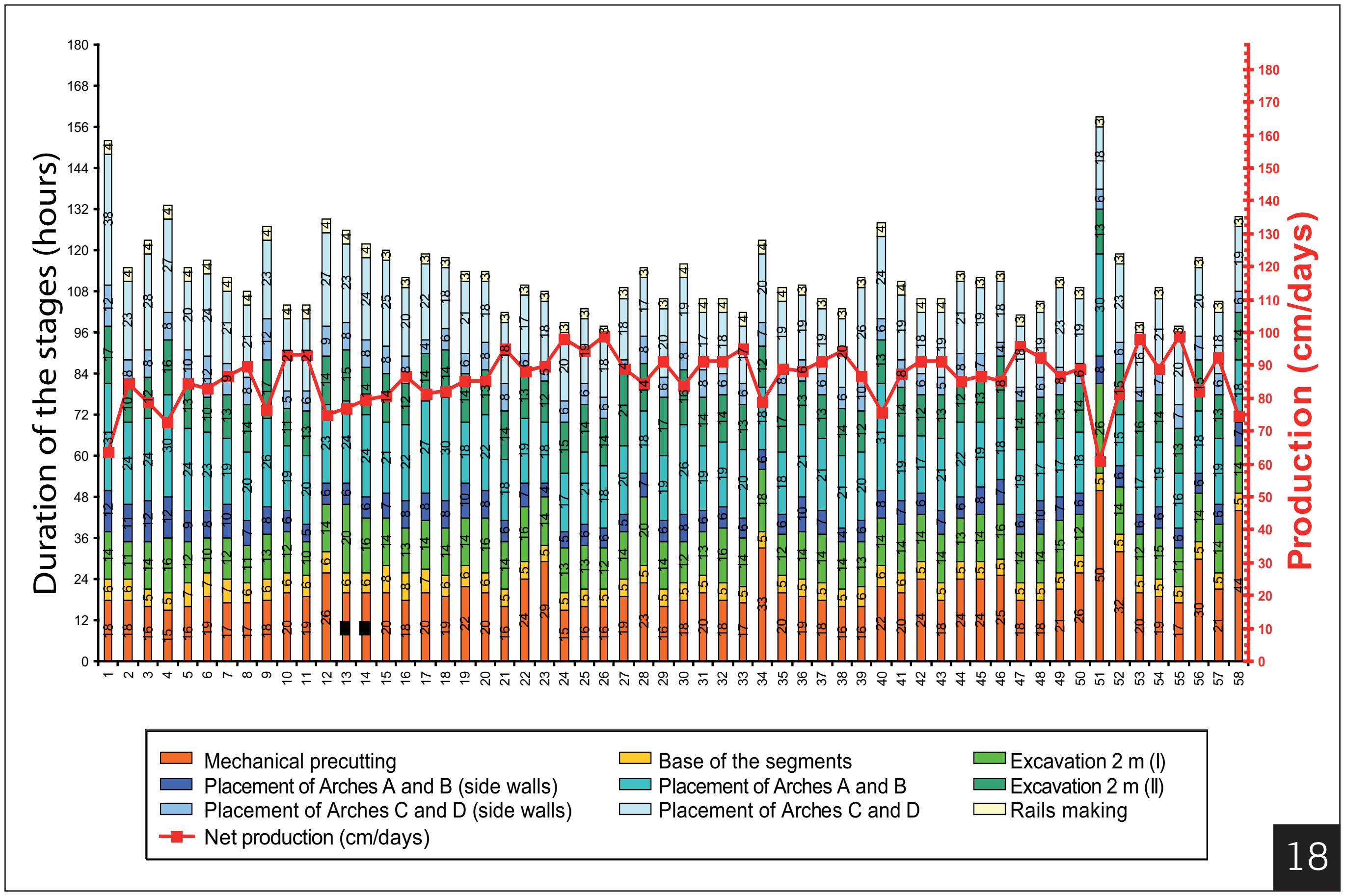
16. *THE TRAFFIC CONTINUES TO FLOW IN SAFETY DURING CONSTRUCTION WORK UNDER THE PROTECTION OF THE STEEL SHELL*

During the continuous operational stage of the works, daily advance rates reached around 0.8 - 0.9 m./day, with peaks of 1 m./day (see Figure 18), which were very satisfactory and sufficient to make the method adopted productively and effectively useable for future tunnel widening in Italy and abroad, especially considering the further technological developments in the field.
Optimisation of the system and the advance cycle carried out while work was in progress mainly concerned the adoption of a longer cutting tool capable of making a "precut" 5.5 m. long and 35 cm. thick (compared to the initial tool of 4.5 m. and 30 cm).
The precut was then followed by two stages of excavation, the demolition of the existing tunnel and the placement of shotcrete at the face, for an advance of 2 m. each, to give a total advance step of 4 m. interposed by the erection of two consecutive "active arches" in the crown with a length of 1 m. each.

CONCLUSIONS

The results of the experiment show that the technology illustrated effectively solves the problems typical of widening a tunnel, while allowing traffic to continue to flow during construction work.





17. *OVERALL DISTRIBUTION OF CONSTRUCTION TIMES FOR THE PROJECT TO WIDEN THE NAZZANO TUNNEL IN THE PRESENCE OF TRAFFIC*

18. *DAILY PRODUCTION RATES DURING THE CONTINUOUS OPERATIONAL STAGES OF THE WORKS*

The main features of the technology are:

1. the adoption of a final lining, consisting of prefabricated concrete segments to stablise the widened tunnel, placed in short steps according to the "active arch" principle, which therefore becomes operational at a very short distance from widening face (4.5 – 6.5 m. max). As a consequence the use of passive stabilisation techniques such as shotcrete and steel ribs is avoided;

2. the ability to make the final lining able to bear loads by using jacks in the key segment in order to recentre the asymmetrical loads should there be bending moments sufficient to make the resisting section of the arch of prefabricated segments act unevenly;

3. the ability to perform ground improvement ahead of the face, if required, to contain or even eliminate deformation of the face and therefore prevent uncontrolled relaxation of the rock mass and thereby ensure the operational safety of excavation work;

4. intense mechanisation of the various construction stages, including the operations for ground improvement ahead of the face, with consequent regular advance rates and shorter construction times, all factors that have advantageous repercussions for construction site costs and the production rates that can be achieved;

5. the extremely linear production rates obtainable (industrialised tunnelling), which it is predicted will be around 0.6-1.2m./day of finished tunnel;

THE DUAL PORTAL OF THE WIDENED "NAZZANO" TUNNEL

A1 MOTORWAY, MILAN-ROME

6. the ability to perform all construction operations while protecting the road with a "steel shell" under which traffic can continue to flow in safety;
7. the extreme versatility of the machine used, which is able to operate under extremely varied ground and stress-strain conditions.

After a significant period spent fine tuning the system, connected with the fact that it was the first time that this technology had ever been used to solve the problem of widening a tunnel with traffic flowing, the experiment demonstrated that the following can be achieved with this technology:

- controlling the effects of the probable presence around the existing cavity of a band of ground that has already been subjected to plasticisation and must not be disturbed any further;
- widening the cross section of the tunnel without causing damaging deformation of the ground and therefore preventing substantial thrusts on the lining of the widened tunnel from developing with differential surface settlement, dangerous for existing structures;
- ensuring that construction occurs on schedule as specified in the design, independently of the type of ground and the stress-strain conditions, with construction times and costs contained and planned in order to reduce traffic deviations and inconvenience to users to a minimum.

As a consequence of the excellent results produced during the experiment, Società Autostrade per l'Italia S.p.A., which together with ANAS operates the road and motorway networks in Italy, is already planning to use the "Nazzano Method" for all future projects to widen tunnels without interrupting traffic, which will be necessary to implement the Italian Government's ambitious programme to modernise and expand the existing road network. The numerous projects planned include preparations currently in progress to widen the two bores of the Montedomini Tunnel approximately 450 m. in length, along the A14 motorway near Ancona.

BIBLIOGRAPHY

LUNARDI P., 2008. Design and construction of tunnels - Analysis of Controlled Deformation in Rock and Soils (ADECO-RS), Ed. SPRINGER, 576 pagine, marzo 2008

LUNARDI P., LUNARDI G., CASSANI G., 2007. Widening the Nazzano motorway tunnel from two to three lanes + an emergency lane without interrupting traffic, Atti del convegno Internazionale su "Tunnels, drivers of change", Madrid, 5-7 novembre 2007

LUNARDI P., 2007. Progetto e costruzione di gallerie - Analisi delle deformazioni controllate nelle rocce e nei suoli (ADECO-RS), Ed. HOEPLI, 575 pagine, agosto 2007

LUNARDI P., 2003. Widening the load at Nazzano, Tunnels & Tunnelling International, Luglio 2003

LUNARDI P., 2003. Un metodo costruttivo innovativo per allargare gallerie stradali, autostradali o ferroviarie senza interrompere il traffico: l'applicazione alla galleria Nazzano sull'autostrada A1 Milano-Napoli, Strade & Autostrade, 2-2003

LUNARDI P., CALCERANO G., 2001. A new construction method for widening highway and railway tunnels, Proceedings of the AITES-ITA 2001 Word Tunnel Congress on "Progress in Tunnelling after 2000", Milano, 10-13 June 2001, vol. 2, pp. 665-671

LUNARDI P., 1999. Une methode de construction innovante pour elargir les tunnels routiers, autoroutiers et ferroviaires sans interrompre la circulation; son application au tunnel de Nazzano sur l'autoroute A1 Milan-Naples, Atti della Conferenza su "Instandsetzung von Tunneln" - Olten, 21 Ottobre 1999

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NEGRI Roberto
NICOLETTI Lorella

PALA Dario
PALOMBARINI Alberto
PANETTIERI Onofrio
PASSERINI Stefano
PELOSO Dario
PENNINO Fiorenza
PINNA Alessandro
PITARO Michele

ROMA *Andrea*
ROMANO *Geremia*
ROSIELLO *Luca*

SALVATORI *Patrizia*
SALVI *Fabrizio*
SAMANNI *Agostino*
SAVARESE *Francesco*
SCARMATO *Alessandro*
SCORDAMAGLIA *Marco*
SICILIA *Rosanna*

SOGNI *Mauro*
SONCINI *Afro*
SOVRAN *Massimo*
SPAGNUOLO *Pina*
STEFANELLI *Ubaldo*

TARCA *Eugenio*
TINELLI *Nadia*
TONINATO *Francesco*
TORRICELLI *Valerio*
TRIBBIA *Nicoletta*

VAILATI *Cesare*
VAIR *Daniele*
VERGAGNI *Pietro*
VILARDI *Alessio*
VISONE *Ciro*
VITIELLO *Antonio*
VOZA *Antonio*

ZACCARIA *Luca*
ZEDDA *Osvaldo*
ZEULI *Vincenzo*