Muir Wood Lecture 2023

The Tunnel Behavior at The Face of Excavation
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I am very honored to deliver the 2023 Muir Wood Lecture. I thank the International Tunneling Association for this invitation, and I am grateful to the French Tunneling Association for its support.

I remember Sir Alan Muir Wood I met in the sixties in Paris and London for preliminary studies of the Channel Tunnel. I was a young engineer very impressed by the eminent engineer, partner of Sir Halcrow and Partners. I remember a perfect gentleman with a British sense of humor.

Tunnels are obviously among the most difficult civil works to design because of the many uncertainties to deal with and their three-dimensional character. A comprehensive evaluation of the ground behavior at the excavation face of is highly important for the design and for the selection of the excavation method. It is the subject of this lecture, based on the experiences from various works in France and abroad.

I shall mainly consider deep tunnels driven in difficult ground conditions, I mean tunnels with an overburden such that the initial stresses may bring about failures or large displacements at the excavation wall.

As a guideline I shall refer to the Convergence-Confining Method (CV-CF Method) which has become very popular at least at a preliminary stage of design analysis. The CV-CF Method stems from former studies of the interaction between the ground and the support conducted by Fenner and Pacher, but it became operational in considering the displacement which occurs before the installation of the support at some distance from the face (M. Panet, P. Guellec, 1974). The CV-CF Method was first codified by the French Tunneling Association (AFTES) in 1979.

Three curves are to be considered:
- The Ground Reaction Curve (GRC) which is the convergence curve of the unsupported tunnel,
- The Support Confining Curve (SCC) giving the support pressure in function of the radial displacement,
- The Longitudinal Displacement Profile (LDP) which represents the radial displacement at the tunnel wall according to the distance to the face.

For the three states:

$$\lambda = 1 \qquad u_R = \frac{\sigma_0 R}{2G} \qquad \sigma_R = 0$$

$$0 < \lambda < 1 \qquad u_R = \lambda \frac{\sigma_0 R}{2G} \qquad \sigma_R = (1-\lambda)\sigma_0$$

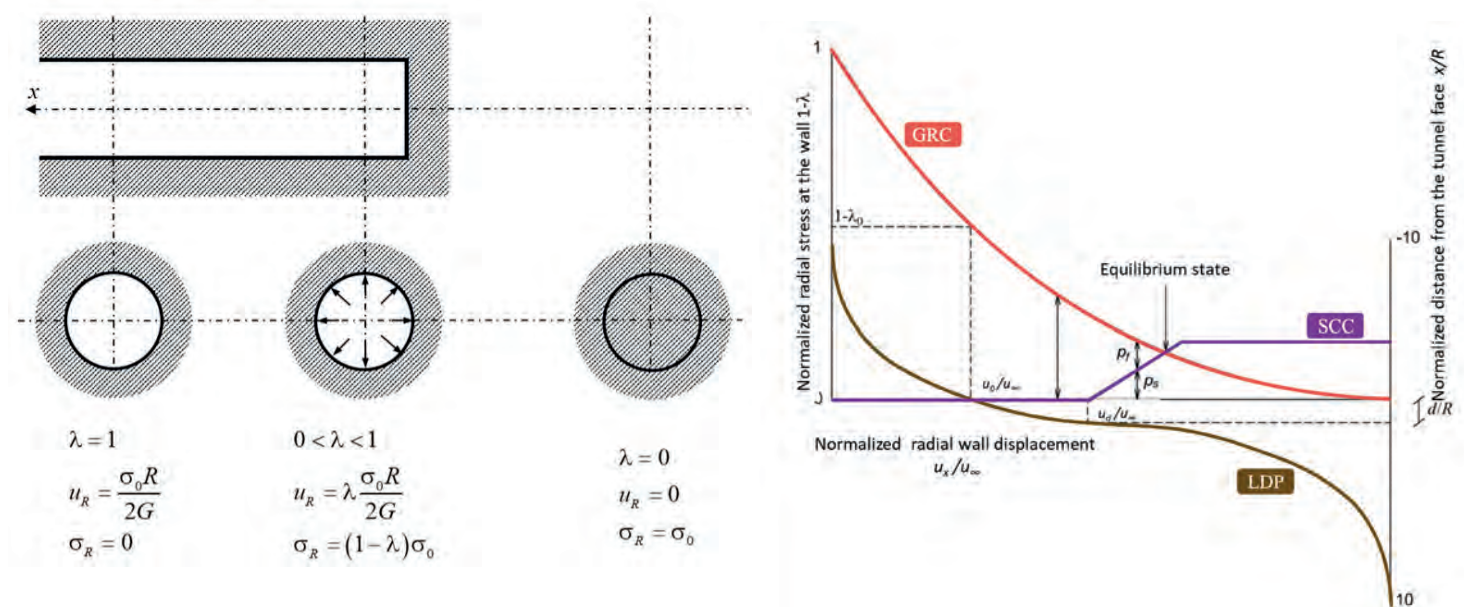$$\lambda = 0 \qquad u_R = 0 \qquad \sigma_R = \sigma_0$$

Fig.1. Convergence-Confinement Method.

This last one depends on the ground behavior at the excavation face, on the unsupported distance behind the face and on the rigidity of the support.

I shall first analyze the ground displacements in the vicinity of the face then the instabilities which may occur at the face.

The monitoring of ground displacements during the excavation has become very common in modern tunnelling.

Near the face, three displacements are now considered:
• The radial displacement behind the face, the convergence
• The radial displacement ahead of the face, the preconvergence
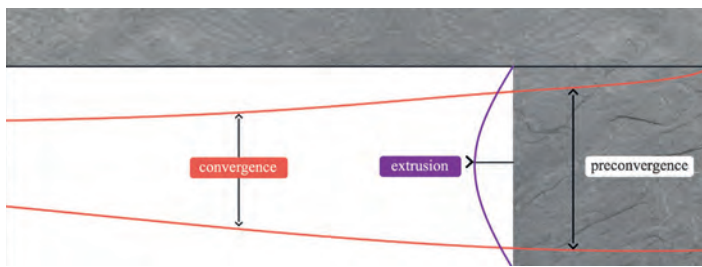• The axial displacement ahead of the face, the extrusion.

Figure 2: Convergence. Preconvergence. Extrusion.

The measurement of convergence was introduced a long time ago in mining to control the closure of mined strata. In tunneling convergence is the relative displacement of two points of the wall in an orthogonal section of the tunnel. The zero measure is to be made as close as possible to the face.

For example, the convergences were extensively used in the French works of the Frejus Tunnel to control the excavation.

The Frejus Motorway Tunnel is a 12.9 km long and 11.6 m wide Alpine tunnel between France and Italy built in the seventies. The overburden along most of the layout is over 1000 m with a maximum of 1800 m. The major stretch was driven in an anisotropic rock mass of calcschists the strike of the schistosity plane being parallel to the tunnel axis. The tunnel was excavated full face by a drill and blast method. Large convergences occurred during excavation. The largest convergences were measured in the direction perpendicular to the schistosity due to buckling failures. The convergences were stabilized by end-anchored steel bolts.
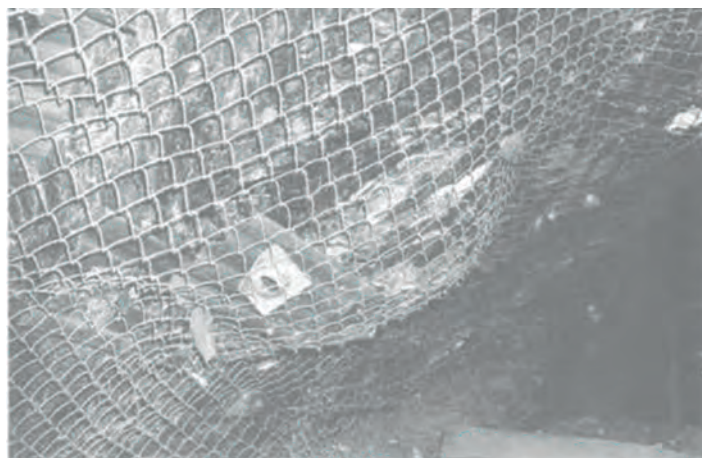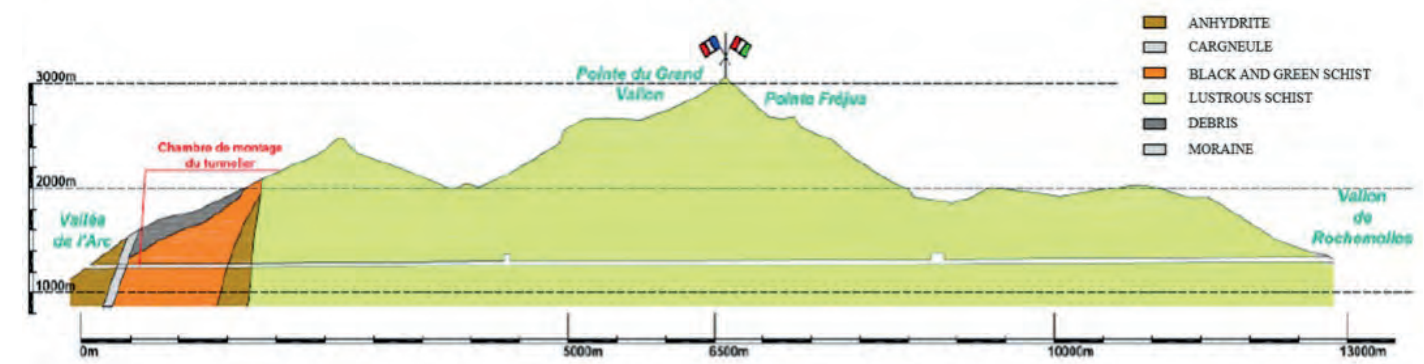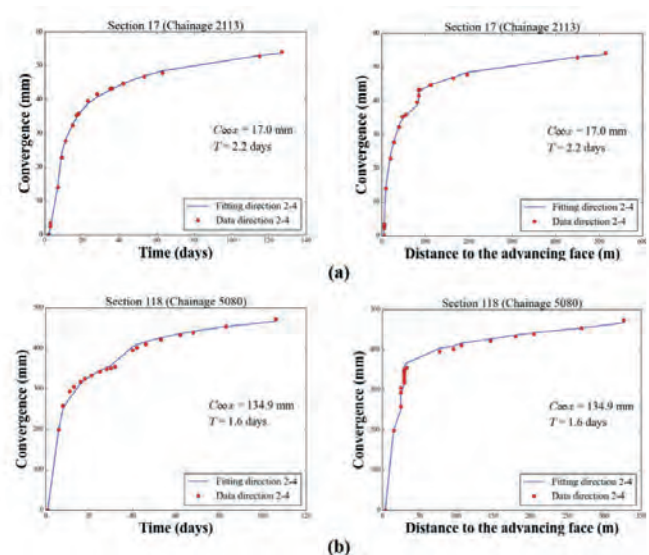

Figure 3: The Frejus Motorway Tunnel.

Jean Sulem made an accurate analysis of the large set of measured data and proposed a semi-empirical law of the variations of the convergence versus the distance of the face x and the time t:

$$C(x,t) = Af(x)\left[(1 + mg(t))\right]$$

$$f(x) = 1 - \left(\frac{X}{X+x}\right)^2$$

$$g(t) = 1 - \left(\frac{T}{T+t}\right)^n$$

X characterizes the distance of influence of the face and T the time-dependent convergence.

For the Frejus Tunnel, X=1.3 L , T=3.75days, m= 4, n=0,3

This law was further applied with a good agreement on several tunnels exhibiting moderate or large convergences.

The notions of preconvergence and extrusion were introduced by P. Lunardi to analyze the zone disturbed by the excavation ahead of the face. The notion of preconvergence is more theoretical than practical because the difficulty of its measure. The extrusion may be measured with a multi-point extensometer set in horizontal boreholes. The extrusion measurements proved to be very useful in difficult conditions (P. Lunardi).

1.1 THE ELASTIC MODEL

An elastic model of ground behavior allows these notions to be clarified. An elastic model is always a crude approximation of the actual ground behavior. Nevertheless, it remains a useful tool for the sake of simplicity and understanding.

Let us consider a circular tunnel (radius R) excavated without support in a homogeneous elastic ground (shear modulus G) with isotropic initial stresses (σ_0).

At a distance x from the face, the radial displacement may be written:

$$u_r(x) = a(x)\frac{\sigma_0}{2G}R$$

$$a(x) = (1 - a_0)\left[1 - \left(\frac{mR}{mR+x}\right)^2\right]$$

$$a_0 = 0.25$$

$$m = 0.75$$

For x>4R, the decrease of $u_r(x)$ is negligible; 4R may be considered in the elastic case as the distance of influence of the face. The notion of the distance of influence of the face is important for the choice of the lining. It may be determined from an accurate analysis of the convergence data.

At a distance x ahead of the face, the extrusion v(x) may be written:

$$v(x) = \beta(x)v_0$$

$$\beta(x) = e^{\frac{x}{R}}$$

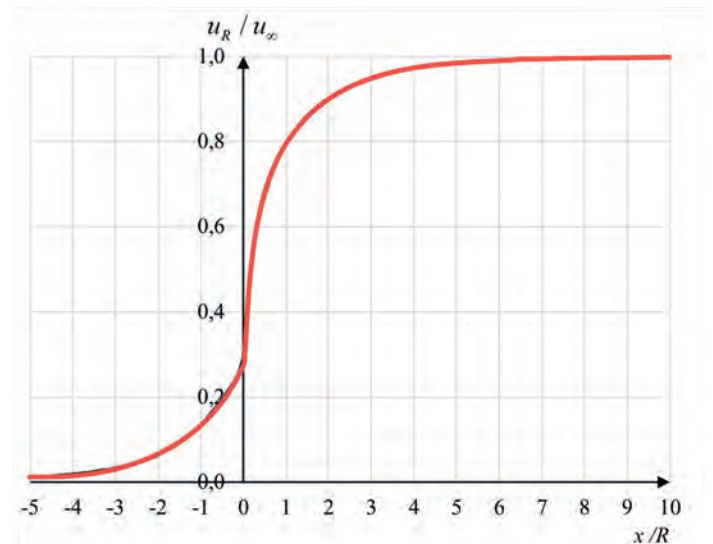For x>2R, the extrusion is negligible.

Figure 4: Longitudinal displacement profile of the unsupported tunnel.

1.2. THE ELASTOPLASTIC MODEL.

When the initial state of stress is large enough to induce extension or shear failures around the excavation, a damage zone or yielded zone develops around the excavation. Various elastoplastic models are commonly used for analysis. The most common are the Tresca, Mohr-Coulomb and Hoek and Brown models.

For a brief assessment of the extension of the damage zone, the stability number N may be considered. N was first introduced by Broms and Bennermark (1967) to evaluate the short-term stability of the face of a tunnel excavated at a depth H in a clay soil of undrained cohesion C_u.

$$N = \frac{\gamma H}{c_u}$$

It has been generalized:

$$N = \frac{3\sigma_0^1 - \sigma_0^3}{\sigma_c}$$

σ_0^1, σ_0^3, principal stresses in the plane orthogonal to the axis of the tunnel of the initial state of stress, σ_c, the uniaxial compressive strength of the ground.

It should be noted that the uniaxial compressive strength to consider is the rock mass uniaxial compressive strength and not the value R_c obtained in laboratory tests on samples. According to E. Hoek and T. Brown, the relationship between σ_c and R_c is:

$$\sigma_c = \sqrt{s}R_c$$

s, being a rock mass parameter determined by the GSI value. Most of time, the tensor of the initial state of stress being unknown, we use:

$$N = \frac{2\gamma H}{\sigma_c}$$

H, the depth of the tunnel axis.
As a first approximation, the extension of the yielded zone is:

• N<1, no yielded zone,

• 1<N<2, very limited yielded zone at some distance of the face,

• 2<N<5, yielded zone developed behind the face and partially at the face,

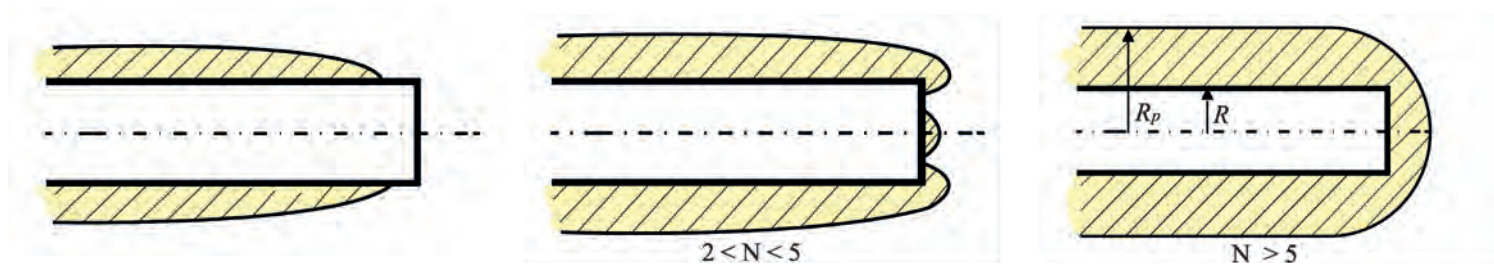• N>5, large, yielded zone developed ahead from the face.

Figure 5: The development of the yielded zone versus the stability number N.

Hoek and Brown gave a graph of the radial displacement of the wall of a great number of tunnels versus σ_c / σ_0, i.e., 1/2N.
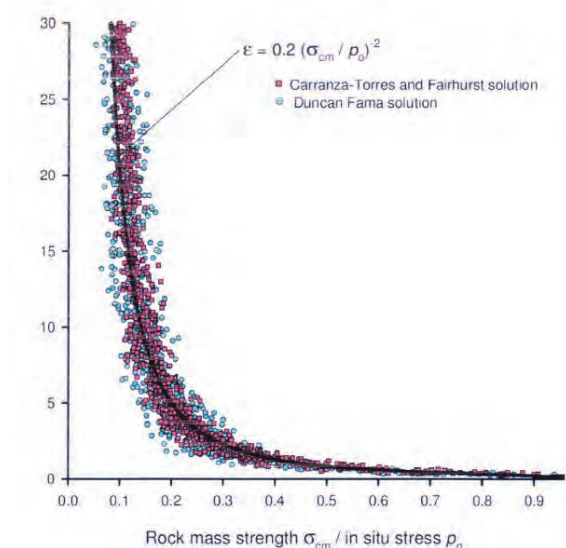

Figure 6: The radial displacements versus the stability number N.

2 >> INSTABILITIES AT THE EXCAVATION FACE.

Various instabilities may occur at the excavation face:
• structural instabilities
• running of cohesionless soil
• debris and water inflows.
• stress induced failures.

2.1. STRUCTURAL INSTABILITIES

Gravity driven falls of sliding wedges or blocks defined by intersecting structural discontinuities are common hazards at the face of tunnels excavated in rock masses. In conventional excavation, the face stability is ensured by rock bolting. Such failures may block the advancement of tunnel boring machine.
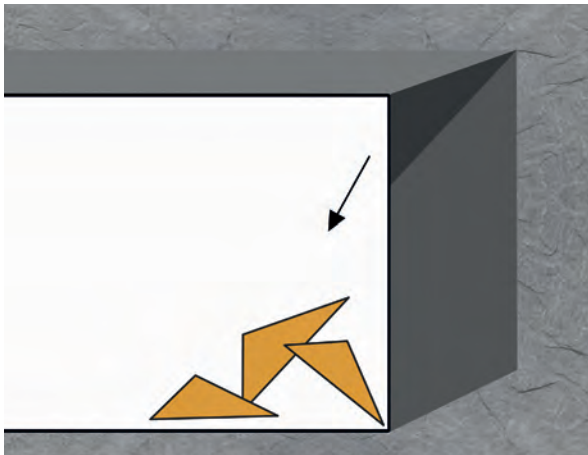

Figure 7: Block instability at the face of a tunnel.

2.2. THE RUNNING OF COHESIONLESS SOIL.

Cohesionless soil or highly broken rock are classified as running ground; the face is unstable.
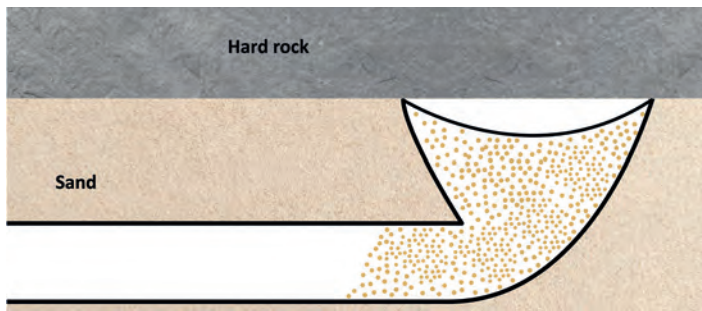

Figure 8: Instability of running ground at the face of a tunnel.

Grouting or ground freezing are necessary in conventional excavation mode.

2.3. DEBRIS AND WATER INFLOWS.

Debris inflows occur when the face of a tunnel advances close to the boundary between a watertight ground and a zone of loose cohesionless soil or highly fractured or weathered rock under high

ground water pressure. The sudden blow in of the face causes a flow of material which may fill the tunnel over a considerable length.
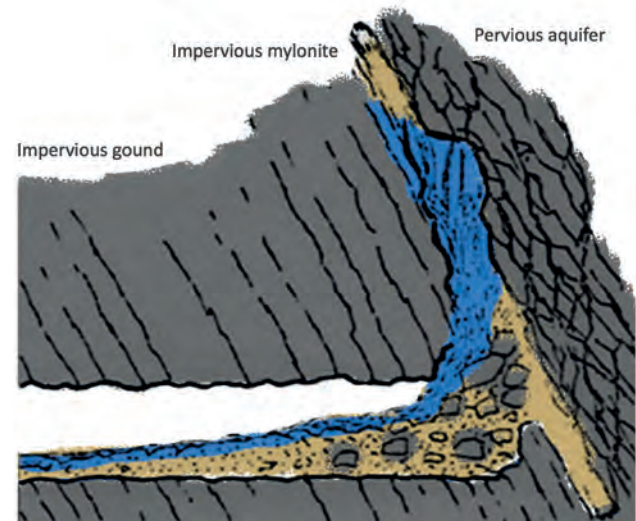

Figure 9: Debris and water inflows at the face of a tunnel.

Throughout the history of tunnelling there have been a great number of such accidents sometimes with dramatic consequences.

One of the most memorable and tragic accident occurred during the construction of the Lötschberg Raiway Tunnel in Switzerland on July 24th, 1908. Under the Gastern Valley at 2675 m from the Kandersteg Portal, a flow of mud and gravels filled in the tunnel over a length of 1880 m. 24 miners could not escape and died. The geologists had strongly assumed the presence of a cover of about 100 m of limestone between the roof of the tunnel and the base of the alluvial deposits.

Figure 10: Debris inflow at the Lötschberg Tunnel

Jean Launay has reported a more recent example of debris and water inflows during the works of the hydro-electric Dul-Hasti gallery in India. The 10 968 m gallery is situated north of the Main Central Thrust in Himalaya and crosses the Kishwar graben; this zone is presently tectonically active; the boundary faults of this graben exhibit a relative displacement of 20 mm/year.

From the Dul portal the 8.30 m diameter gallery was excavated by a Robbins TBM for hard rocks with grippers. The rock was mainly an abrasive quartzite with a uniaxial compressive strength of 251 MPa.
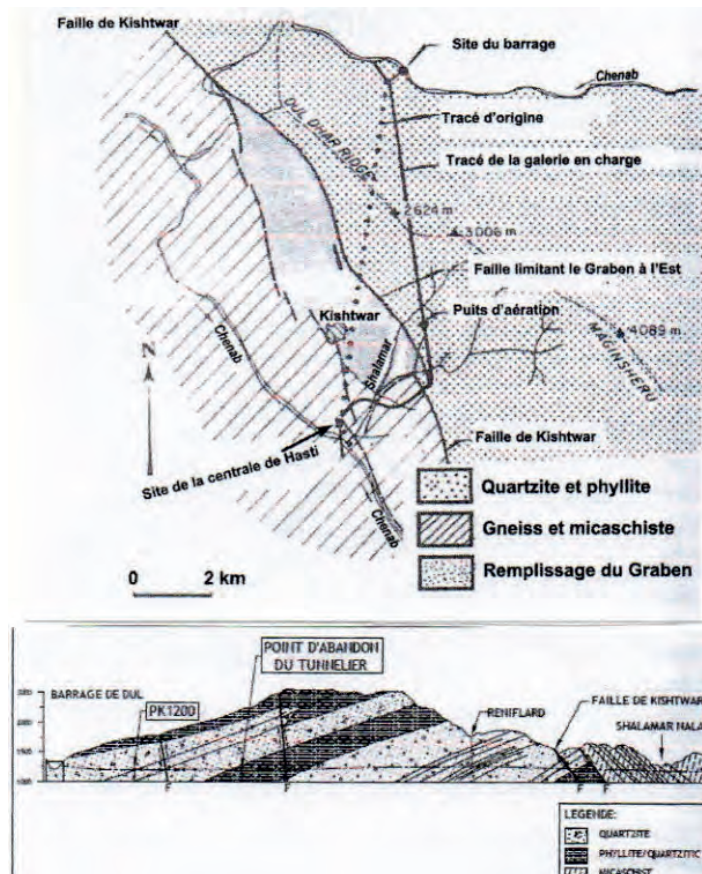

Figure 11: Debris inflow in the Dul Hasti Gallery.

The rock mass was only slightly fractured and only local support with steel arches was provided. In May 1992, the gallery after crossing a watertight layer of phyllite approached a zone of quartzite mylonite, a cohesionless material, under a waterhead of 500 m. The face blew out with a flow which reached 1100 l/s. The volume of material which filled the gallery over a length of about 200 m was estimated to 2500 m³.

Large water inflows do not always imply debris inflows infilling the tunnel behind the face. Such was the case of the construction of the upward Salazie gallery in the Reunion Island which had to face severe water inflows.

The 8.5 km long Upstream Salazie Gallery is part of a water transfer from East to West in the Reunion Island cumulating 30km of galleries.
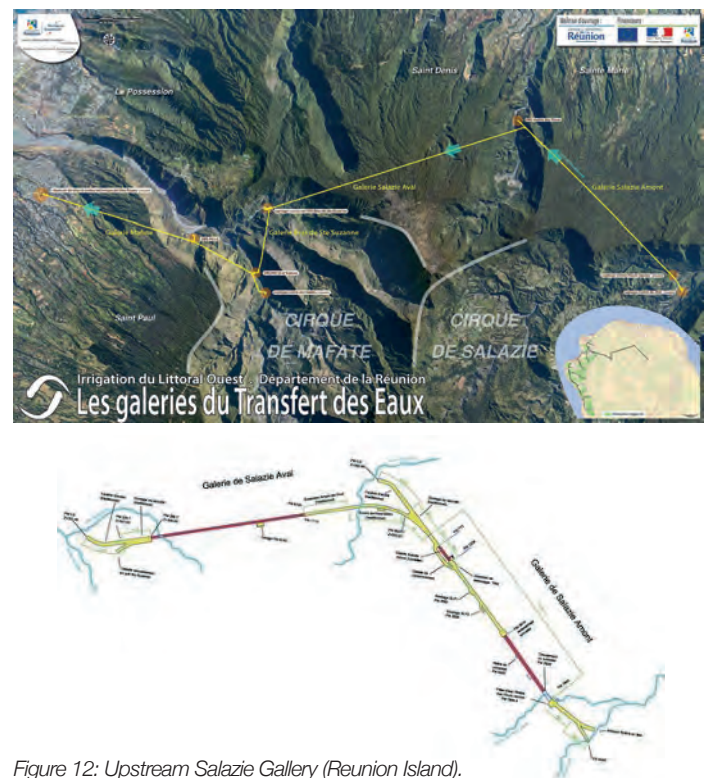

Figure 12: Upstream Salazie Gallery (Reunion Island).

The Upstream Salazie gallery was driven in volcanic formations alternating compact or vacuolar basalts with breccia or scoria cut by dykes and sills. The overburden is about 1000 m. Several large water inflows up to 1200 l/s occurred under a pressure of 33 bars with almost no debris.

The risk of water inflow at the face may be forewarned by boreholes ahead of the face. In the Salazie Gallery, two horizontal boreholes respectively 428 m and 800 m long were drilled.

Drainage and grouting are commonly used to prevent water inflows at the face.

2.4. STRESS-INDUCED FAILURES.

Stress-induced failures are due to the development of shear or extension fractures at the face.

These failures have been accurately studied in the Andra Underground Laboratory in Bure (France).

Andra is the French agency for the radio-active wastes disposal. A site for a deep level storage has been selected in the eastern part of the Bassin Parisien. The host rock is a very homogeneous Callovo-Oxfordian formation of argillite exhibiting no faults or natural fractures with a very low permeability (less than 10-12 m/s). The uniaxial compression strength is about 21 MPa.

For several years, Andra has developed an underground laboratory at a depth of 490 m with a large number of galleries.
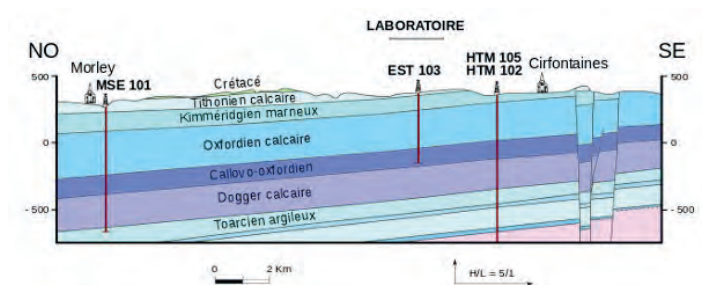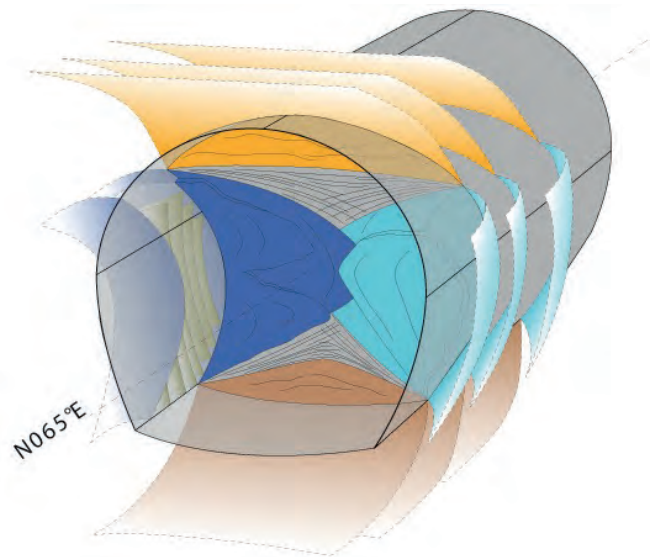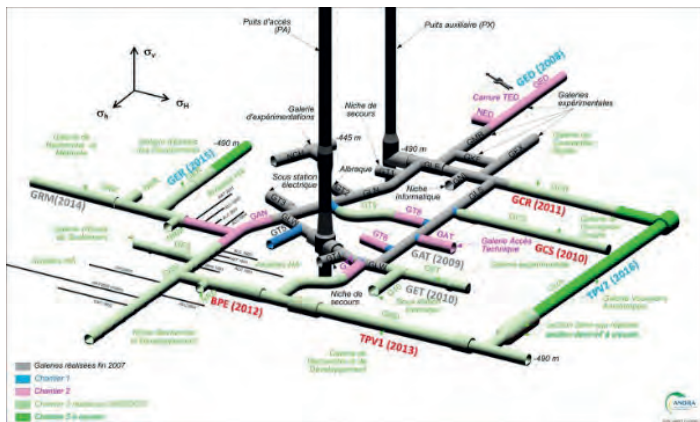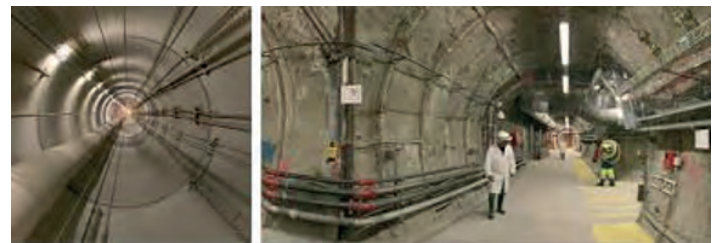

Figure 13: Andra Underground Laboratory for radioactive wastes.

The natural state of stress has been obtained from various tests in boreholes. It is anisotropic with a maximum horizontal stress of 16 MPa striking N 155 °; the minimum horizontal stress is equal to the vertical stress, 12 MPa.

The experimental galleries are orientated orthogonal or parallel to the maximum natural horizontal stress. The shear or extension fractures due to the excavation of the galleries were accurately described in the vicinity of the face. They vary significantly according to the orientation of the galleries.

It may be noted than the stability number N is respectively equal to 2 and 3 according to the orientation of the gallery.

In brittle rocks, the crack initiation stress is the threshold at which they become damaged by the onset of microcracking during compression. When the compressive tangential stress at the excavation wall exceeds the crack initiation stress, extension fractures develop at the walls of the excavation and produce a slabbing.

In brittle granitic rocks the damage initiation stress is more or less lower than half of the uniaxial compressive strength.
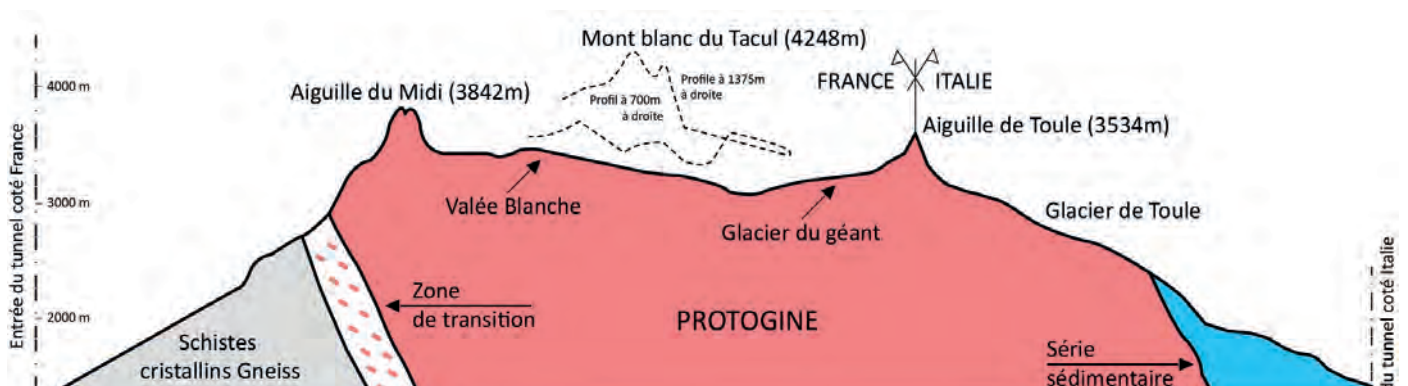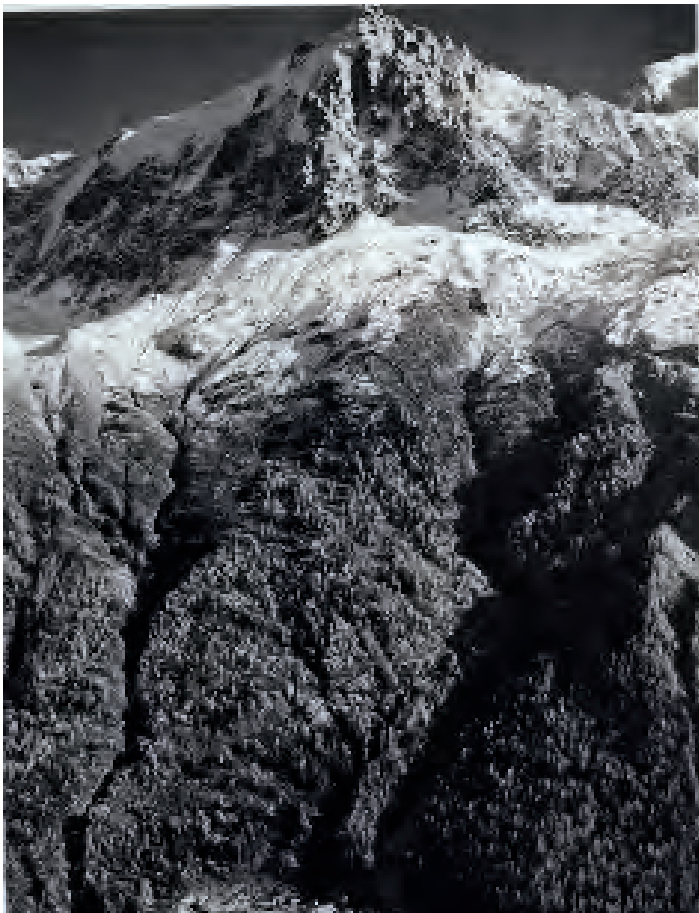
In the case of rockbursting, the slabbing may be uncontrolled and violent. Rockbursting occurred in deep mines in South Africa. But it occurred also during the construction of Alpine tunnels in granitic or gneissic rocks with a large overburden. Though important

research has been carried out in South Africa and Canada about rockbursting, the violent character of the failure has not been well explained.

Rockbursts occurred during the excavation of the Mont Blanc Tunnel. On most of its length the tunnel was driven in a metamorphic rock called protogine with a drill and blast excavation mode. The uniaxial compressive strength of this rock σc is about 120 MPa and the crack initiation stress about 50 MPa. On the longitudinal profile of the tunnel, the maximum overburden is below the Aiguille du Midi (3850 m) but laterally it may be over 4000 m (Mont Blanc du Tacul). Let us consider that the maximum initial stress is about 110 MPa. In situ stress measurements gave a horizontal stress of 50 MPa. Therefore, the stability ratio N is about 2.3.
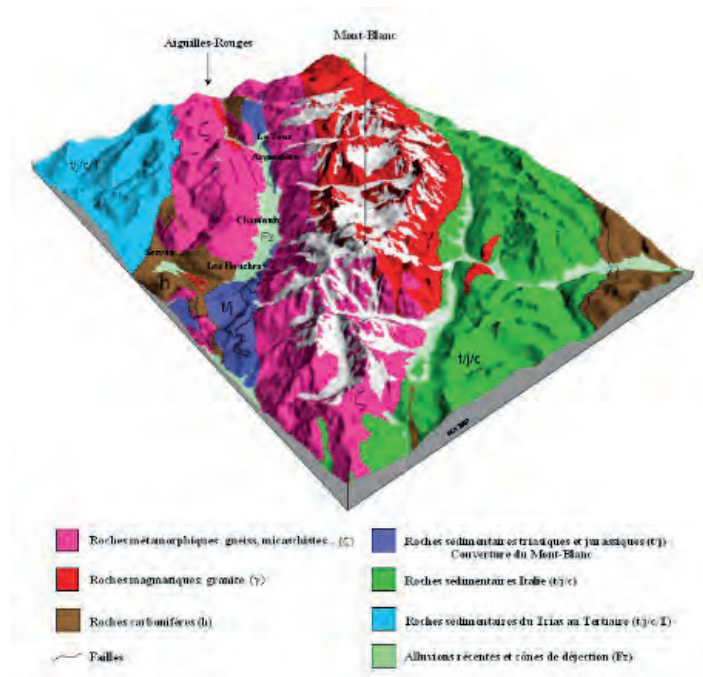
Figure 14: Rockbursts in the Mont Blanc Tunnel.

In the severely impacted sections, rockbursts occurred even at the face. Rockbolting of the face before drilling the blastholes was necessary. Among the 165 000 rockbolts installed to ensure the stability of the walls, 28 000 were installed at the face.

For tunnel driven in schistose rocks with the schistosity plane vertical and perpendicular to the tunnel axis, extension fractures may bring about buckling failures at the face.
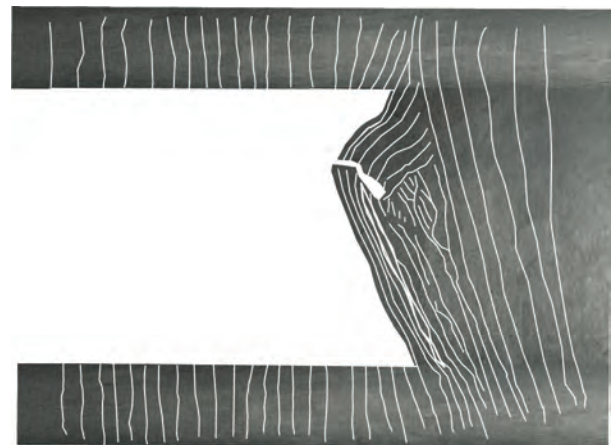

Figure 15: Buckling failures in tunnels in schistose rocks.

For the construction of large tunnels, the engineer must choose between a conventional excavation mode or the use of a Tunnel Boring Machine.

In the past, full-face exaction was used only for small galleries; for large tunnels, a sequential excavation method (in separate sections) was used because of the available equipment and to face the risks of failures on large and high faces. In European countries, different types of division of the cross section were proposed and named according to the country (Belgian, Austrian, Deutsch …).
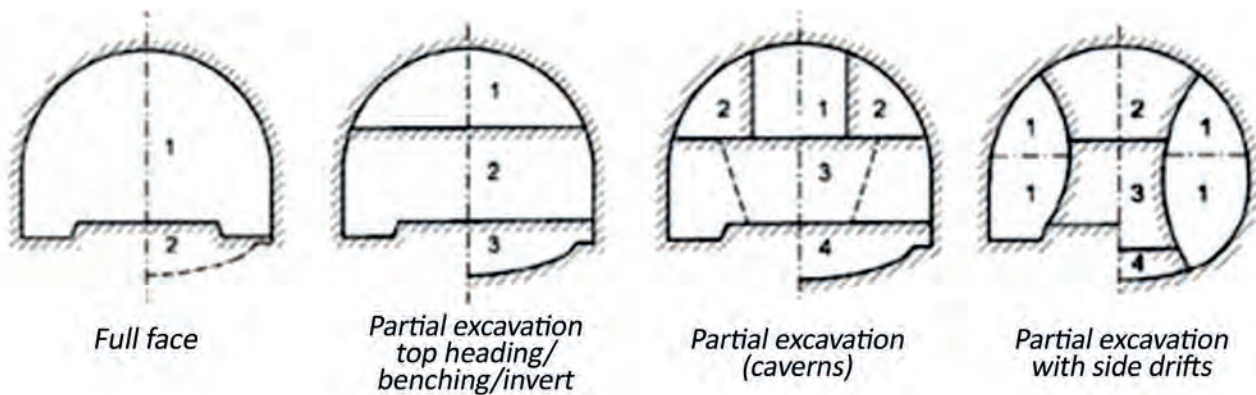

Figure 16: Different types of division of the cross section.

Furthermore, the top heading and bench sequence of excavation became very common. Sequential mode of excavation is also often associated with the New Austrian Tunneling Method (NATM).

Figure 17: Tunnel driven in semi section on the motorway A6 (France).

More recently with the ADECO-RS method Pietro Lunardi developed a full-face tunnelling method with a reinforcement of the face. A stiff support composed of steel-fiber shotcrete and heavy steel sets is installed behind the face and the concrete invert and concrete lining are cast as close as possible to the face ensuring the face stability and limiting the extrusion and the preconvergence. The consequence is a larger ground pressure on the support.
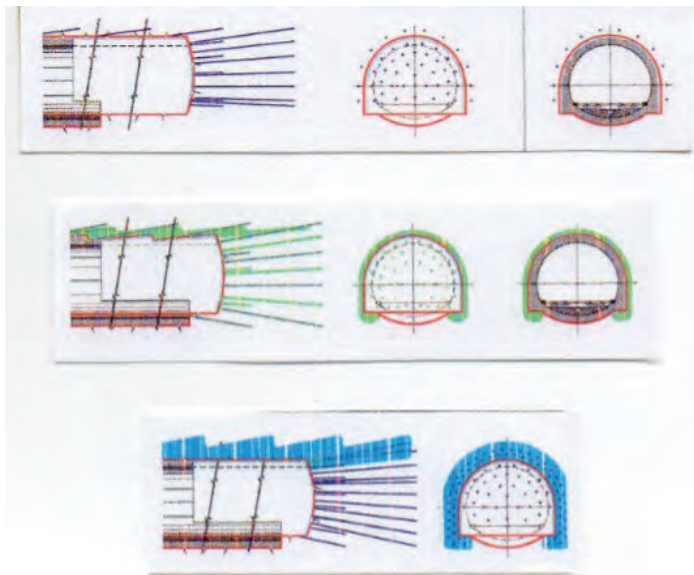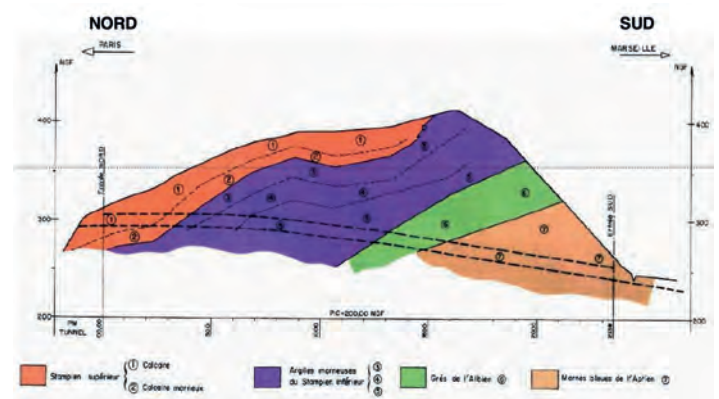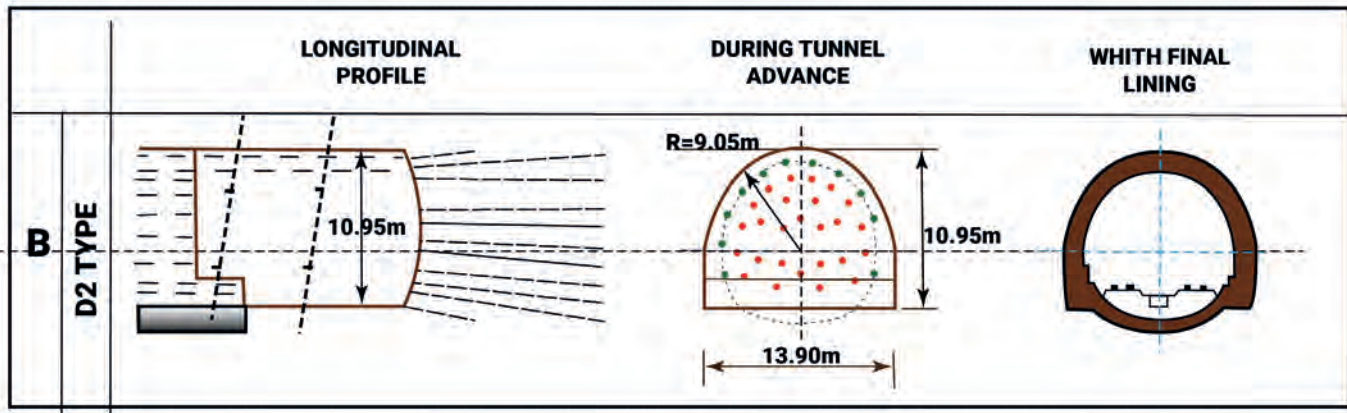
Figure 18-1: ADECO-RS Method.

This method has been applied successfully in various tunnels with cross-sections ranging from 120 m^2 to 220 m^2.

The most common technique of ensuring the face stability, first developed in Italy, is the reinforcement by fiber glass bolts which can be easily broken by mechanical excavation engines.

In more difficult ground conditions complementary techniques maybe used to control convergence and extrusion such as:
• forepoling,
• sub horizontal jet-grouting,
• mechanical precutting.

The ADECO-RS method was applied in France for the Tartaiguille Tunnel.

Figure 18: ADECO-RS method applied in the Tartaiguille Tunnel. TGV Lyon-Marseilles (France).

The Tartaiguille Tunnel is a railway tunnel of the high-speed railway line between Lyon and Marseille (length: 2430 m, cross section 180 m^2, overburden; 140 m). It was driven in a marly rock of the Lower Stampian. The stability number N may be estimated to be about 2.

With the ADECO-RS method a considerable part of the initial ground pressure act on the support and on the lining. With large overburden and difficult ground conditions, this method becomes impractical. Such was the case of the Saint Martin la Porte Adit along the Lyon-Turin Base Tunnel which encountered very severe squeezing conditions.

The Saint Martin la Porte Adit was driven in a highly heterogeneous fractured rock mass. This Carboniferous formation comprises black schists, sandstones, clay-like shales, coal. From back analysis, the σc value is assumed to be about 2.5 MPa. Accordingly, the N value is over 12. The convergences reached 200 cm.
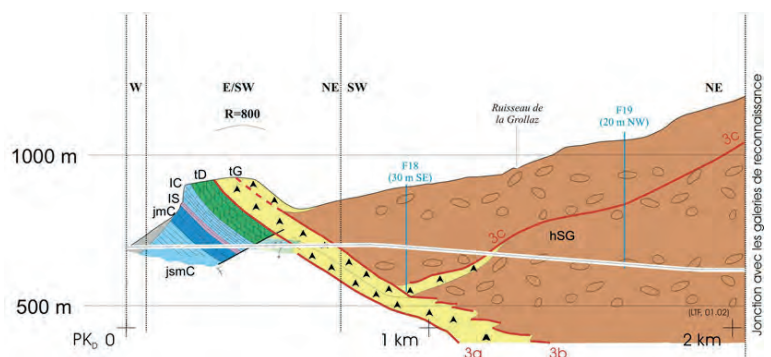

Figure 19: Saint Martin la Porte Adit. Lyon-Torino Base Tunnel.

Figure 19: Saint Martin la Porte Adit. Lyon-Torino Base Tunnel.

To cope with these severe squeezing conditions, a flexible support solution was feasible. A yielded multiphase support was implemented. It consisted of steel ribs with sliding joints (TH type), of dense rockbolting, and of shotcrete layers in conjunction with slots fitted with HiDCon (Highly Deformable Concrete Element) elements. The extrusion of the face was controlled by fiber-glass bolts. The final concrete lining was cast 80 m from the face.

4 >> TUNNEL BORING MACHINES IN DIFFICULT GROUND CONDITIONS

Excavation by tunnel boring machines has become increasingly common for the construction of long tunnels even in difficult ground conditions owing to its high rate of excavation.

Modern TBM may now be designed to deal with a great variety of geological conditions: open TBM, single-shield TBM, double-shield TBM.

The limitations of the use of TBM in deep tunnels are due:
- the difficulties to control the overbreaks before the cutterhead of the machine,
- the limit of the confining pressure to be applied at the face,
- the possible entrapment of the shield in severe squeezing conditions,
- the maximum thrust and torque capacity of the TBM,
- the segment lining capacity to support loads applied by the ground and the thrust of the TBM.

In difficult ground conditions, the problem of stability at the face is the major factor for the choice of the type of TBM. The main difficulties to overcome are running ground, overbreaks which brings about a cavity at the upper part of the face with falling blocks which may block the rotation of the cutter head, squeezing conditions.

When these conditions are to be encountered over a long stretch of the tunnel, the TBM is designed with a pressurization of the face by compressed air, slurry, or earth pressure. The confining pressure may be determined to obtain a stability number lower than 2. The stability of the confining pressure is an important factor for the choice of the type of pressurization. To deal with varying conditions along the tunnel mixed face TBM may function in open and closed mode with different confining and mucking techniques.

The Villejust Tunnel on the Atlantic TGV in France is a double-track 4805 m long tunnel driven in cohesionless sand (Sable de Fontainebleau). To prevent the running of the ground at the face, it was excavated by two slurry tunnel boring machines.
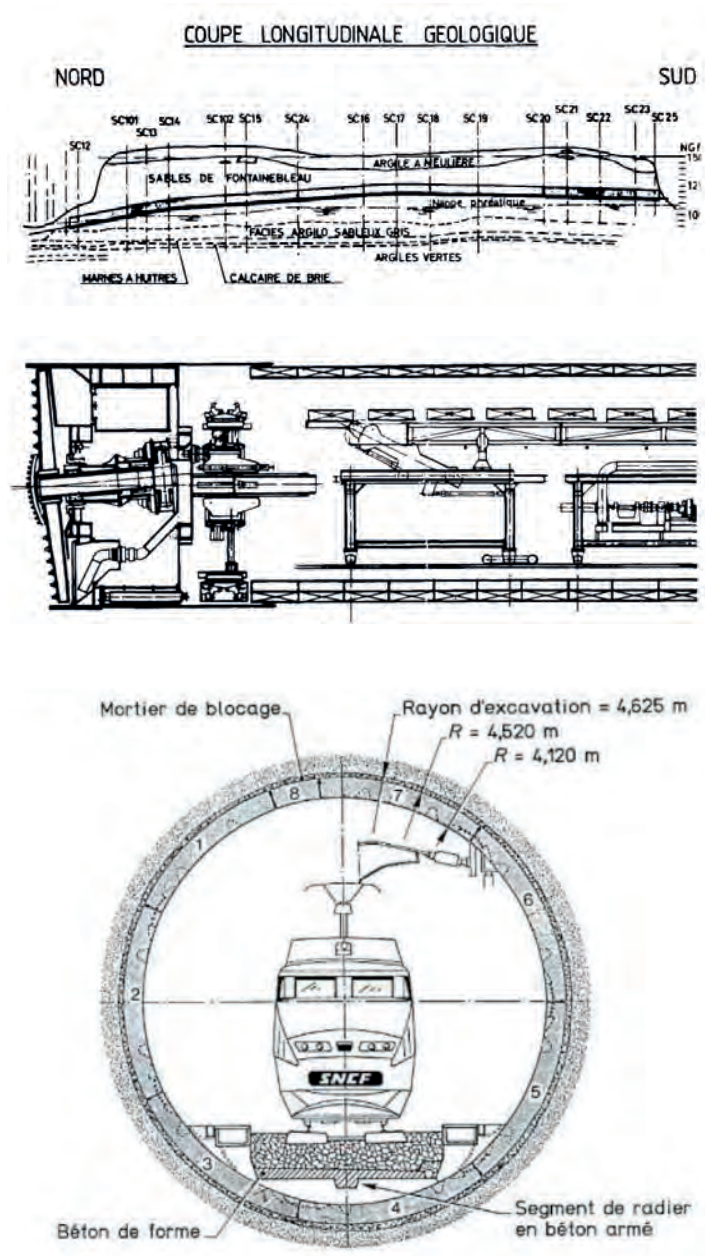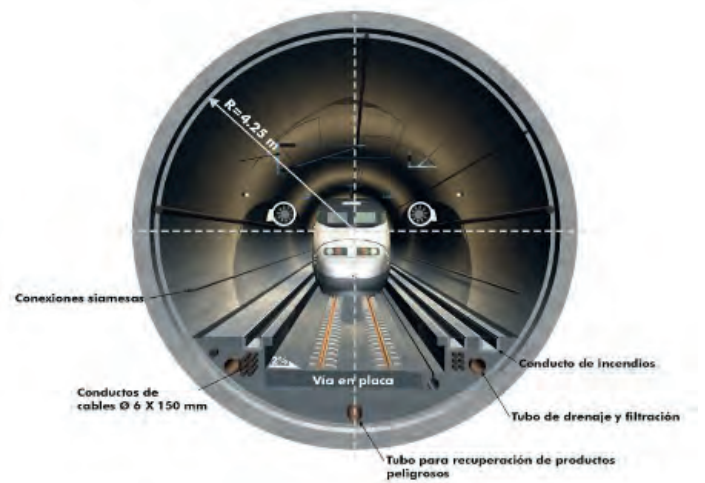

Figure 20: Villejust Tunnel.

However, the major incidents in TBM tunnel excavation occur when local unexpected geological conditions at the face stop the advance. The stoppage of the excavation may last for a long time with large consequences in terms of the project cost and schedule. In some cases, it was been necessary to give up the TBM.
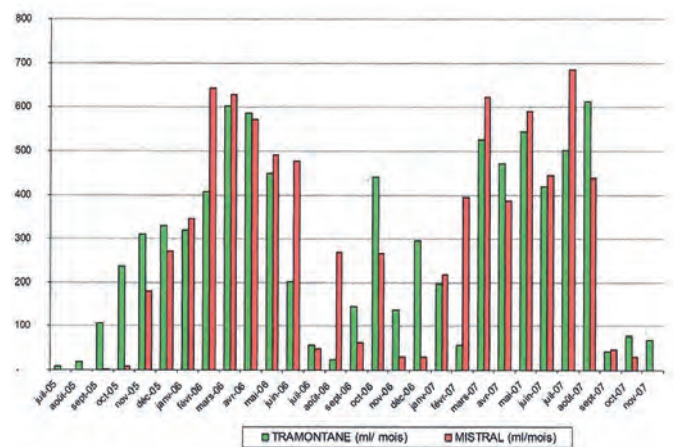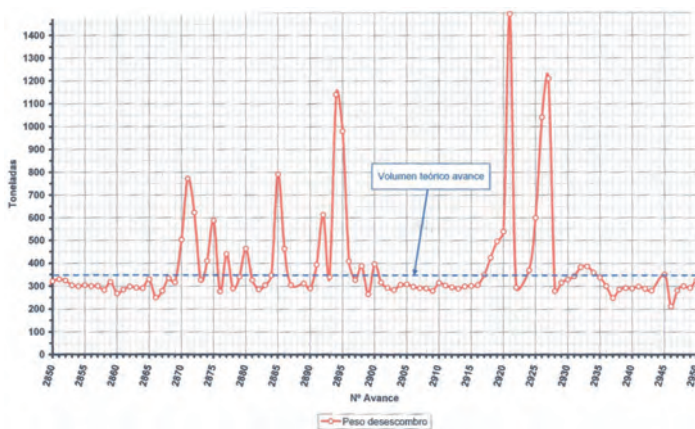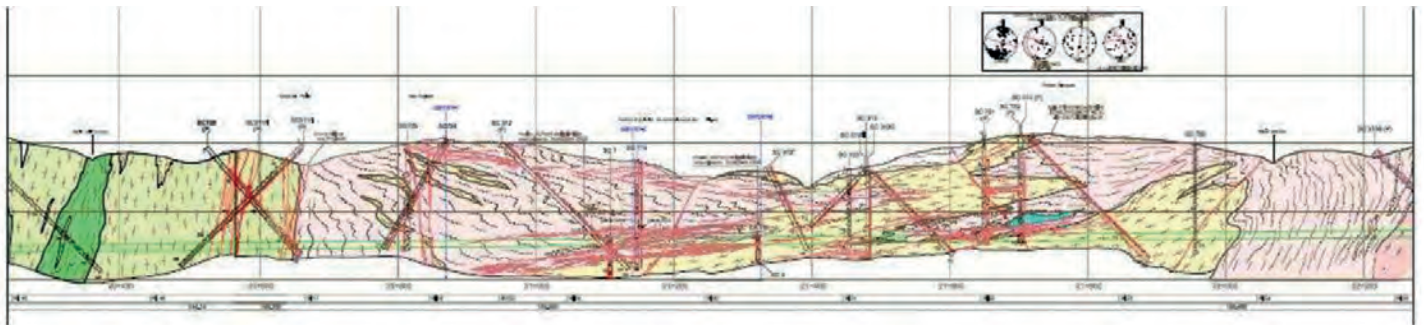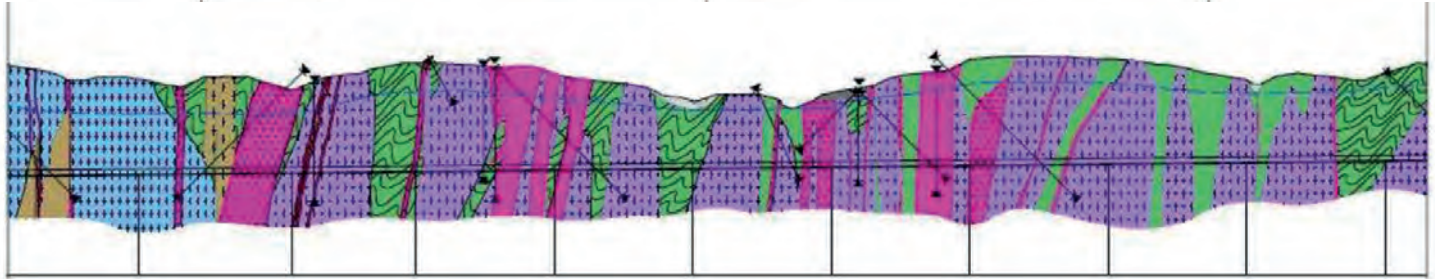
The Perthus Tunnel on the TGV line Perpigan-Figueras is a twin-line railway tunnel between France and Spain under the Eastern Pyrénées.
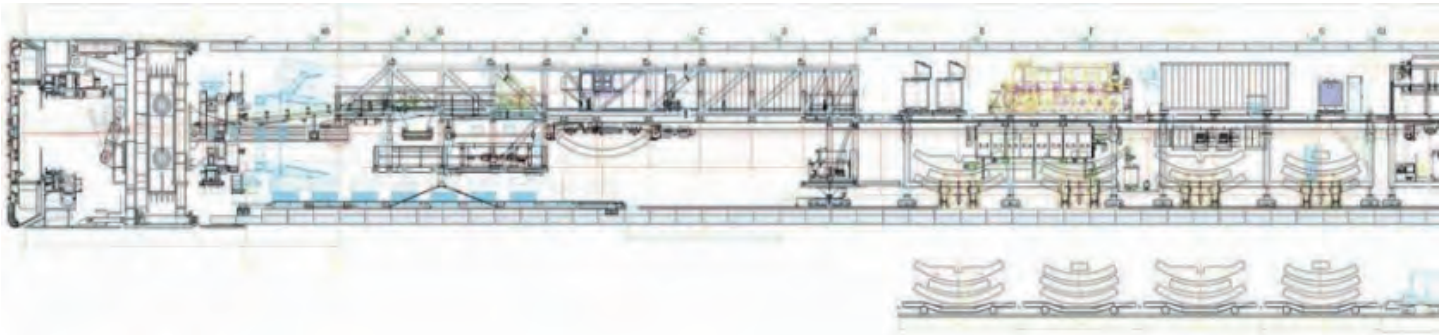
Figure 21: Tunnel du Perthus.

This 8.3 km long twin-tunnel excavated by two open double shield Herrenknecht TBMs with a diameter of 9m.
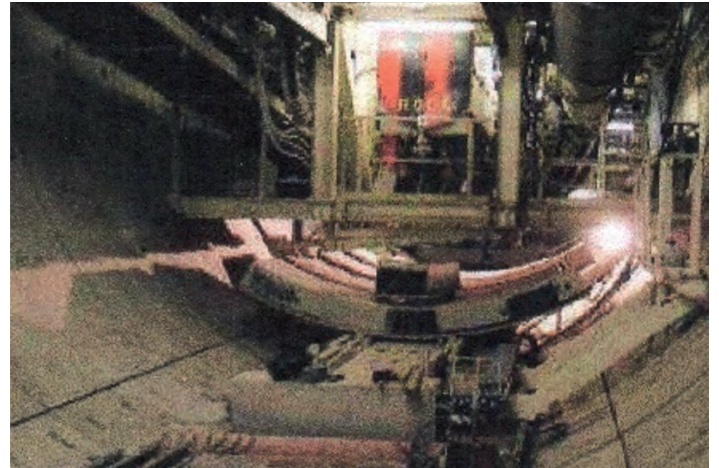
The tunnels were driven in granites, granodiorites, gneisses, and schists.

The main structural features expected from the geological investigations were subvertical faults. In fact, over a length of about 500 m the tunnels encountered a flat lying tectonic shear zone in a formation made of fractured gneiss and band of clay material.

The face instability caused caves-in; one of them went up to the surface 125 m above.

Several rings of concrete segments were ovalized and heavily fissured. The mucking corresponding to every ring was weighted. The measures clearly show several zones with the formation of caves-in.

The cavities were filled by grouting large quantities of polyurethane foam.

These incidents increased substantially the tunnel construction duration.

In high squeezing rock conditions, the shied TBM may be blocked by the ground stresses acting on the shield.

Such was the incident which occurred in the Tabellout hydraulic gallery in Algeria. It is a 12.4 km hydraulic transfergallery between the Tabellout and Draa ed Diss dams. It has a 2 m diameter circular section. It was driven in a flysch formation alternating marls and sandstones. The single shield TBM was trapped in highly fissured sheared marls at PM 1465 under a cover of 200 m despite the use of the maximum thrust available, 26000 kN. The concrete segments of the lining were heavily fissured from PM 1368 to 1450.

From the GSI value, the uniaxial compressive strength of the sheared marls may be estimated between 500 kPa and 700 kPa. With a cover of 200 m the initial stress is about 4500 kPa. Then the N value is estimated between 13 and 18 corresponding to very high squeezing conditions.

To free the trapped TBM, two lateral adits along the shield and ahead of the cutter wheel were excavated.

Figure 22: Tabellout Gallery (Algeria).

My last example will discuss the feasibility of the Gibraltar Strait Tunnel project under study for several years. The project foresees a 38 km long twin-bored tunnel with an undersea section of 28 km. The tunnel will mainly be driven in various Flysch formations. The presence of two deep paleochannels filled with clayey breccia may compromise the feasibility of the project.
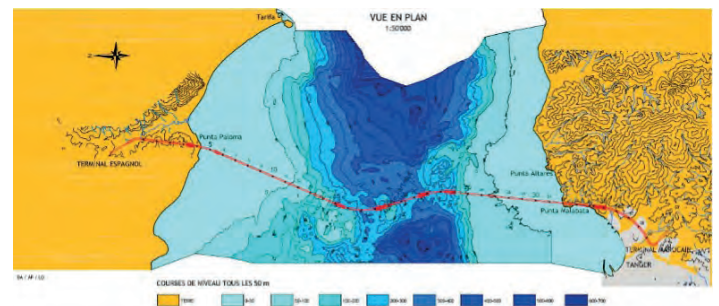

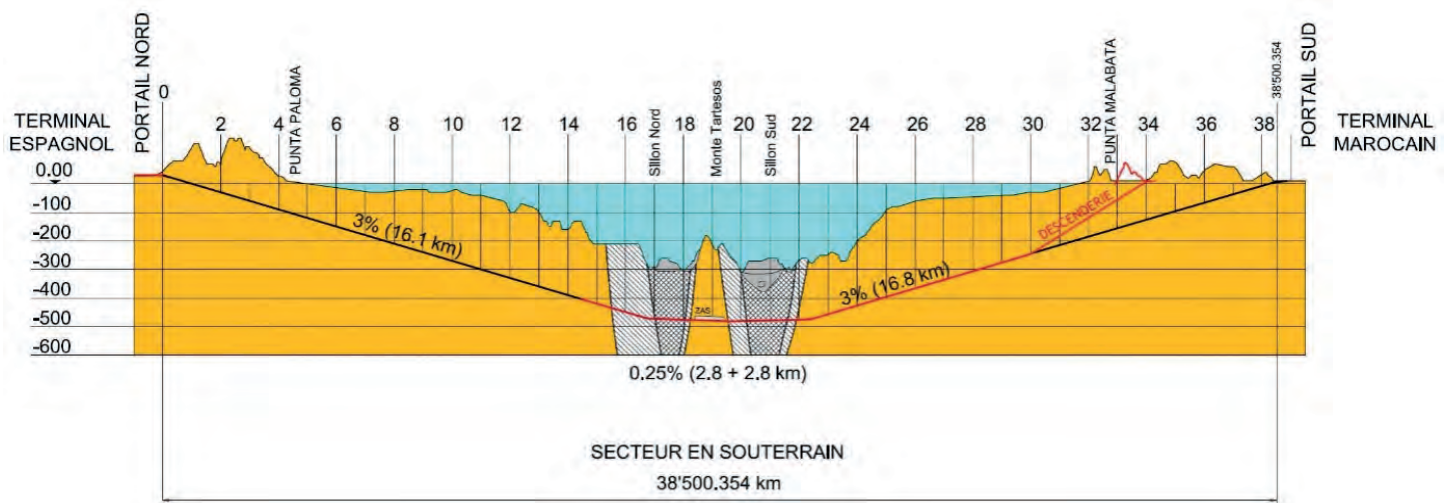
Figure 23: Gilbraltar Strait Tunnel.

Figure 23: Gilbraltar Strait Tunnel.

These channels were eroded in Eocene during the Messina crisis when the water of the Atlantic Ocean flushed down in the empty Mediterranean Basin. The shape and the depth of these two channels are not precisely known. The geological investigations in the Strait are made difficult by the sea depth and currents as well as the ship traffic.

The clay breccia filling the channels are rather heterogeneous. The very low permeability of the soil and the very high pore water pressure make the testing of the samples rather complex. From the laboratory tests it may be assumed that the permeability coefficient is about 10^{-11} m/s, the cohesion comprised between 100 kPa and 200 kPa and the internal friction angle of the order of 20°.

The sea depth above the channels is 300 m and the ground cover above the project alignment is 200 m. A rough approximation of the undrained uniaxial compressive strength of the breccia may be 450 kPa. These data give a stability number N of the order of 150. Considerable face instability and extremely squeezing conditions may be anticipated.

To cope with these conditions, a drainage ahead of the face has been studied by Lombardi to consolidate the breccia ahead of the face. However, the time even for a partial consolidation may be too long to allow a reasonable rate of advance of the TBMs. The time for consolidation is proportional to the coefficient of permeability and there is an uncertainty about this parameter larger than ten.

5 >> CONCLUSIONS

The various cases studies of the difficulties encountered in deep tunnel construction presented in this conference clearly show the necessity of evaluating displacements and instabilities which occur in the vicinity of the excavation face in order to adapt accordingly design and selection of the most appropriate excavation method. The geological, hydrogeological, and geotechnical investigations are of utmost importance. However, for deep tunnels, data from these investigations are often not sufficient and at the time of construction many uncertainties remains. They cannot be solved by further investigations. Therefore, during the excavation, the geological and geotechnical conditions should be constantly reevaluated by a continuous geological drawing, and the deformations in the vicinity of the excavation face should be monitored (observational method). In some cases, boring ahead of the face proves to be necessary to solve ground uncertainties which might stop the progress of the work. Most accidents described above are due to unanticipated ground conditions. The unforeseen geological and geotechnical conditions give birth to most of the contractual disputes between the owner and the contractor.

This conference gives me the opportunity to try to sort out lessons from about sixty years of tunnel engineering from rock mechanics studies of the violent rockbursts observed in the Mont Blanc Tunnel in the early sixties to recent advices expressed as a consultant for the Grand Paris Express Project. During these years, I have been privileged to work with many organizations and individuals. Together we have tried to achieve the best for the design and construction of tunnels. They are too numerous to be cited but I am indebted to them for their contributions.

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