NUMERO: 2144A- ANNO: 2017

A P P U N T I

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MATERIA: Tunnelling - Prof. Peila.

GENERAL ASPECTS OF TUNNELLING

1 What is the role of tunnels for engineering?

Tunnels are realized for many scopes.

→ roads, railways, metro

→ hydraulic tunnels for electricity production, clean water, sewer (= "fogna"), collection of the first water when it rains - dirty water

→ underground services

→ hydraulic tunnels to lower down the water table and stabilize a landslide, in case of big landslides where the problem of water level is important.

In this way, we can solve different problems with tunnels.

The first tunnel was excavated in ancient Greece for hydraulic purpouse and was a tunnel 1 km long, excavated by hand.

Nowadays, tunnels are realized in different context, with different sets of problems.

→ tunnels outside the city, with problems of stability of excavation and interference with water

→ tunnels inside the city, with problems of interaction with urban areas

2 DIFFERENCES BETWEEN SURFACE CONSTRUCTION - UNDERGROUND CONSTRUCTION

→ above ground construction

→ MATERIAL:

during the construction, we add material.
The material has known behaviour and it can be controlled and the material is used for purpouse - we want to use it.

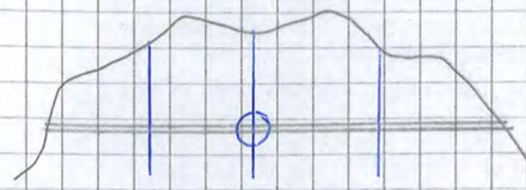
→ LOAD:

the load to which the structure is subjected is known and the design is performed accounting the load.

4 Principles of underground excavation

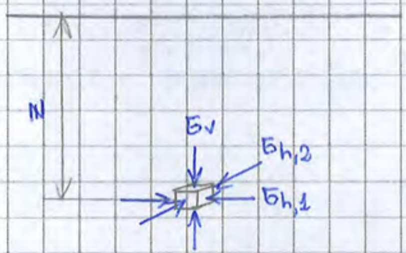
Tunnel excavation is done in already stressed rock mass.

The evaluation of underground in situ stress is important but it is difficult to take measurements.



Investigation of in situ stress state requires to go inside the ground and we can use only drilling from the surface - geophysics is useless for this purpouse.
On the other side, we are able to do a limited number of measurements of stress state and, even if measurements of good quality, they do not give a complete idea of stress state.

Thus, excavation is done in unknown rock mass and in unknown stress state.
This makes the design very complex and we should make ASSUMPTIONS.



Generally, given an element at depth z from the surface, we assume a tri-axial stress state with

1 vertical stress component σ_v

2 horizontal stress components $\sigma_{h,1}$ $\sigma_{h,2}$

For the sake of simplicity, we make some assumptions.

→ the two horizontal components may be different, but in geotechnical engineering they are assumed as equal

$$\sigma_{h,1} = \sigma_{h,2} = \sigma_h$$

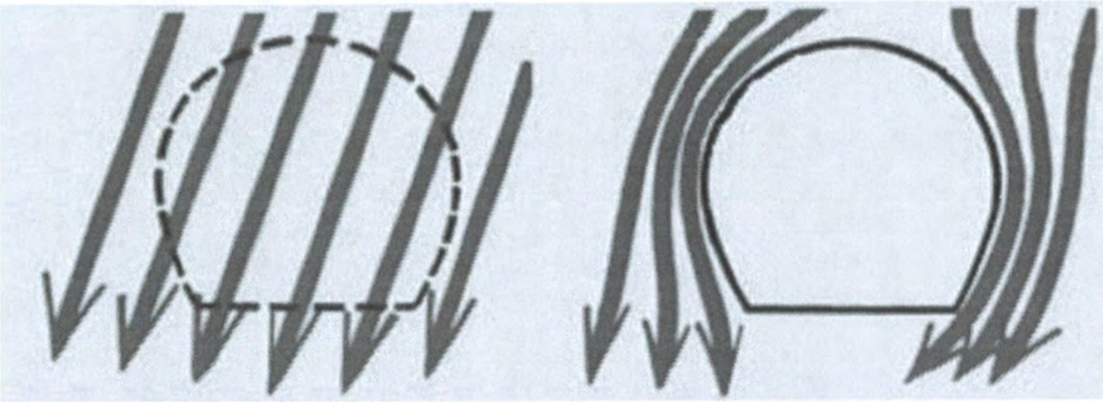
→ the vertical stress state is done by gravitational load.

$$\sigma_v = \gamma z$$

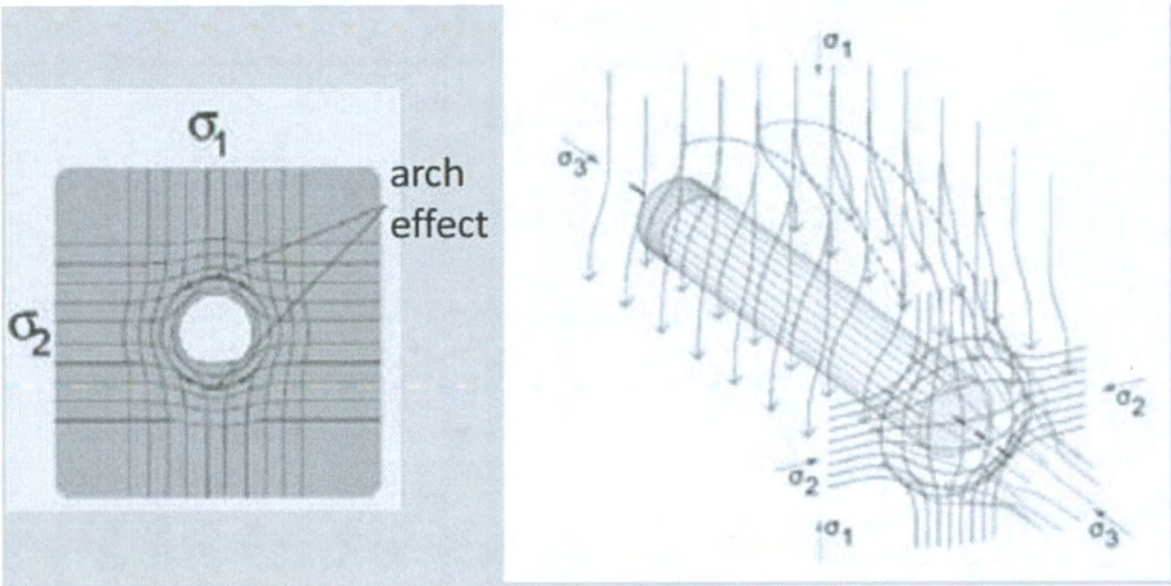
→ the horizontal stress state is linked to the vertical stress state by means of the rest coefficient k_0.

$$\sigma_h = k_0 \sigma_v$$

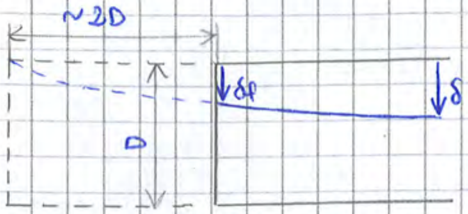
→ the horizontal and vertical stresses are principal stresses.



Redistribution of natural stresses



3D effect of excavation



Radial displacement starts ahead of the excavation face, at average distance of

2D

depending on geological conditions and in situ stress.
~~By con~~

By consequence, at the excavation face we have already experienced certain radial displacement and, in elastic conditions, this is equal to 30% of the final displacement.
That's why we place displacement instruments at the excavation face to get measurements.
To control face convergence, we should do something ahead of the face.

To get some indications on what happens at the excavation, we may merge together IN SITU STRESS STATE and ROCK MASS FRACTURING DEGREE, seeing different behaviours of the excavation

→ stable conditions in good rock mass under low natural stress state

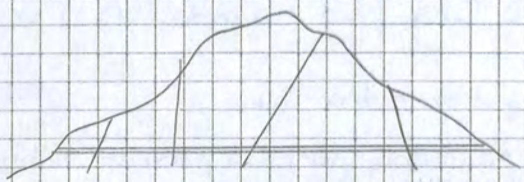
→ unstable wedges in case of fractured rock mass under low natural stress state

→ rockburst in case of stiff and few fractured rock mass under high natural stress state - deep tunnel.
Rockburst ("colpo di tensione") is a sudden release of elastic energy and gives rise a seismic effect, with the collapse of the tunnel.

→ squeezing, in the intermediate case.
Squeezing consists of large plastic displacements, from few mm to tens cm.

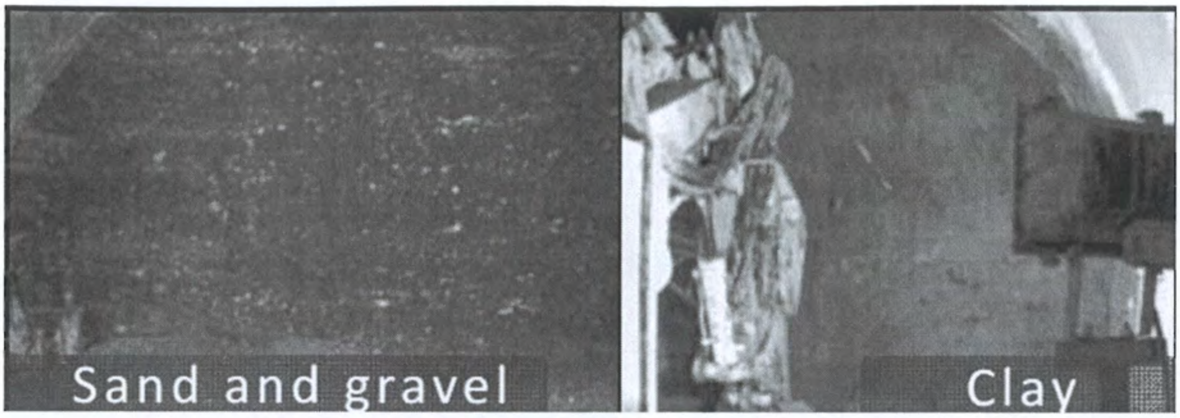
→ geological properties
→ in situ stress state
→ excavation process

By consequence, one goal of design is the understanding of the ground behaviour and its answer to tunnelling operations.
This can be done with many different procedures and we have to choose the best design procedure, taking into account global conditions.
+
the second goal is the correct assessment of the position of the rock masses along the alignment.

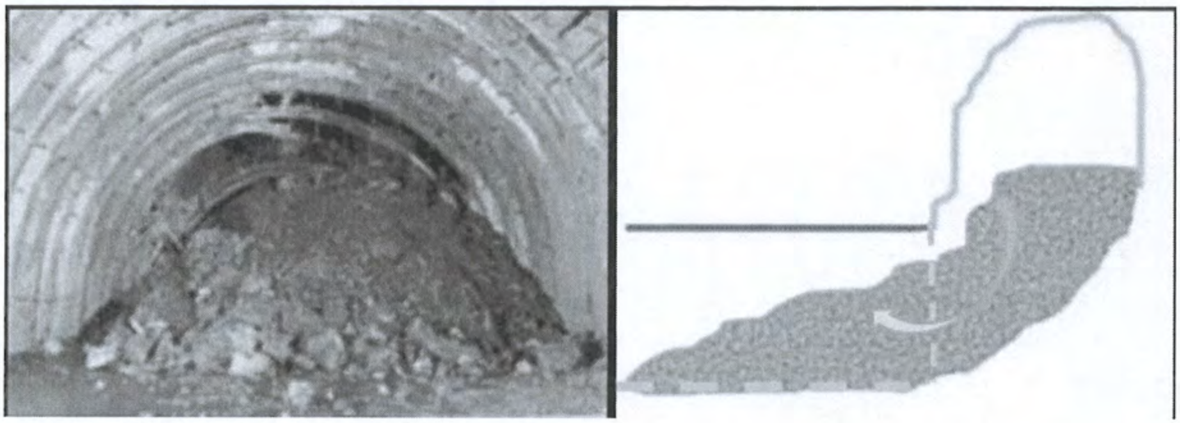


Indeed, if we have different types of rock masses, we have to understand where we encounter them, because they behave in different way and we should face them in the best way.

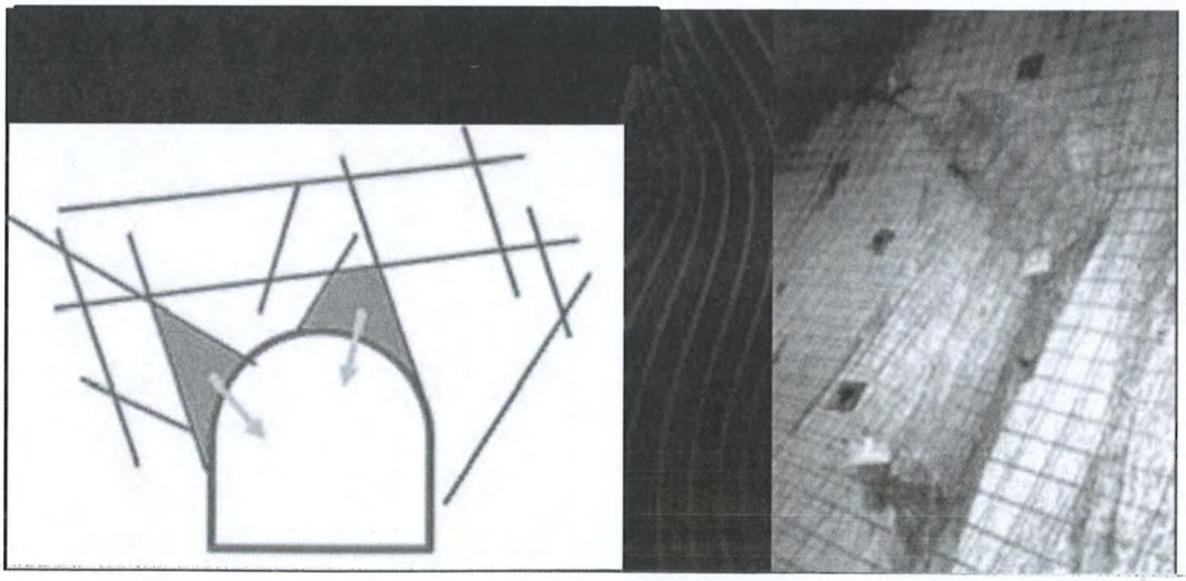
In order to assess geological properties and the position of rock masses, PRELIMINARY INVESTIGATION plays an important role.



Overburden soil or heavily weathered rock



Fractured rock mass with not stable blocks



Massive rock mass with few joints

In this condition, we should study the behaviour by using different methods than in other situations. Indeed, if global displacement is evaluated with numerical methods, stability should be evaluated with LIMIT EQUILIBRIUM METHOD.

On the other side, the detachment of blocks may create a new free surface and new wedges may start to collapse. This brings to systematic collapse and the creation of big holes.

Generally, we have to introduce elements stabilizing blocks, e.g. bolts, and we do not need a systematic system of support. Actually, for the sake of simplicity, we use a systematic system of support, maybe improved in some sections.

→ MASSIVE ROCK MASS UNDER HIGH STRESS CONDITIONS

If rock mass is quite plastic, we can have SQUEEZING, with high convergence and regular displacement without collapse. Otherwise, we can have ROCKBURST.

From the general point of view, the stability conditions of a tunnel are controlled by

→ GROUND GEOTECHNICAL PROPERTIES

→ IN SITU STRESS STATE

→ SHAPE OF THE TUNNEL SECTION:

in case of hydrostatic stress, the best shape of the tunnel is circular, since support will not have bending moment.
If corners are present, we have concentration of stresses there.

→ SIZE OF THE TUNNEL SECTION:

the bigger is the tunnel section, more difficult will be the control of deformation.

→ PRESENCE OF UNDERGROUND WATER:

underground water influences stability conditions, work environment and brings some environmental aspects. For instance, the drainage of water may affects drinking water shafts or springs or the excavation may pollute underground water.

→ CONSTRUCTION METHOD, which is intended to control deformability and stability.

Tunnel excavation methods

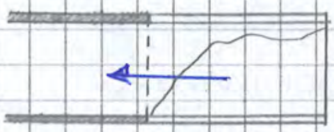
We can divide excavation methods into two cathegories.

① Conventional method

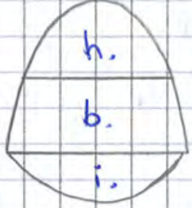
It is the hystorical method, in which excavation is carried out in a CYCLIC PROCESS.
In the cycle, we have an excavation phase for a certain advancement step, removal of excavation muck and installation of the support.
Inside the cycle, we can also install presupports ahead of the face.

→ Drill and Blast ("Scavo con esplosivo")

A drilling machine, called Jumbo, realizes small drilling holes at the excavation face, required for the blast.
Then there is the blasting, the muck is removed and the support is installed.



Conventional excavation is carried out in full face excavation or partialized face excavation.
In this case, the excavation is performed at 3 levels.



→ head = "calotta"

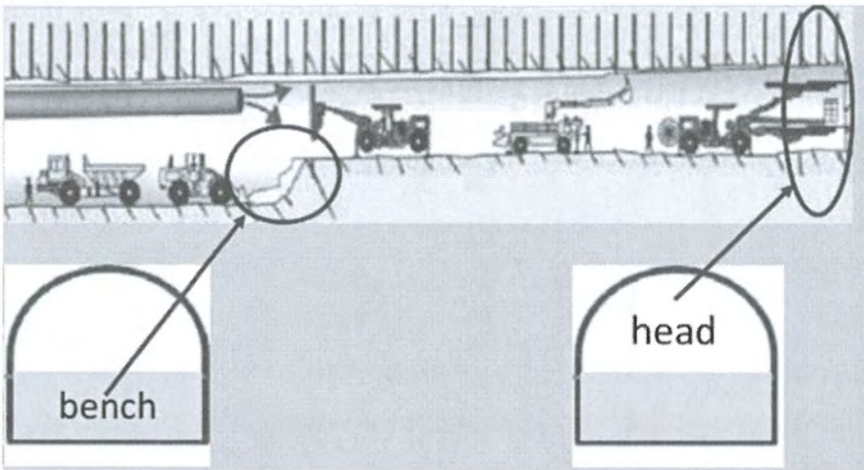
→ bench = "ribasso"

→ invert = "arco rovescio"

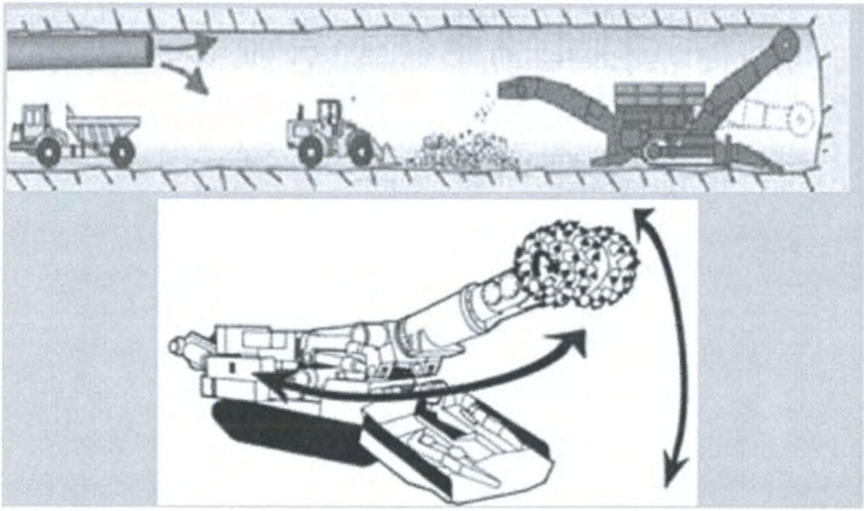
Historically, in weak rock they tent to adopt the partialized face technique, since an excavation with smaller size allows easy control of displacements and stability.
Nowadays, the trend is to use full face excavation, with face bolts.

→ Punctual machines

We use a mechanic tool, called ROADHEADER (= "fresa puntuale").
It is a rotating cutter head, carried out by a boom and moved at the excavation face. By rotation, it excavates the soil and this passes in a system, going through the machine to be carried out behind it.
The technique is good in easy excavating rock masses, e.g. gipsum, coal, marl, weak limeston → the cutter head scratches the rock mass.



Drill&blast



Roadheader

Mechanized excavation

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II Full face mechanized method

The basic principle is that excavation is carried out in a CONTINUOUS PROCESS, with low ciclicity.
Indeed, the steps of excavation, removal of the muck and support installation are done by the machine at the same time, with short stops.

→ **Rock TBM**

Excavation is done by a cutter head, carrying balls rotating around a neck, which break the rock in small parts and the elements are then transported back.

→ **Shield**

The shield is a steel cylinder, protecting the machine and workers. Below the protection of the shield, the final lining is installed.
The machine is good for soils and it is also able to apply a pressure on the face, granting face stability.

Rock TBMs are used only in good and stable rock mass but, nowadays, the distinction between TBM and shield is going to become more weak, since TBMs are now coupled with a shield element.

2 Assumptions on in situ stress state in rock mass

In practical applications, we estimate in situ stress state through two assumptions.

→ the vertical component σ_v is due to the weight of the overlaying rock at that depth and it is related to the unit weight γ of the rock.

$$\sigma_v = \gamma z$$

The horizontal component is uniform and computed as

$$\sigma_H = \sigma_h = k_o \sigma_v$$

→ BOTH THE HORIZONTAL AND VERTICAL STRESSES ARE PRINCIPAL STRESSES.

The coefficient k_o may be assumed as

→ AT BIG DEPTH

$$k_o = 1$$

→ AT SMALL DEPTH

$$k_o = \frac{\nu}{1-\nu}$$

This last expression is correct in strata of sedimentary rock in geologically undisturbed regions, provided these strata have never carried a heavy temporary load.

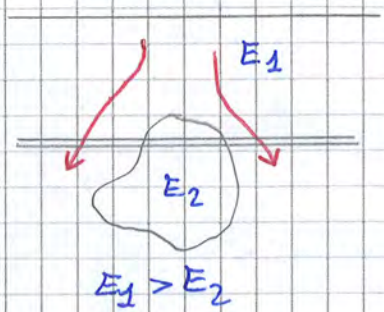
Actually, measurements have shown that horizontal stresses differ from those predicted by the assumptions.
Furthermore, many cases show that horizontal stresses exceed the vertical stress and, generally, we have non uniform horizontal stresses.

4 Factors influencing the in situ stress state

In situ stresses can vary in magnitude and orientation due to several factors.

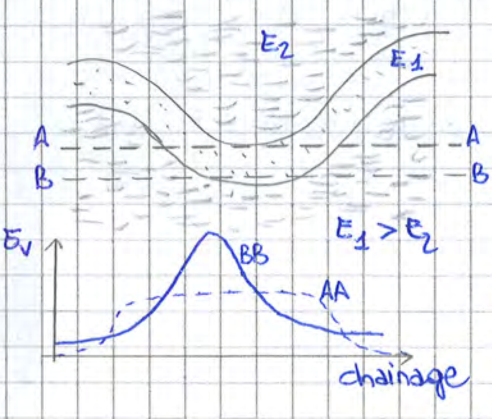
→ stratification and heterogeneities

If there is an inclusion with a Young modulus very different from the one of the embedding rock, stress state will be linked to Young modulus itself.



In this situation, stresses will concentrate in the stiffest zone, whereas the inclusion is less stressed.
By consequence, when we excavate a tunnel, we pass from a more stressed to a less stressed zone.
Thus, we need to take care in investigation and know the geology, because vertical stress can be bigger.

The disturbance of the in situ stress field due to heterogeneities can be understood using the analogy of a solid inclusion embedded into an infinite medium.



A different case is the one of a folding.
Considering the alignments AA and BB, stress state is different and it is smaller in the alignment AA.
Indeed the folding creates an important concentration of stresses in the bottom zone and this leads to a different response of the rock mass.
By consequence, in folded rock masses, the in situ stress field at given depth should not be expected to be uniform, even if the ground surface is horizontal.

→ geological structures, as faults, dikes and shear zones.

As those rock masses tend to relax under reduced load or temperature changes, restraints are created by the interlocking fabric of the rock itself. The rock tends to reach a new equilibrium with balanced internal - tensile and compressive - forces.
The presence of residual stresses can be critical for the stability of underground openings, since may induce rock bursts and spalling.

In this situation, we require more investigations or we should run a simulation in order to understand the role of residual stresses and study the stress distribution.

5 In situ stresses in soils

In this case, we can use some relationships in low overburden tunnels.

→ NC SOILS, in which we consider the friction angle φ'

$$k_{0,NC} = 1 - \sin\varphi'$$

→ OC SOILS, in which we consider the overconsolidation ratio OCR.

$$k_{0,OC} = k_{0,NC} \cdot OCR^{\alpha}$$

α = empirical coefficient

Guidelines for tunnelling design

We should follow a checklist, in order to make a correct tunnel design.

I. General setting of the underground work

Ia) General setting of the work and its relationship with the general design

Generally, the tunnel is part of a bigger project and we should take into account the relative position of the tunnel linked with the general work.
For instance, the position of the tunnel is forced due to the general setting of the work and the generale frame of the project, i.e. transport conditions.

This element includes

→ functional requirements, with location, route, interference and definition of the geometry of the work.

→ design constraints, due to local regulation

→ environmental aspects, e.g. the water drainage

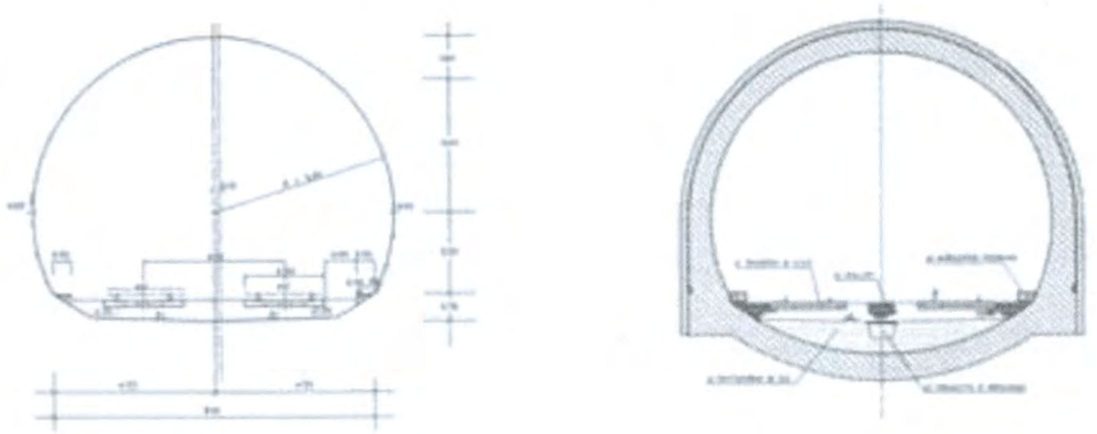
→ comparative analysis of alternative routes, in order to find the best position, if possible, based on technical and economical comparisons.
The solution should also minimise interference of new works with normal operations of existing infrastructures, by means of suitable construction techniques or appropriate scheduling of the works.

Important aspect are

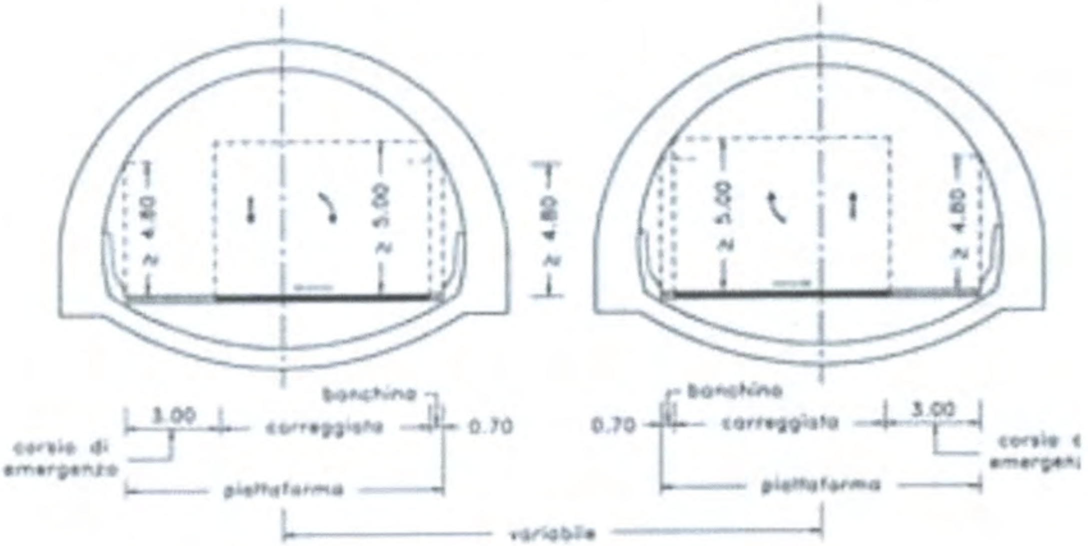
→ NUMBER OF ATTACK POINTS, e.g. shafts, in long tunnels. Indeed, with different attack points, we can attack the excavation from different points and the length of excavation is reduced into different sections.

→ in conventional tunnelling, it is better to EXCAVATE UPWARDS to facilitate the natural removal of water. Otherwise, we should design the plant to carry out water with pumps.

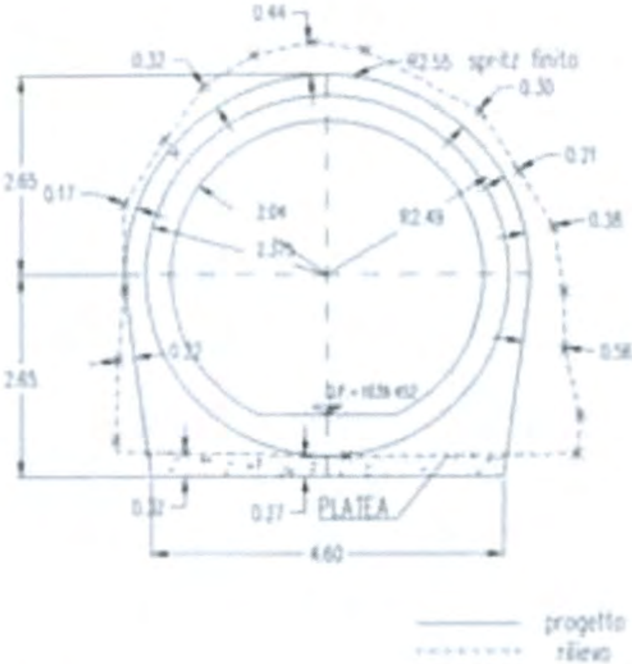
TUNNEL CROSS SECTIONS



Railway tunnel – Manuale Italferr (1995)

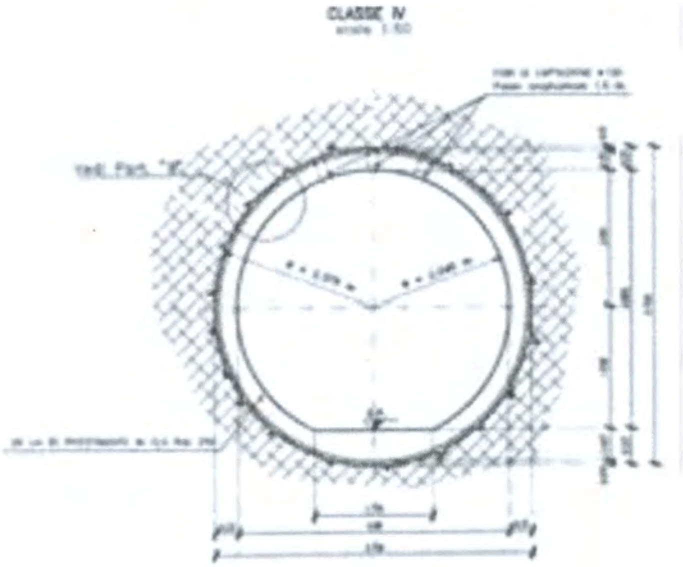


Highway tunnel – Italian DL 285 del 1992



Galleria Pont Ventoux – Hydraulic Tunnel

Tunnel excavated by D&B



Galleria Pont Ventoux – Hydraulic Tunnel

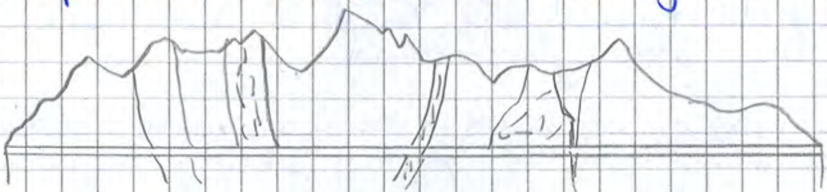
Tunnel excavated by TBM

The geological survey is performed in several steps.

(IIa) Acquisition of available data, from literature (books, articles, maps, aerial photographs, geophysical data, wells and water springs)

(IIb) Preliminary geological model

It is practically a map containing information about structure, stratigraphy, geomorphology, hydrology and hydrogeology along the alignment.
This provides information for investigation and design.



(IIc) Site investigation, which consists of direct investigations and indirect investigations and should be planned:

→ evaluation of interaction with geotechnical and geomechanical investigations

→ planning of investigations, in type, location and analysis of results

(IId) Final geological model

This is the result of the coupling of preliminary geological model and site investigations.
It contains structural setting, lithostratigraphy, mineralogy and petrography of the ground.
Mineralogy is important because, in tunnelling, we worry with some minerals like sulfur, gipsum, quartz, etc.

Furthermore, we should define the reliability of the geological model.

(IIe) Definition of geomorphology, i.e. the system of faults.

(IIf) Hydrology and hydrogeology

This study is related to the fact that tunnel construction may affect hydrogeologic regimen and pollute ground-water.
We have to focus on different aspects.

→ general hydrology and hydrogeology, in terms of hydrological context, features of hydrogeological structures, underground water regime and evaluation of hydrogeological risk (influence on regimen and pollution).

→ water chemistry.

→ structure - acquifer interaction.

→ presence of other fluids, like methane - in this case, we have to change the machine.

(IIg) Geothermal studies

The investigations and studies shall allow the determination of the thermal gradient, presence and flow of geothermal fluids and ground temperature in the tunnel.

(IIh) Seismicity

The topic is important for shallow tunnels and at portals, where seismic effect may trigger a landslide.
Generally, deep tunnels are more resistant to seismic action than above ground structures, except in correspondence of seismic faults.

The evaluation of seismicity of the area is made with literature review, seismic classification and identification of seismic structures.

III c) Soil or rock mass characterisation

In the analysis, we should take into account that rock mass is a mix up of intact rock and joints and we need to study the two components.

→ ground structure, defining the type and frequency of main discontinuity systems.

→ soil or intact rock characterisation, defining geotechnical and geomechanical parameters and rock mass technical classification.
The classification systems give indications about the type of support but not for design, since they give the behaviour of rock mass and the average mechanical parameters.

→ mechanical characterisation of discontinuities, defining shear strength and deformability parameters.

→ hydraulic properties of the soil or rock mass

→ geomechanical classification of rock mass

→ geotechnical and geomechanical model, with physical and mechanical properties from literature, quantification of scale effect, behaviour models and failure envelopes and geotechnical models of soil and characterisation of the different groups.

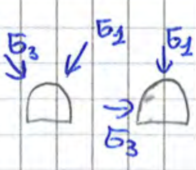
→ geotechical and geomechanical zoning (HOMOGENEOUS ZONES)

Along the tunnel alignment, we have to perform a subdivision not only from the geological point of view but also in sectors of rock mass having the same geomechanical behaviour.
This is an important setting because different supports will be assigned to different portions of alignment.

In the zoning, we have also to consider

→ hydrogeological conditions, since the behaviour of the zone above and below the water table is different not in geology but in machinery.

→ excavation geometry, e.g. enlargement for parking area, which may be critical

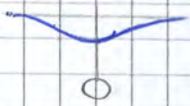


→ depth of excavation and stress state - entity and rotation

IV c) Surface and underground conditions

→ EFFECT OF UNDERGROUND EXCAVATION ON THE SURFACE

We should evaluate the effects on surface, i.e. in structures and landslides, with the definition of acceptable displacements for structures and limits for relaxation of slopes.



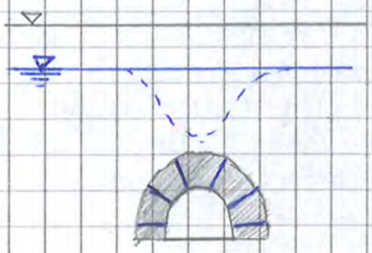
Then, we evaluate subsidence - described with a Gaussian curve, having maximum on tunnel axis - and its effects on buildings

→ EFFECT OF EXCAVATION ON THE SURROUNDING MASS

We should define acceptable stress-strain variations on the underground structures and on the surrounding mass and compare with the actual ones, in order to determine the general working criteria.

→ EFFECT OF TUNNELLING ON EXISTING HYDROGEOLOGIC EQUILIBRIUM

From the hydrogeologic balance within the mass, we have to estimate the flow rates induced by excavation and the impact on wells and springs and subsidence.



For instance, if a good rock mass is characterised by a water table, we have not problems of stability and we have to drain water through cracks.
If drainage is not an acceptable solution, we should do something on cracks, injecting mortar in the joints.
Yet, this choice implies bigger cost and variations in the construction cycle.

By consequence, this effect changes tunnel design.

VI

VI a) Design of auxiliary works

→ DESIGN OF PORTALS

The portal is the entrance and this is the only portion in which we have architectural aspects. This is a transition zone, in which structural design is not simple and we should take into account of seismicity.
This is coupled with the design of ventilation systems.

→ MONITORING PLAN

The plan should include monitoring during construction, in order to check if design is correct, and monitoring during operation, which is long lasting monitoring.

→ DISPOSAL AND BORROW AREAS

In excavations, we take part lots of material and partially it can be reused, but partially will go in waste disposal. By consequence, we have to design a system to carry the muck in the disposal system.
On the other side, we have to evaluate the quantity of concrete.

VI b) Tender documents → tender = "gara"

They are technical documents regarding the contract, since the document of calculations is not enough.
It includes

→ bill of quantities ("computo metrico"), in terms of m^3 of concrete, m^3 to be excavated, number of bolts, etc.

→ cost list ("elenco dei prezzi")

→ technical specifications ("Capitolato Speciale d'Appalto"), where we write all the types of work and details, i.e. material, standards to be fulfilled, etc.

→ quality plan design

Finally, we define the plan for safety and construction.

DRILL AND BLAST

Explosives and initiation devices

1 Drill and Blast:

it is a technique of excavation which uses explosive to demolish the rock mass and create the hole!

2 Explosives

Explosives are classified into 2 big cathegories

→ LOW EXPLOSIVES:

they are not able to detonate and they just deflagrate from an initiation, e.g. black powder.
They are not of interest for our purpouse.

→ HIGH EXPLOSIVES

→ PRIMARY EXPLOSIVES:

primary explosives are used just as initiator of secundary explosives.

→ SECUNDARY EXPLOSIVES:

it is the biggest cathegory and includes

> MILITARY EXPLOSIVES

> INDUSTRIAL EXPLOSIVES, which are of our interest

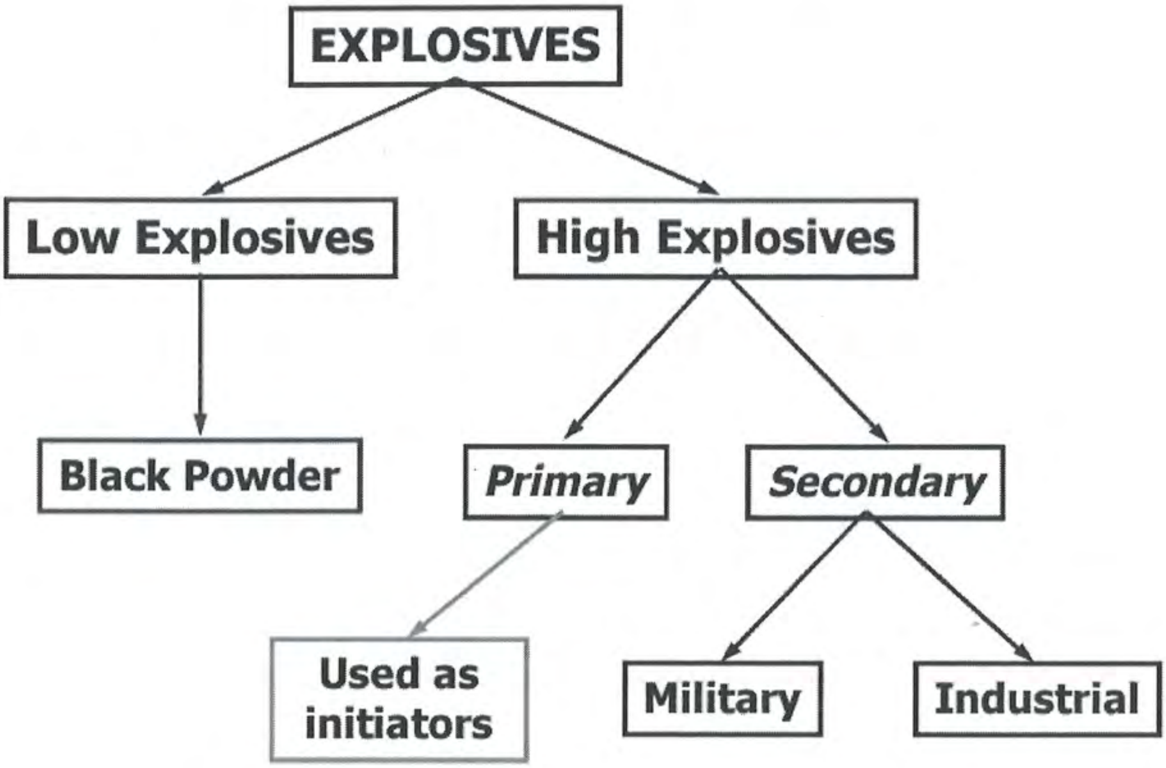
In principle, to use secundary explosives, we need a device able to initiate them, that is primary explosive.

After initiation, low explosives just react developing in big amount of gas - gas energy.
High explosives, on the other side, react producing gas energy together with the so-called SHOCK ENERGY. This is due to the high velocity of reaction, since the detonation velocity V_d is

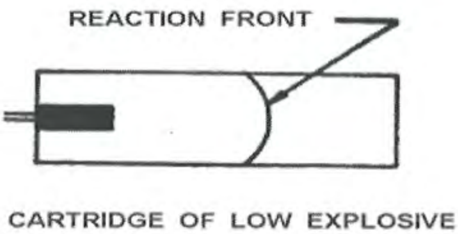
$$V_d = 2000 \div 9000 \ m/s$$

The important aspect of high explosives is that they are very fast and this develops high pressure in the detonation front and it is

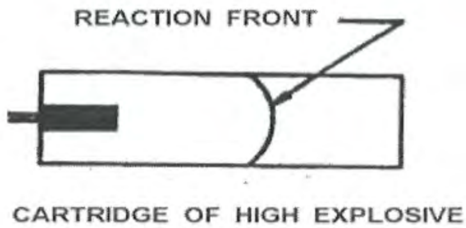
CLASSIFICATION OF EXPLOSIVES

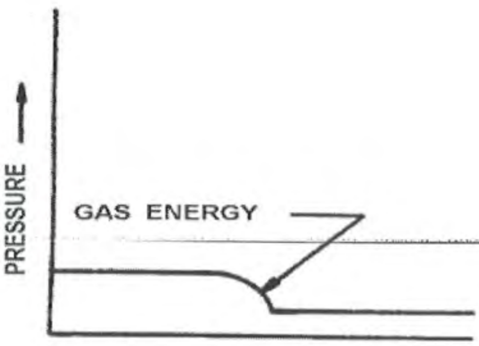


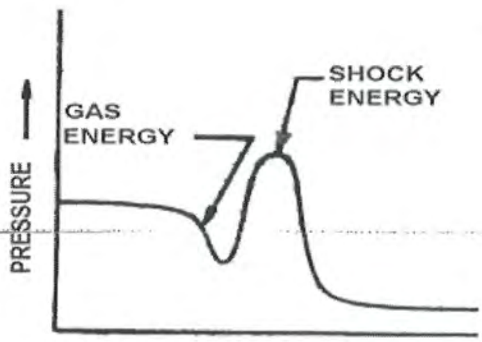
Low explosives



High explosives







In this process, the impedance of the rock mass should be smaller than the impedance of the explosive, velocity of propagation of the detonation should be able to crack the wanted amount of rock mass.

$$I_R < I_e$$

→ Detonation velocity

It is the velocity at which detonation progresses through an explosive and it is a characteristic of the explosive.

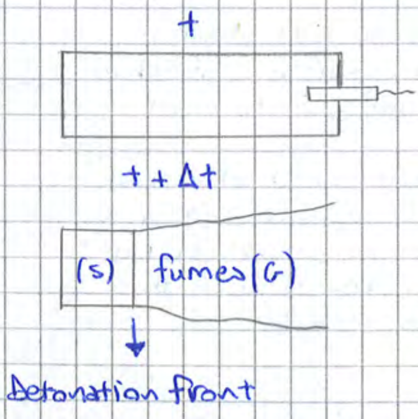
Dynamite	$v_d \leq 7000$ m/s	→ better in hard rock
EMULEX	$v_d \leq (5500 \div 6000)$ m/s	

→ Density

→ Water resistance, which is important because in underground there is water and explosive should resist to it.

→ Detonation pressure, expressed as

$$p_d \sim 0,25\, \gamma_e\, v_d^2$$



Indeed, at time t, we have the initiation of the cartridge from the initiation point.

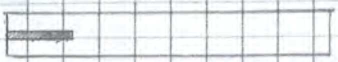
After time Δt, part of the explosive is decomposed and we can define a detonation front, separate solid explosive and gas. In the gas, a pressure p_d is acting and we should take it into account. The pressure is given by gas density, detonation velocity and a parameter proportional to the velocity of development of gas product. This is not easy to be evaluated and, after several site experiments, it was approximated as $0,25\, \gamma_d$.

→ Impedance

→ Sensitiveness to the device responsible of detonation.

6 Initiation systems

In order to initiate secundary explosives, we use a device called detonator, which is practically a primary explosive.



The detonator is assembled to the cartridge of secundary high explosive, called BOOSTER. Typically, it is placed inside it.
The assemblage of detonator and booster is called primer.

It is not rare to find in the blast hole a configuration of a hole with certain length and the primer located at the bottom.

The other cartridges may be made of a different type of explosive, not necessarily dynamite but ammonium-nytrate-based explosives, e.g. slurry.
Indeed, we need a big amount of energy to initiate the column of the cartridges and the first part is responsible of transmission of the detonation to the second part, whereas the second part may have performance even smaller compared with the booster.

Actually, we do not fire a single blast hole, but we blast a sequence of several holes.
They are not blasted simultaneously, since we need collaboration among charges in order to get good fragmentation of the rock mass and we have to define the portions to divide the section and how organize the sequence of initiation.

⇒ it is not so much important the type of explosive, since it should be only water resistant device - dynamite or emulsion - but the way to organize the blasting, i.e. explosion timing.

To reach the pre-determined sequence of blasting, we can use different detonation devices.

→ electric detonators

They contain a primary explosive, which is sensitive to electricity but also to thermal shocks.
By consequence, if dynamite is stable, detonator are difficult to be manipulated and very sensitive and we ~~sh~~ have to take care not to accidentally detonate the explosive. Thus, we must work separately the primary and the secondary explosives and link them just before the initiation.

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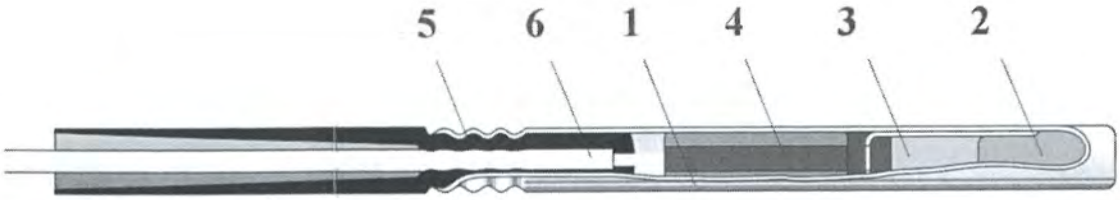
NONEL DETONATORS

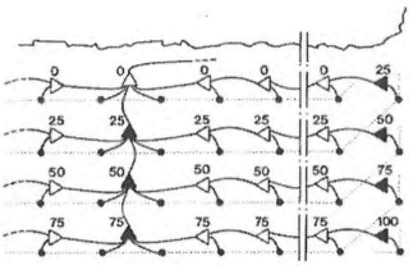


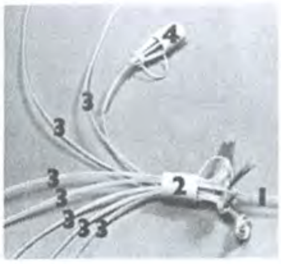
Detonators and connecting units

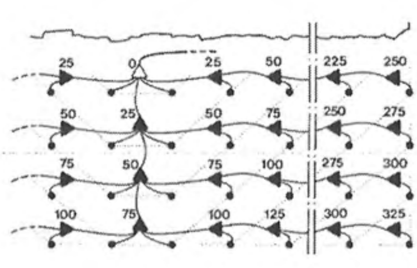
Long-delay detonators(500 ms)

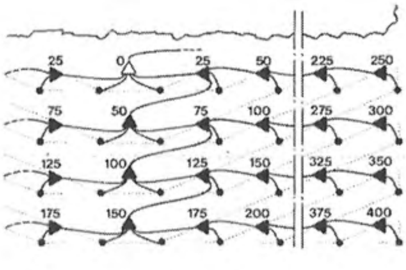
Sgort-delay units (usually 0, 17, 25, 42 ms)

Examples of arrangement of the connecting units









problems in the precision required to reach the results.
The precision of the delayed detonator for each kind of detonator, due to uncertainties in explosion time, is

$$\pm 8 \text{ ms}$$

As consequence, there is a GAP IN DETONATION TIME and we have to accept this unprecision, which is small but sometimes not negligible for our purpouses.

→ long delay, in which we have a number of different scales.

100 ms   200 ms   250 ms   500 ms

Due to the precision - not infinite - and the limited length of the delay element, in the detonators we have a limit in the number of available delays.

MICRODELAY	30 DELAY NUMBERS
LONG DELAY	18 DELAY NUMBERS if 100 ÷ 200 ms
	12 DELAY NUMBERS if 250 ÷ 500 ms

Electric device have to be organized following ohm law:

each electric device is given an electric resistance R and, closing the circuit, we have to exert a voltage V - available thanks to the explodittor - and an electrical current I is passing in the circuit.

$$V = RI$$

The electrical current can be of low, medium or high intensity.
In Italy, due to security laws, we are forced to have high current intensity detonators, in order to avoid accidental initiation.

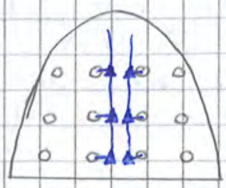
Low intensity	1,2 A	(not allowed)
High intensity	30 A	(allowed)

Of course, before blasting, we have to be sure that, in the circuit, the resistance R_d of the detonator computed is correct, through an ohmeter.
Theoretically, the resistance of the detonatore is

$$R_d = R_h + R_w$$

How the system works?

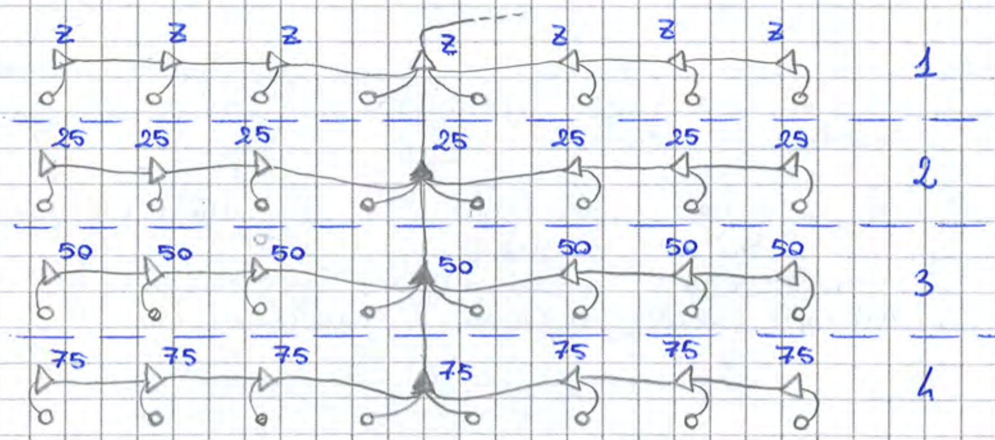
Inside the blast hole, we have a long delay detonator, with delay of 500 ms, coupled with a number of cartridges.
At surface, there is the short-delay device, with delay of 25 ms, and the tube starts from the plastic cap.



Actually, there is not an unique blast hole but, in a cross section, we have different holes.
From each hole, a tube is going out and tubes are linked to the connecting unit - the plastic caps - and one to each other.

In this way, in the blast hole there will be certain delay, whereas in surface there may be different possible delays.

For instance, we can use the same delay in each row and follow a certain sequence.
In the first row, the connecting units are linked to each other and they have no delay - white triangle.
In the second row, the central connecting unit has 25 ms of delay - black triangle - and it is connected to other ones, having no delay, and so on.

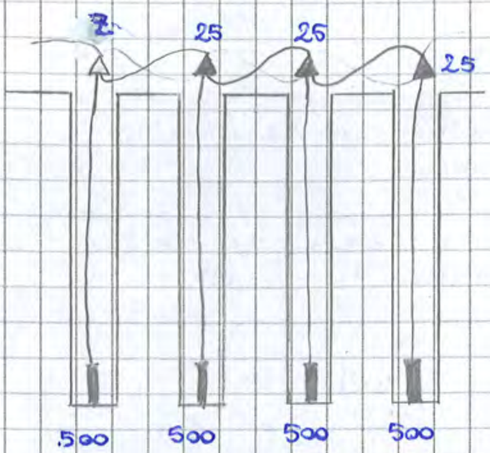


In this way, we obtain an ORGANIZATION OF DELAYS, where

→ in the first row, there is no delay from the initiation

→ in the second row, the central unit detonates at 25 ms and the other ones do at 25 + 0 = 25 ms

Observing the numbers, we can notice a SEQUENCE OF DECOMPOSITION OF ROCK MASS, in which each 25 ms a row is decomposed.



What happens at the bottom of the hole?

We have a connecting unit at surface and a long-delay detonator at bottom.

The initiation starts at surface and, following a certain sequence, each connecting unit is alerted, receives the ignition signal and detonates. Nothing happens at bottom because there is a long-delay detonator and all the connecting units are activated before the first blast hole detonates.

In this case, the blast holes detonate at 500 ms, 525 ms, 550 ms and 575 ms, since the connecting unit activation sequence is 0 ms, 25 ms, 50 ms, 75 ms and the detonator at bottom has a delay of 500 ms.

⇒ the sequence of detonation is organized on the surface and it dictates the explosion of detonators in contact against the rock, which break at least after 500 ms.

This techique allows to avoid risk of interruption of detonation on the surface because, when a borehole detonates, it will break a volume of rock. If the volume is broken before the sequence is accomplished, we have interruption of the detonation, since wires can be broken and detonation can not go on.
Thus, if we can dictate the sequence of detonation from the surface - instead from the bottom -, we will be able to dictate a good sequence on surface and it will be exausted before the first detonator detonates.

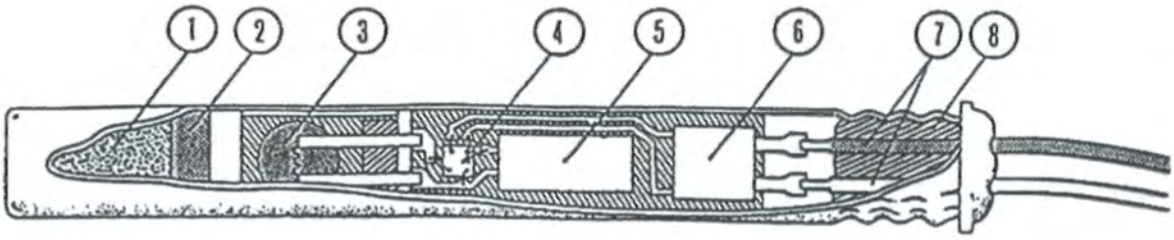
Thanks to this, we can deal with a very high number of delays because we have many possibilities of connection.
Furthermore, NONEL detonators can face different delays, with reference to the type.

→ NONEL GT/MS — 75 ÷ 500 ms

→ NONEL GT/T — 25 ÷ 6000 ms

→ NONEL UNIDET

ELECTRONIC DETONATORS



1. Base charge
2. Primary charge
3. Ignition head
4. Integrated circuit
5. Capacitance
6. Safety circuit
7. Leg wires
8. PVC seal

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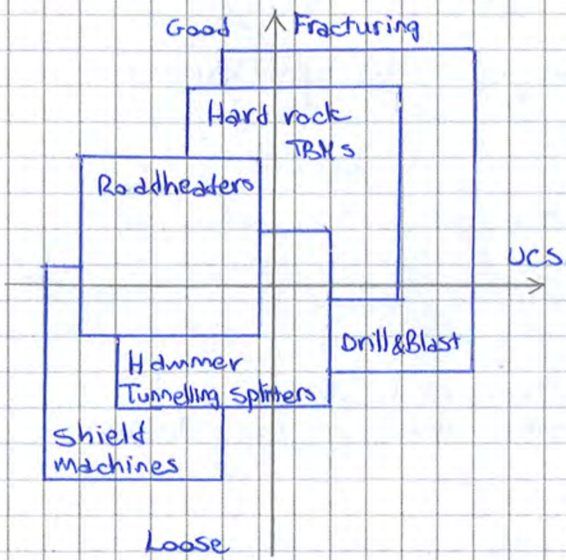
Drill and blast technique

1 Drill and blast

In order to get a first idea, we can represent the different tunnelling methods in the plot

→ x = uniaxial compressive strength - resistance

→ y = fracturing, going from good or homogeneous - microscopic fractures - to loose.



In case of loose ground with low UCS, we use shield machines.

If the class is better, we use roadheaders or hammer tunnelling splitters.

Hard rock TBMs are used when rock mass is strong enough to justify the employment of mechanized excavation.

Drill & Blast occupies a wide field and it is performed when rock mass is strong and homogeneous at the scale of the stope, with

$$UCS = 40 \div 280 (600)\ MPa$$

Indeed, in case of high strength or in presence of abrasive rock mass - big amount of quartz - , mechanized technique is not convenient.

ADVANTAGES → versatile equipment, since it is able to adapt to different conditions of ground

→ fast start-up - it doesnot need specific preparations for advancement

→ low capital cost

DISADVANTAGES → cyclic nature, which requires good work site organization since each phase should be completed before the next one starts.

→ vibrations and noise, implying restrictions in function of site - existing infrastructures.

TYPES OF ROCK SUITABLE FOR DRILL&BLAST

Rock Type	Rock (Minerals)	Range of Compressive Strength (10^6 N/m²)*	Comments
Igneous	**Granite** (quartz, feldspar, hornblende, and mica)	100 - 280	Covalent bonds. Fine grained granites with few fractures are the strongest. Granite is generally suitable for most engineering purposes.
	Basalt (olivine and pyroxene)	50 to >280	Covalent bonds, but small unstable minerals. Brecciated zones, open tubes, or fractures reduce strength†.
Metamorphic	**Marble** (calcite or dolomite)	100 - 125	Ionic bonds that can be broken by water and weak acids. Solutional openings and fractures weaken the rock†.
	Gneiss (quartz and feldspar)	160 - 190	Covalent bonds. Generally suitable for most engineering purposes.
	Quartzite (quartz)	150 - 600	Covalent bonds of large crystals. Very strong rock.
Sedimentary	**Shale** (feldspar, quartz, and mica)	<2 to 215	Fine grains and weak cementing with ionic bonded minerals can result in a very weak rock for engineering purposes; careful evaluation necessary.
	Limestone (calcite)	50 - 60	Ionic bonds. May have clay partings, solution openings, or fractures that weaken the rock†.
	Sandstone (quartz, feldspar, and calcite)	40 -110	Large grains and weak cementing with ionic bonded minerals can result in weak rocks. Strength varies with degree and type of cementing material, mineralogy, and nature and extent of factures†.

* Compressive strength is a measure of how much pressure a rock can withstand before breaking. Force is applied to both the top and bottom end of a cylinder of rock.

† Visual inspection of a rock can tell us something about the rock strength. Generally, rocks that are porous and permeable are weaker rocks.

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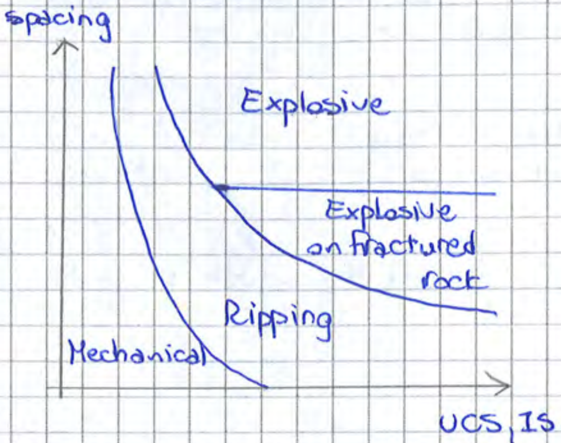
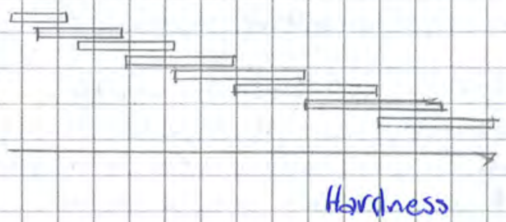
3 Choice of the most suitable technique

We have different diagrams showing us which is the best technique, in function of the kind of rock.

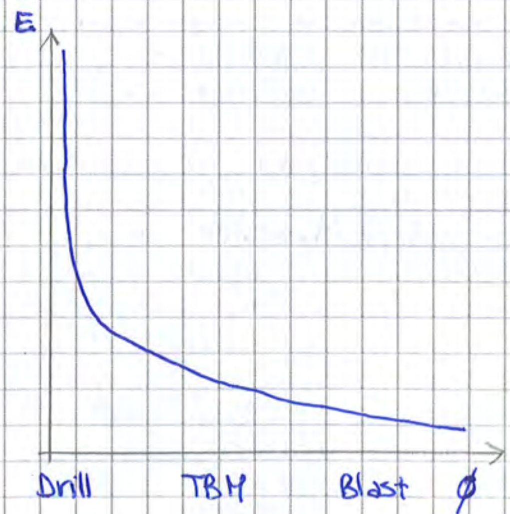


In the plot of spacing between discontinuities and UCS or point load test, we notice that blasting is required in strong and homogeneous rock mass.

Of course, to choose the most correct technique, we need laboratory tests.



An other parameter is hardness, which influence drillability and drilling technique to win the resistance.
Indeed, we need to apply a force able to break rock mass through tools which are not diamonds - they would be too expensive - but in steel and we have some limits.



We can also focus on the energy spent to reduce in fragments a given amount of rock.
The energy per ton - specific energy (MJm^{-3} or kWm^{-3}) - is maximum in case of small particle size and this is the case of drilling.
Indeed, the drill diameter is

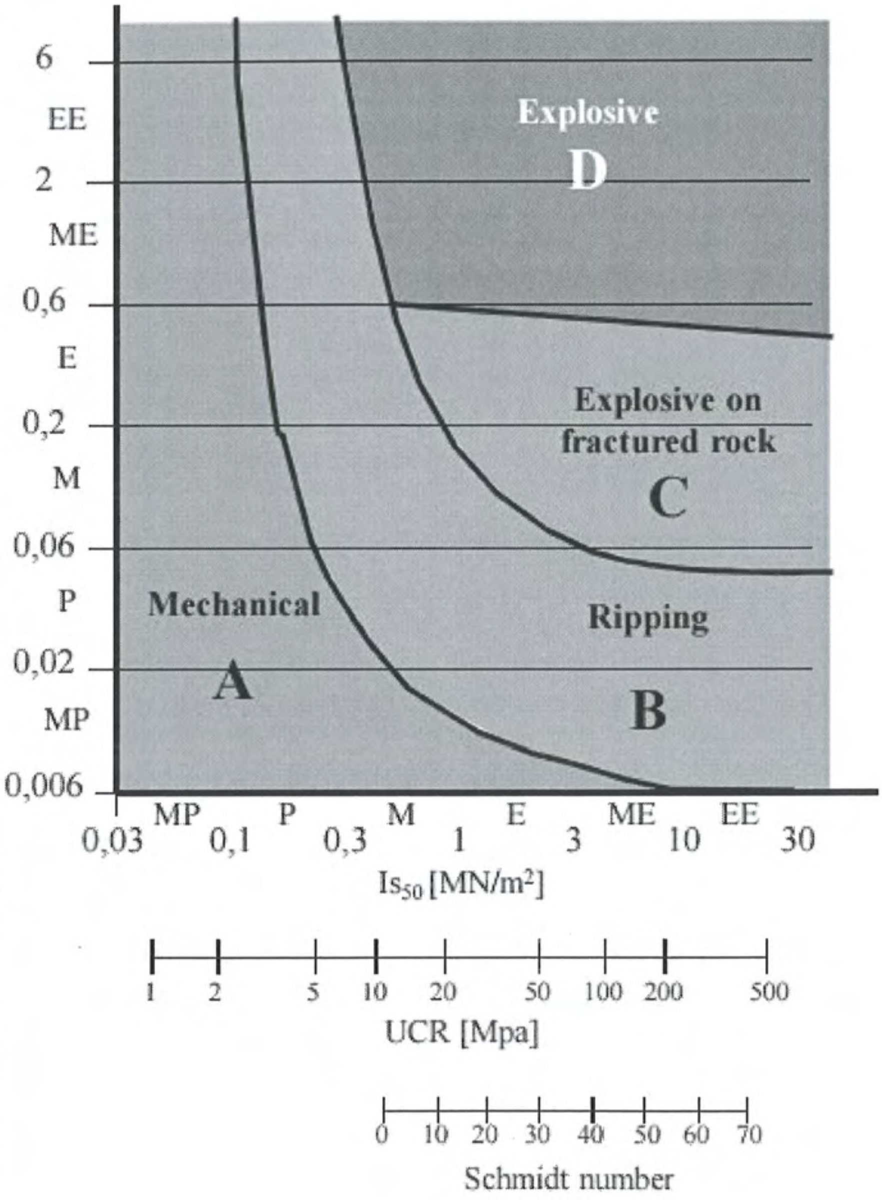
$$\phi = 32 \div 51 \text{ mm}$$

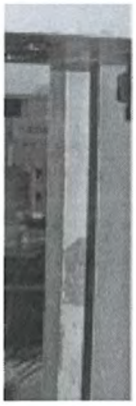
which is a small value, independently from the tunnel section.
To create the drill hole - empty volume -, we need to break the rock in very small elements and we spend lots of energy to create small openings.

On the other side, explosive requires less specific energy consumption because the explosive breaks rock mass in a big volume and in big elements.

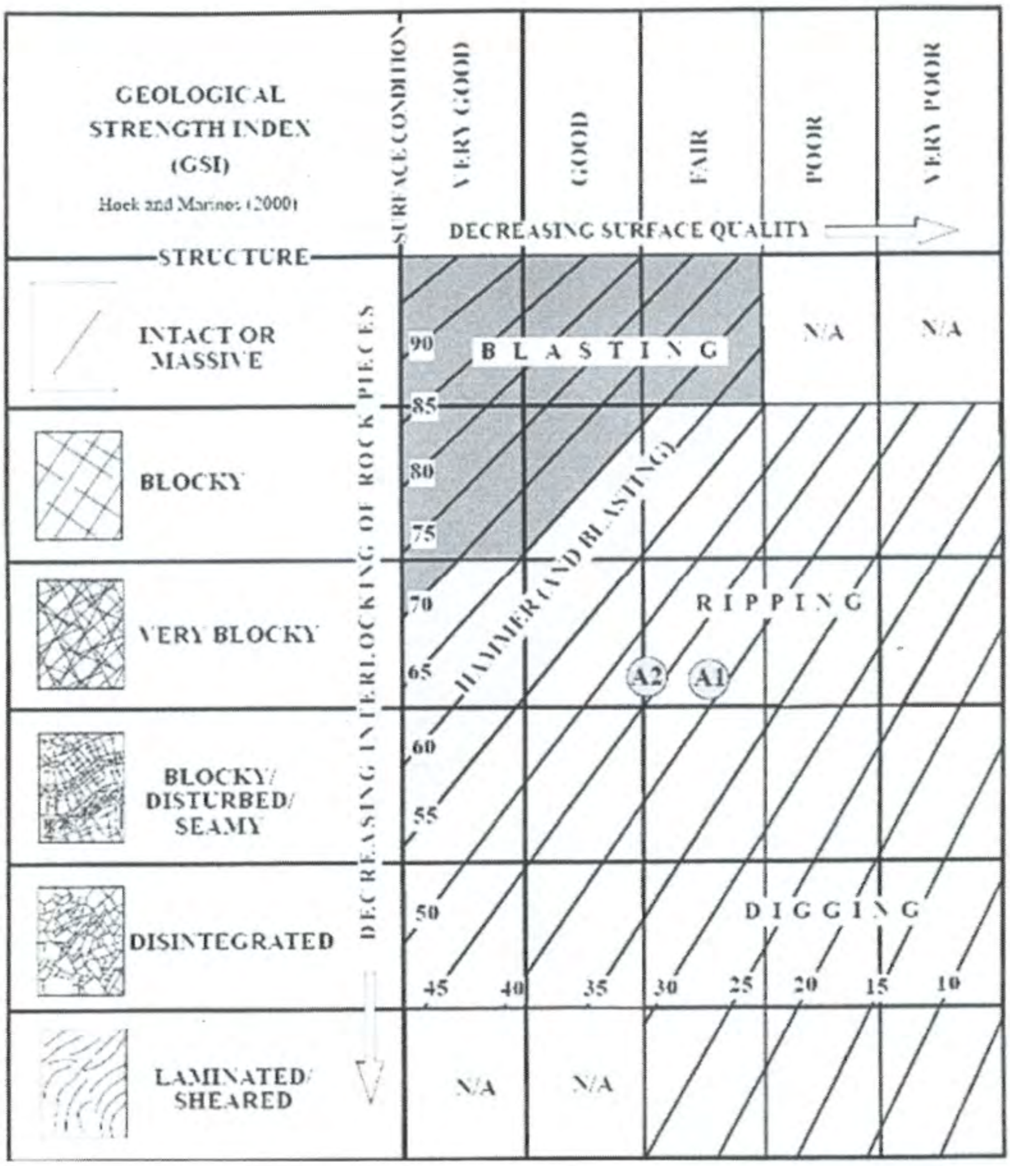
CHOICE OF THE MOST SUITABLE TECHNIQUE

Plot 1

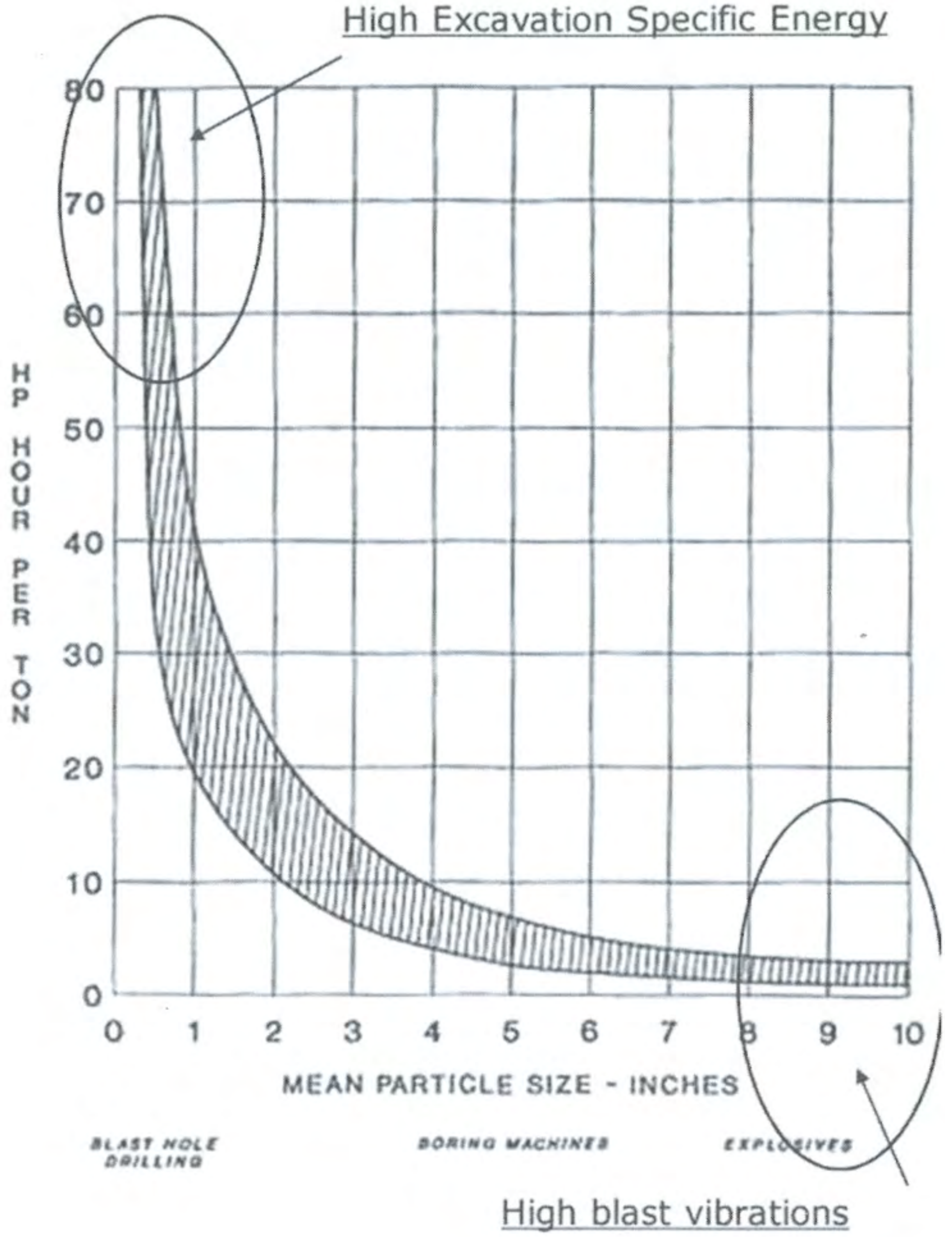




Plot 3


GEOLOGICAL STRENGTH INDEX (GSI)
Hoek and Marinos (2000)
SURFACE CONDITION
VERY GOOD
GOOD
FAIR
POOR
VERY POOR
DECREASING SURFACE QUALITY
STRUCTURE
INTACT OR MASSIVE
BLOCKY
VERY BLOCKY
BLOCKY/ DISTURBED/ SEAMY
DISINTEGRATED
LAMINATED/ SHEARED
DECREASING INTERLOCKING OF ROCK PIECES
N/A
N/A
N/A
N/A
BLASTING
HAMMER AND BLASTING
RIPPING
DIGGING
A2
A1
90
85
80
75
70
65
60
55
50
45
40
35
30
25
20
15
10

Plot 5



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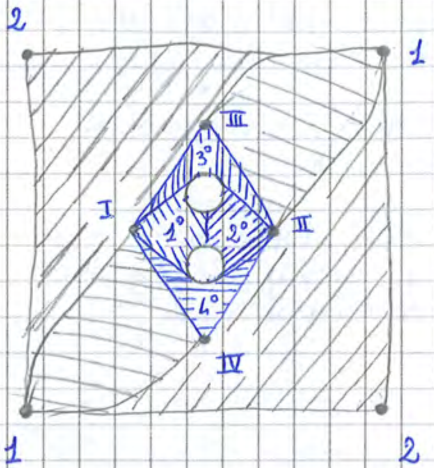
We will focus on the typical case, i.e. parallel hole cut, in which each hole is performed orthogonally to the wall.

What happens during the blasting?

We start from the cut holes.



In this case, we have two empty boreholes and a number of holes.
The ones identified with a roman number correspond to micro-delay detonators, whereas ordinary numbers correspond to long-delay detonators.

When the first hole detonate, a certain volume will be broken by it.

In this situation, we have to perform the dimensioning of the empty hole, judging from the fact that, from the initial volume, the broken rock increases its volume due to voids - bulk factor.
It means tha the volume of the empty hole must be bigger than the volume of the first part of the blast, since that volume has to go there in order to take profit from the free surfaces.

Then, when the other holes detonate, we start from a new situation and the new volume available is great enough to be occupied by the volume partaining to the next holes.

⇒ we have an evolution of the situation

The other holes may be placed at bigger distance with respect to the other ones because a big volume is available thanks to the explosion of the other holes.
This case is quite easy because we can use one delay for each couple of external holes. From these, we get again new volume, until the final volume.

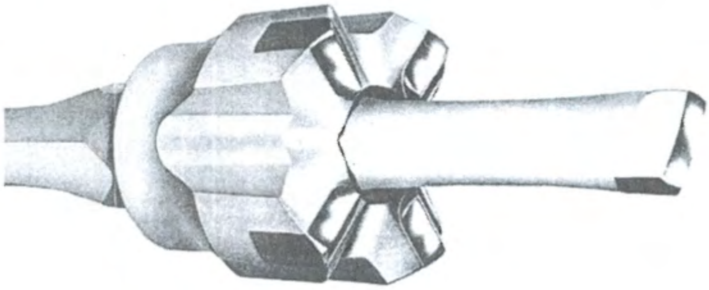
⇒ we have gradual decomposition of the rock mass thanks to the delay

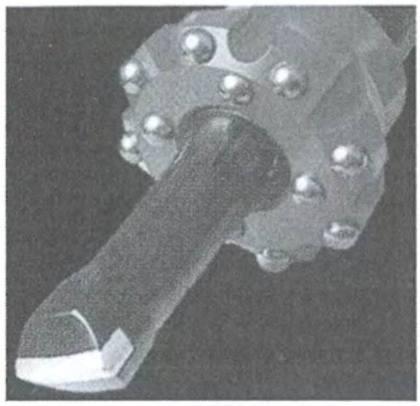
As regards delay, the best solution is to adopt

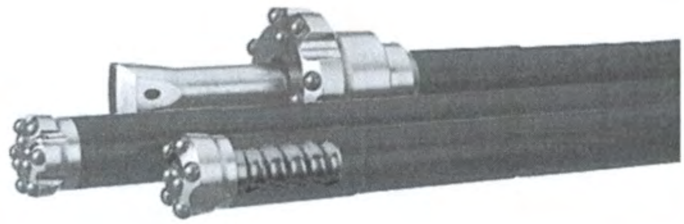
→ MICRODELAY IN THE CUT HOLES

→ ORDINARY DELAY IN THE PRODUCTION HOLES

Reamers



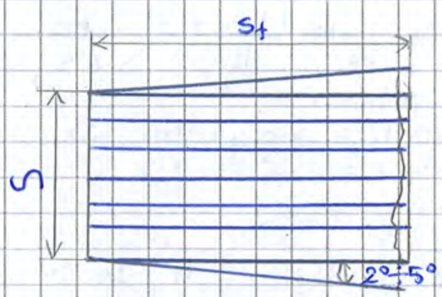




5 Parameters governing drilling pattern design

Drilling pattern design is governed by different parameters
- → tunnel dimensions
- → tunnel geometry
- → hole size
- → final quality requirements
- → geological and rock mechanical conditions
- → explosives availability and means of detonation
- → expected water leaks
- → vibration restrictions
- → drilling equipment

As regards geometry, the length of future blast is called pool s.



In the general situation, we have blast holes with contour holes with different inclination, drilled with look-out system.

Given a blast designed with pool s and section S, the volume blasted is

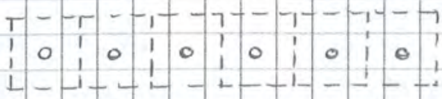
$$V = S \cdot s \quad \text{Volume blasted}$$

Holes are parallel to each other and orthogonal to the cross section, except the holes partaining to the contour, which respect a look-out of 2÷5°. In this way, blast by blast, we respect the direction and the section of the tunnel.

↳ if they were parallel, section would progressively reduce

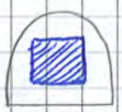
In the evaluation of the pool, we should distinguish
- → THEORETICAL POOL s_t, corresponding to the drilled length
- → REAL POOL s, since we are not sure to respect the theoretical length with the blast

We can notice that, in case of open pit, each blast hole blasts the same part of rock - same volume - and uses the same amount of charge.
It means that powder factor and specific drilling are constant.

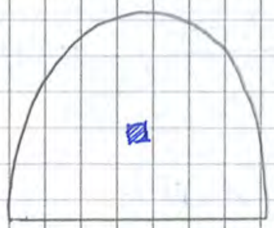


The difference is due to the fact that we have different groups of boreholes in tunnel excavation, with different distribution and geometry.
On one side, we have cut holes, which are accomplished in bad conditions and require a big amount of charge.
On the other side, we have production holes and contour holes.

If we consider two different cross sections, the portion that pertains to the cut is the same part, as it has only to allow the first blast.
Actually, in the biggest section, the influence of the cut is smaller and the blast holes working in better conditions become important.
By consequence, specific drilling and powder factor decrease.



$S = 5\ m^2$



$S = 100\ m^2$

6 Hole size

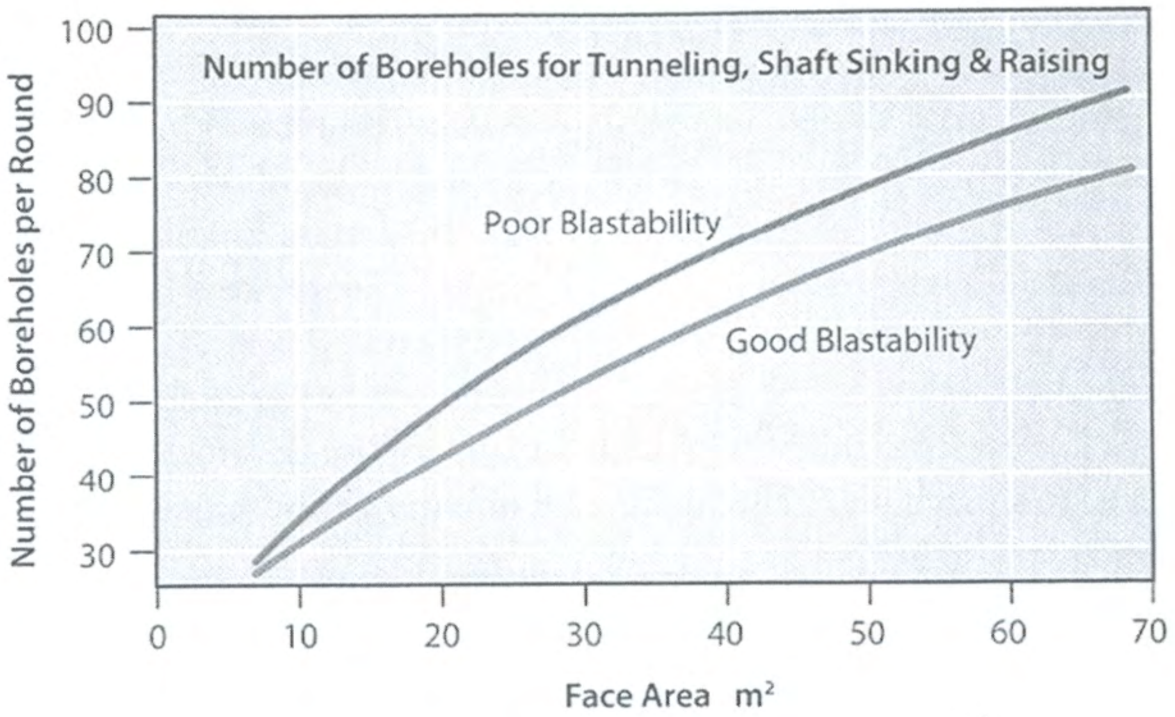
Referring to different cross sections, they collected different cases and distinguished small sections ($< 10\ m^2$), medium sections ($10 \div 40\ m^2$) and big sections ($> 40\ m^2$).

We can notice that, independently from the cross section, the diameter of boreholes is always small, in order to get very well fragmented material because the pattern is smaller.
The most common values are

$$\phi = 32 \div 51\ mm$$

independently from the face area.

Only cut holes are bigger than 51 mm because they have to create an additional free surface.



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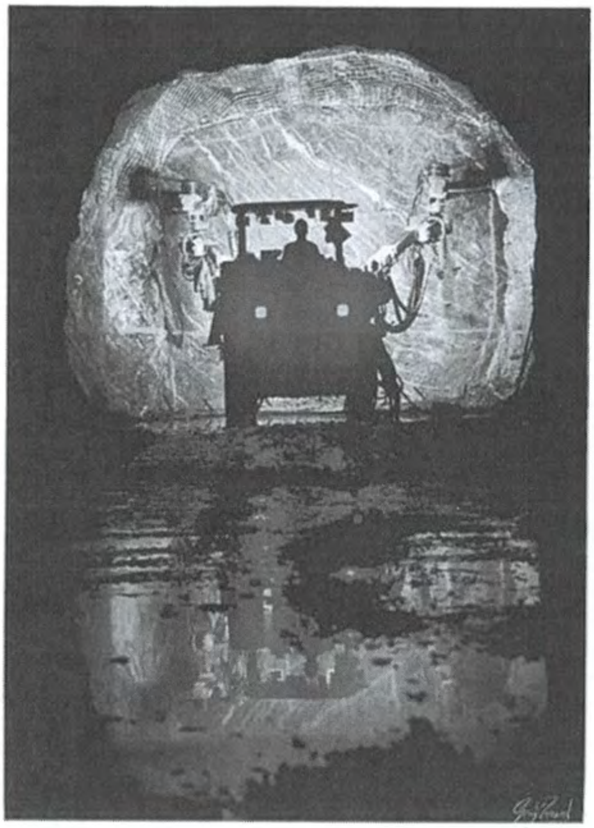
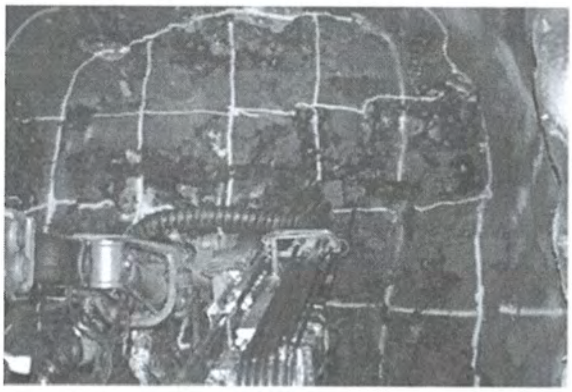
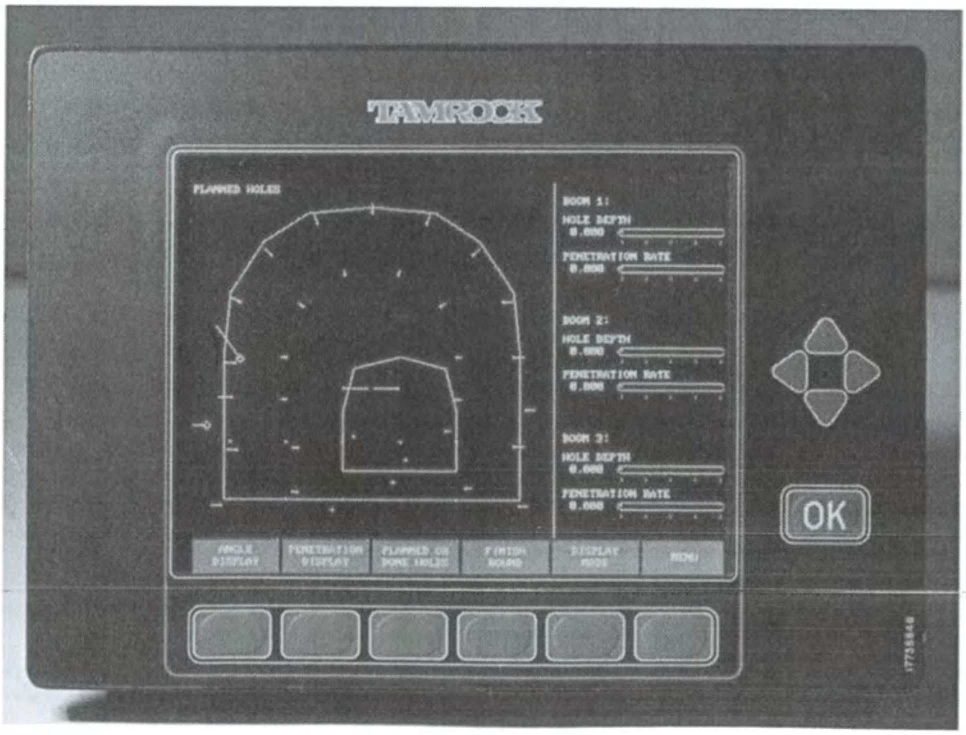
The marking of the tunnel section





Computerized drilling interface



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Tunnel driving round design

1 Parameters governing tunnel driving round design

This kind of design consists of evaluating the

→ AMOUNT OF CHARGE

→ DRILLING HOLES

necessary to perform a blast round.

The design is based on some important parameters

→ ideal volume to be blasted, referred to the theoretical pool, i.e. drilled length.

$$V_t = S \cdot s_t$$

→ actual volume blasted, referred to the effective pool.

$$V_e = S \cdot s_e$$

→ overall drilled length, linked to the ideal volume because it is related to the theoretical pool.

$$L = n s_t = V_t \cdot \frac{n s_t}{V_t} = V_t \cdot SD$$

$$L = V_t \cdot SD$$

→ total charge

$$Q = PF \cdot V_e$$

→ total number of holes, given by the overall drilled length over the theoretical pool.

$$n = \frac{L}{s_t} = \frac{V_t \cdot SD}{s_t} = \frac{S \cdot \cancel{s_t} \cdot SD}{\cancel{s_t}} = S \cdot SD$$

$$n = S \cdot SD$$

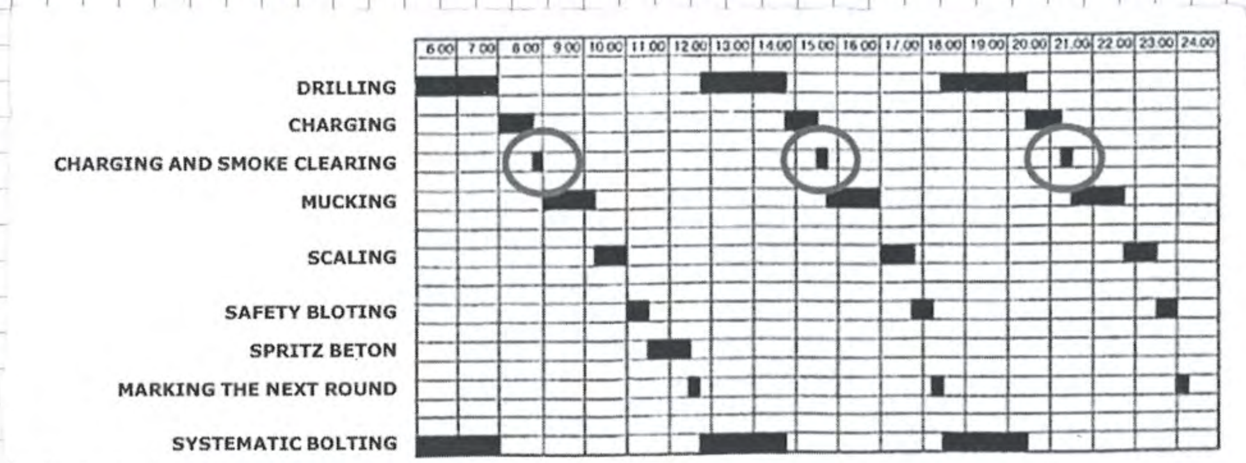
explosive.
Anyway, we have a progressive decomposition of rock, following the progressive evolution of blast - the numbers.

3 Cyclogram of tunnel driving

The cyclogram of tunnel driving contains the different operations we have to perform in a day and depends on the fact that we are working with cycles, in which each operation starts when the previous has been completed.
The operations are

- → drilling : it lasts 2 hours.
- → charging
- → blasting, which lasts few second and it is not represented in this scale, and smoke cleaning
- → mucking
- → scaling
- → safety bolting
- → spritz beton
- → marking of the next round
- → systematic bolting

The only operations superposed are drilling and systematic bolting because they are performed with the same machine - it mounts more than one arm.



A second category is inclined cuts, that are holes drilled at certain angles.

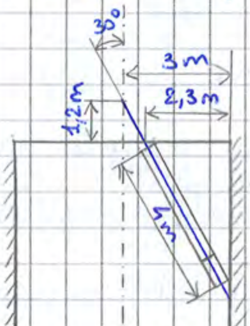
We can see the collection of results coming from different data of literature and evaluate the frequency of use of cut type with reference to the section.

→ in small sections, the frequency of parallel cuts is bigger than the one of inclined cuts.

→ in big sections - especially when the section is bigger than 60 m^2 -, prevail the cases organized with inclined holes.

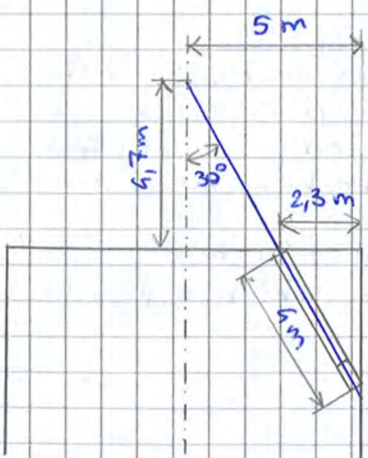
Indeed, inclined cuts do not need the realization of an empty hole, but they have also some limitations:

given two tunnels, which are 6 m and 10 m wide, respectively, the drilling machine is the same and the arms occupy a given space, due to the geometry.



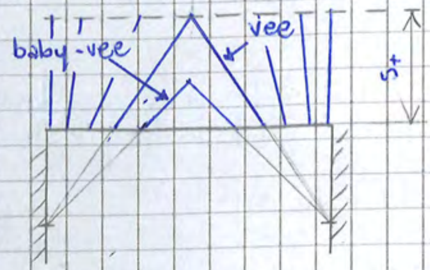
If we want to perform an inclined cut, we need to create something like a wedge that will be extruded from the face, in order to help the work of production holes.
To create the wedge and help its expulsion from the face, the holes should be inclined of $20 \div 40°$.
By consequence, we place the machine in a certain position and, since the arm occupies some space into the section, the maximum pool reached is 1,2 m, with the section 6 m wide and with this geometry.

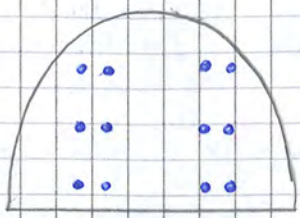
⇒ if we want to perform an inclined cut in a small section, we get a PENALTY IN TERMS OF POOL we can reach.



In case of parallel cut, on the other side, the empty hole can go on.

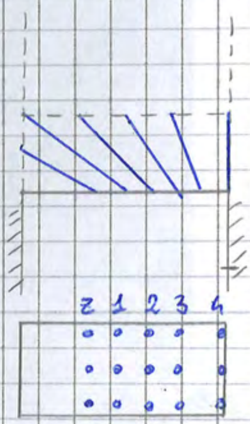
In the section which is 10 m wide, with the same geometry, we can reach a pool with 5 m of length.
Thus, only due to geometry, we can reach a longer pool.





Another type of inclined cut is fan cut:

this technique is employed when the cross section is not big and it is impossible to perform correctly the V cut.
Due to the limited space, we try to adopt a geometry called FAN ("ventaglio").



The blast starts from the left, in order to isolate the triangle defined by the free surface and the first hole.
Indeed, we organize the timing in order to extrude the wedge and, as consequence, each column of blasting holes will detonate simultaneously or with small delays.
Of course, the fragmentation maybe will be not so good, but the scheme works because we can blast big blocks.
Since many holes detonate at the same time, we can have problems of VIBRATIONS.

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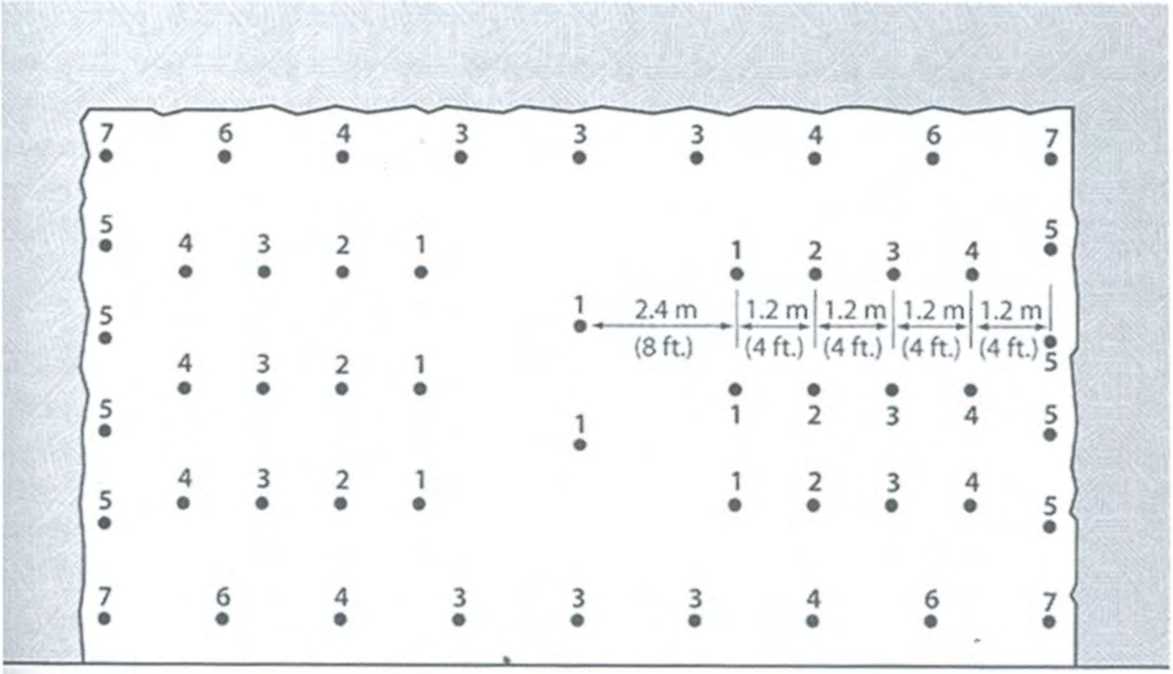
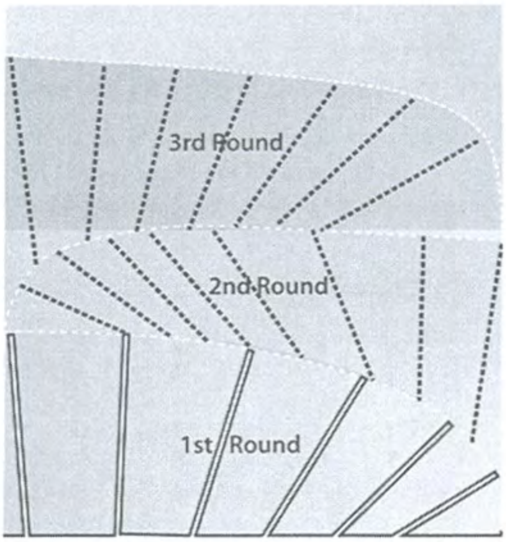
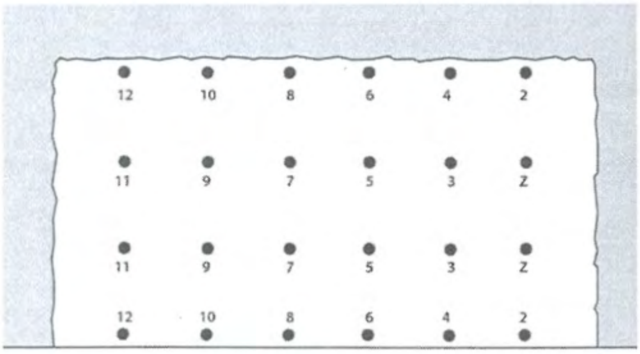
V-cut pattern

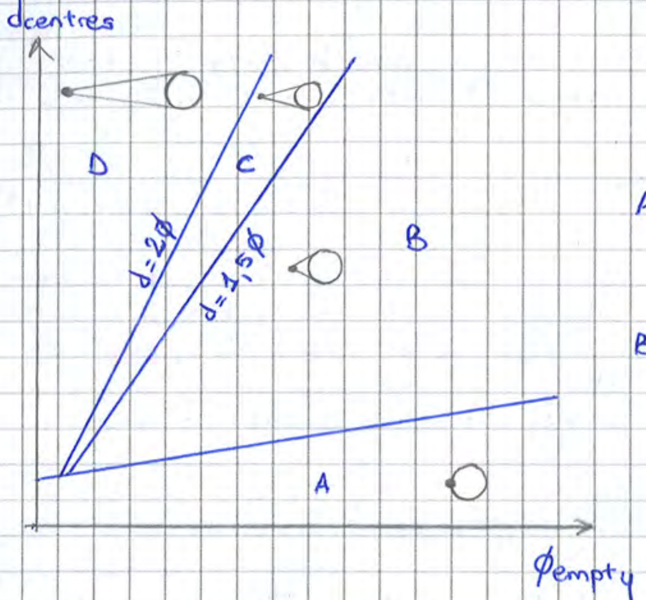


Fan-cut pattern





Then, we design the arrangement of charged holes, starting from the spacing among the centre of the empty hole and the centre of each charged hole - the first ones detonating.



We compare the diameter of the empty hole and the spacing between its centre and the centre of the charged holes.

A) THE HOLES MEET: the spacing is not enough and we get no result, i.e. the blast is not helpful

B) Clean Blasted: the spacing is bigger and the hole cooperates giving free surface and creating a volume which will be broken.

C) Breakage: in this case, the distance maximizes the volume to be broken

D) Plastic deformation: the distance is too much big and the boreholes which detonate first do not see the free surface because they are too far from it.

We should work in breakage or clean blasted conditions, otherwise we could not get any result.

In this way, we can design the 1st square:

the position of the blastholes in the I square is expressed as

$$a = 1,5\,\phi \qquad \text{Position of the blastholes}$$

a = distance centre of empty hole - centre of blasthole

ϕ = diameter of the empty hole or fictitious diameter of the system of empty holes.

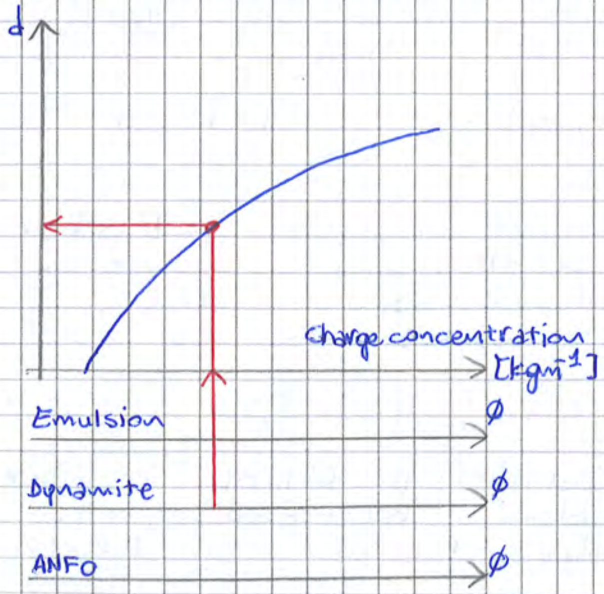
Then, the location of the first square may be in different parts of the tunnel section, related with tunnel shape and size. Generally, the cut is placed vertically in the middle of the section and horizontally on or slightly under the center line of the tunnel.

From the side of the free opening, we can obtain the charge concentration for the second round and the correspondent amount of charge.

In the second round, we get a new opening of side W'. From this, we derive the size of the third square and the amount of charge and so on.

We can repeat the procedure up to the IV square, using the diagram introduced above to derive the amount of charge.
Even we can stop before.

At this point, we design the production holes and we use a scheme which, depending on the kind of explosive used, give the amount of charge for the stoping holes.
The explosives considered are emulsions, dynamite and ANFO - generally avoided in Italy in underground excavations, due to water.

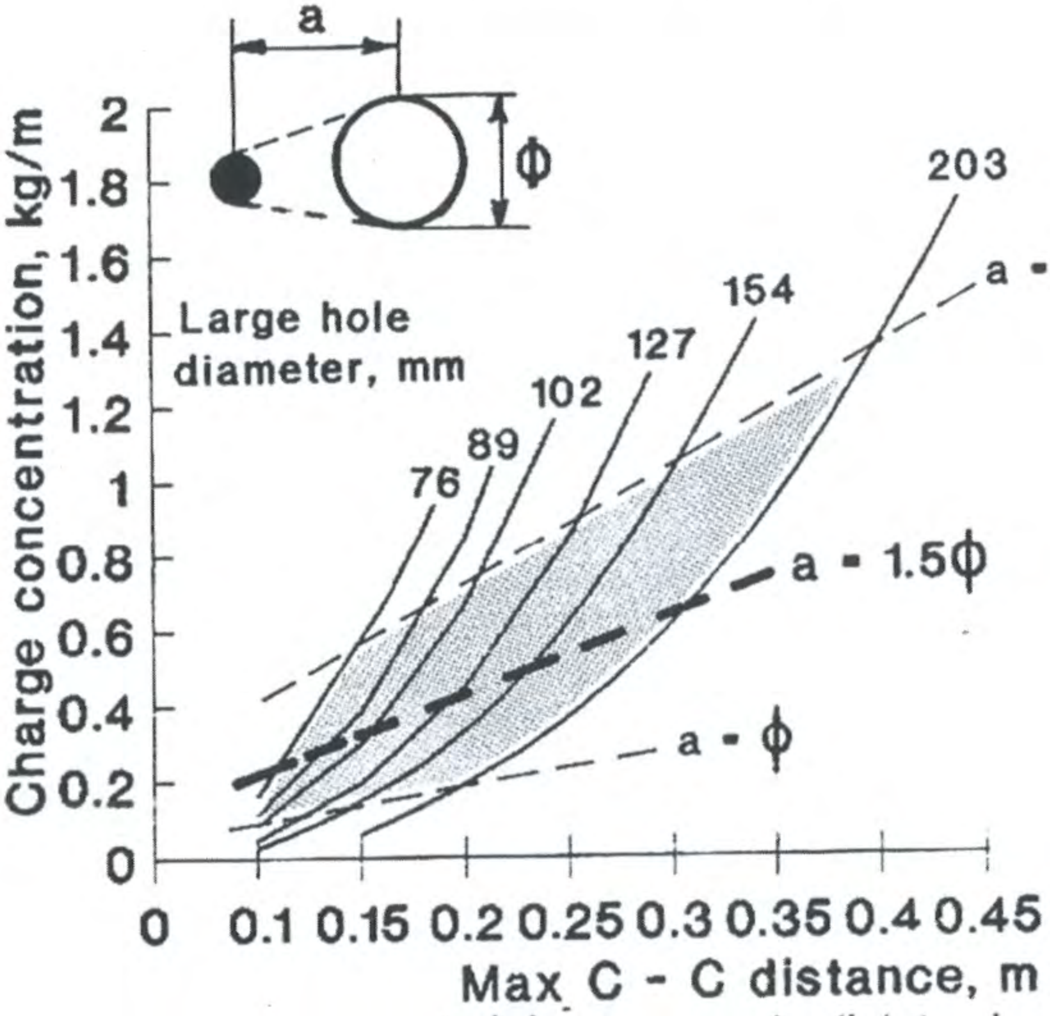


The diameter is the same for each kind of explosive and varies between 30 mm and 51 mm.
We enter with the diametre and, in the plot, we can read

→ on x axis, the CHARGE CONCENTRATION

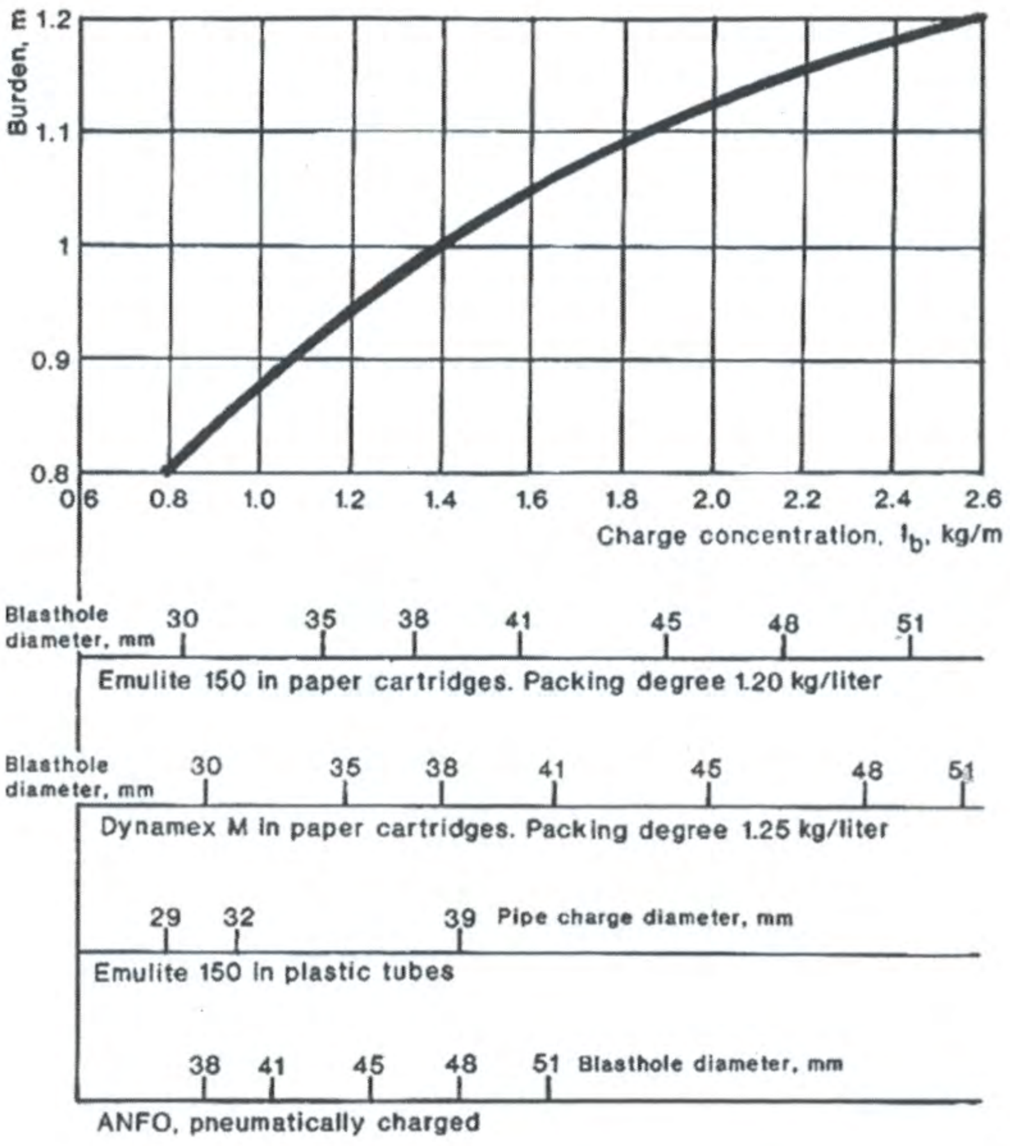
→ on y axis, the BURDEN, i.e. the distance between the holes in the pattern, in order to place the production holes

Charge concentration in the first square



The minimum required charge concentration (kg/m) and maximum C–C distance (m) for different large hole diameters.

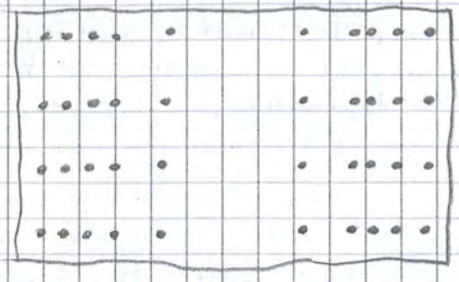
Production holes

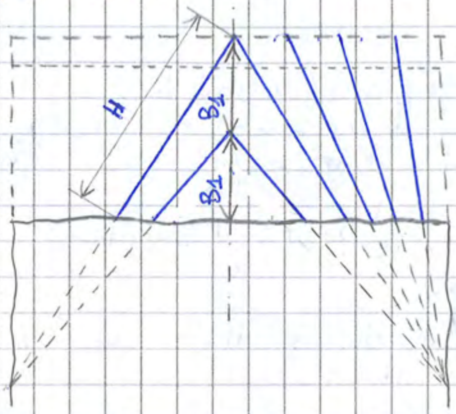


6 Design of V-cuts

In the typical pattern, the machine is placed along the sides of the excavation and rotates the arms in order to cover the section to be blasted, obtaining a certain pool.

The design is based on some parameters.





B_1 = burden for the cut holes

B_2 = distance between the holes at the bottom, in correspondence of the theoretical pool

c = height of the cut, related to how many drilling raws we need to perform in order to get a prism.

The parameters are given by a diagram.

7 Preliminary design formulations

Before starting the project, we can estimate the powder factor expected for drill & blast in function of the cross section S.

$$PF \sim \left(\frac{10}{S} + 0,6\right) \cdot ABC$$ Estimate of the powder factor

This is an empirical formulation and it is valid to give a general idea of the magnitude, before starting the design.

We can notice that the biggest is the cross section, the smallest will be the required powder factor, due to the less influence of cut holes - they have a strong influence on the powder factor.

Coefficients A, B and C are expressed in 3 tables.

- → coefficient A depends on the quality and hardness of the rock mass - in class IV, drill & blast is not useful.
- → coefficient B depends on explosive type.
- → coefficient C depends on the type of round - parallel holes require a stronger amount of explosive.

We can also evaluate the specific drilling, through an empirical relation.

$$SD \sim 2,3\left(\frac{10}{S} + 6\right) \cdot AB'$$ Estimate of the specific drilling

Coefficient A is the same as before - related the rock class - and coefficient B depends on the explosive type.

It is possible to have an approximated idea of the CHARGE DISTRIBUTION, i.e. the average powder factor partaining to cut holes, production holes and contour holes

CUT HOLES $PF_{cut} = ($[illegible]$) PF_{min}$

PRODUCTION HOLES $PF_{pr} = (4 \div 5) PF_{min}$

CONTOUR HOLES $PF_{cont} = (2 \div 3) PF_{min}$

Empirical formulations

CLASS	PROTODYAKONOV CLASS	EXAMPLES	A COEFFICIENT
1	0	Quartzites, sound porphyries	1.3
2	I	Sound granites and gneiss	1
3	II	Strong limestones	0.9
4	III	Strong schists and slates	0.8
5	IV	Soft limestones, marl, gypsum	0.5

CLASS	EXPLOSIVE TYPE	B COEFFICIENT
1	Straight gelatin dynamite ($\gamma>1.5$)	0.95
2	Semigelatin dynamite ($\gamma>1.4$)	1
3	Other heavy explosives ($\gamma>1.2$)	1.1
4	N.A. based, powder explosives ($\gamma<1.2$)	1.2

CLASS	TYPE OF ROUND	C COEFFICIENT
1	Inclined holes rounds (V cut)	1
2	Fan	0.9
3	Parallel holes	1.45

EXPLOSIVE CLASS	B' COEFFICIENT
1	0.6
2	0.65
3	0.8
4	1.2

→ geophone (or transducer):

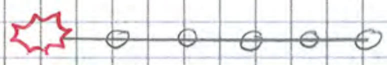
used with a seismograph, it is an instrument which gets the ground displacement in one or three directions, by converting ground movement to electrical signals for recording.

→ seismogram:

it is a permanent record of seismic events.

3 Determination of site parameters

In order to check vibrations, we have to measure them not only in one point but in different points because we need different measures to find the representative law of the ground.



An easy way to measure vibrations is to blast in a point and place different instruments along an alignment, at different distances from the blast.

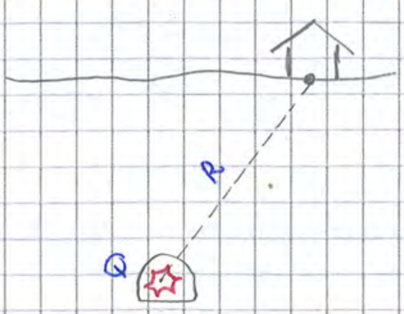
The instruments receive a signal - a certain excitation - which can be rendered in a velocity.

Due to the different distances of instruments from the source, instruments will receive different signals

→ with bigger distance, instruments will measure smaller velocities

→ with closer distance, instruments will measure bigger velocities

Passing to tunnelling applications, instruments are installed with certain geometry to know the vibrations around the tunnel during excavation.
Instruments can also be used to derive rock mass characteristics.



We imagine now a blast at certain depth and there is an object on surface to be protected - it is sensitive.

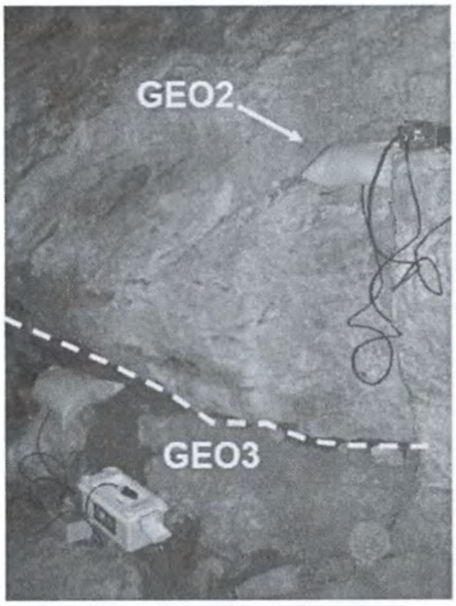
A charge Q is blasted and it corresponds to the amount of explosive simultaneously blasted, i.e. amount of charge in a single delay.

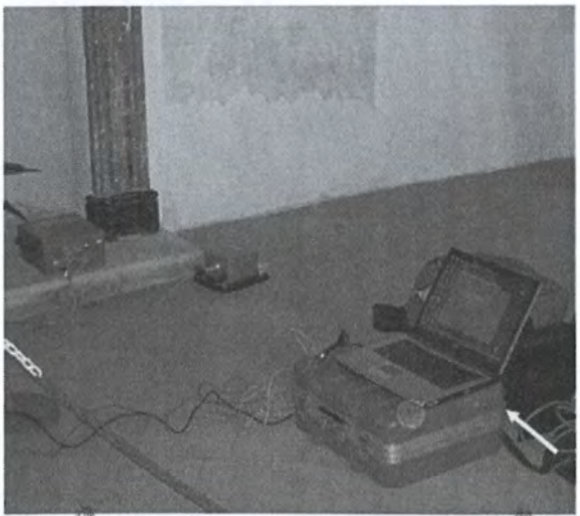
EXAMPLE OF BLAST TEST

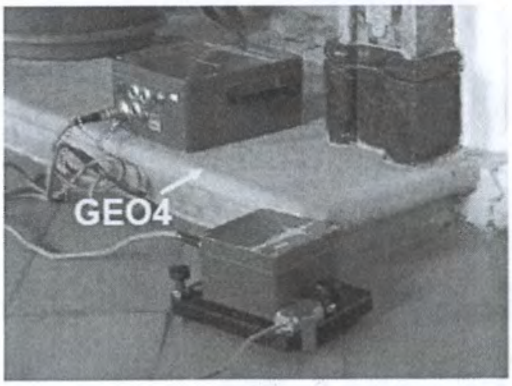
BLAST TESTS

We need to perform some blast tests before starting with the tunnel excavation.

Blast tests consist of drilling some holes to place a known amount of charge into the rock mass, which will be detonated.
A great number of instruments – geophones – is placed closed to the strategic points, e.g. sensitive buildings, where we want the velocity not to be bigger than the prescribed value given by the codes.







The blasting produces with a certain history of velocity, characterized by several impulses, called *vibrogram*. We can notice that also mechanized excavation produces an appreciable vibration, but it is continuous and quite regular in time. Of course, this kind of vibration is able to create problems, anyway.

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LIMITS ON VIBRATIONS

The limits on vibration entity are given by two codes.

- DIN 4150-3: it is the German code, which divides the structures into three classes.
- SN 640312a: it is the Swiss code.

Norma	Classe	Designazione, esempi
DIN 4150	1	Poco sensibili (costruzioni industriali)
	2	Sensibilità normale (edifici abitazioni ed assimilabili)
	3	Sensibili (edifici protetti)
SN 640312a	1	Molto poco sensibili (es: ponti, tunnel, basamenti)
	2	Poco sensibili (es: costruzioni industriali in calcestruzzo armato, silos, capannoni)
	3	Sensibilità normale (es: abitazioni, scuole, edifici pubblici)
	4	Particolarmente sensibili (costruzioni delicate o comunque protette)

According to the German code, there are some limit speeds we can reach and not overpass, with reference to the class of the building.

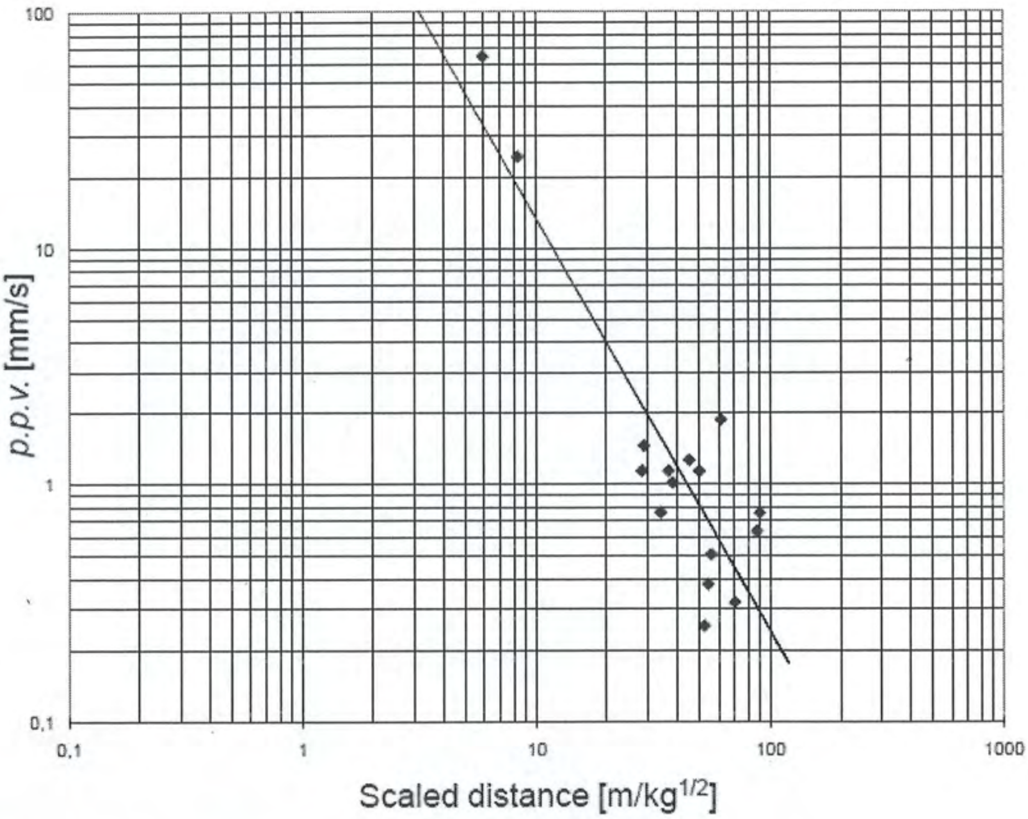
The most restrictive class is class 3, where we must not overpass a speed of 3 mm/s at low frequencies, whereas the limit values are bigger at higher frequencies. Hopefully, in blasting, we usually work in the field of high frequencies.

DEFINITION OF THE SITE LAW

The experimental data coming from the campaign of tests are represented inside a bi-logarithmic plot, with the following axes.

- X axis is the scaled distance $\frac{R}{\sqrt{Q}}$.
- Y axis is the peak particle velocity.

Each single result correspond to a point in this plot.



The data are treated with a regression and we get the parameters k and α.

- k is read from the vertical axis in correspondence of $\frac{R}{\sqrt{Q}} = 1$.
- α is the tangent of the regression line, i.e. the angle on the horizontal axis.

$$k = 744$$

$$\alpha = -1{,}744$$

These parameters define the site law referred to these measures.

At this point, knowing the distance of the sensitive elements from the blasting point and the limits, we can find the maximum charge required in order not to go beyond the prescribed limits:

EXERCISE 1: PRELIMINARY GEOLOGICAL SECTION

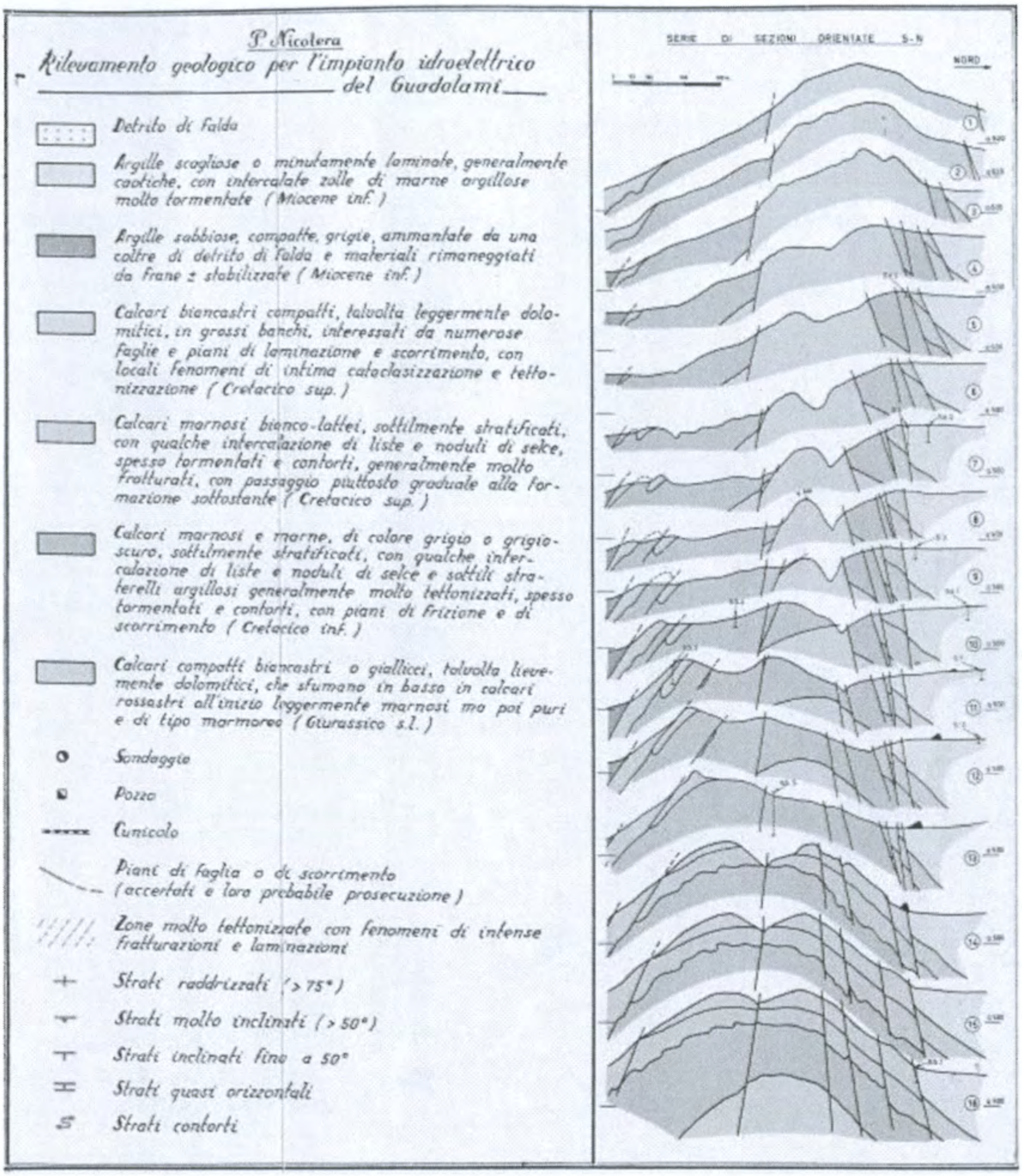
Versione italiana

Per il corretto progetto di una galleria, è necessario conoscere esattamente le proprietà e la posizione delle varie formazioni geologiche presenti lungo il tracciato.

Un piccolo tunnel per il trasporto di acqua deve essere costruito da un lago verso la centrale elettrica e due percorsi alternativi sono dati.
È disponibile il rilevamento geologico svolto da geologo, che si caratterizza per il tipico graficismo geologico.

- Le linee rosse rappresentano le faglie.
- Simboli collegati all'inclinazione e immersione degli strati, che possono essere molto inclinati, leggermente inclinati (meno di 50 °), sub-orizzontali, strati piegati o zone molto tettonizzate, con fratture e laminazione.
- I punti rappresentano le aree in cui la massa di roccia è coperto da detriti.

L'indagine fornisce anche una serie di sezioni Nord-Sud, che mostrano una zona centrale di calcare bianco compatto, in origine coperta da marne, quindi esposta dall'erosione.


P. Nicotera
Rilevamento geologico per l'impianto idroelettrico del Guadalami
Detrito di falda
Argille scagliose o minutamente laminate, generalmente caotiche, con intercalate zolle di marne argillose molto tormentate (Miocene inf.)
Argille sabbiose, compatte, grigie, ammantate da una coltre di detrito di falda e materiali rimaneggiati da frane ± stabilizzate (Miocene inf.)
Calcari biancastri compatti, talvolta leggermente dolomitici, in grossi banchi, interessati da numerose faglie e piani di laminazione e scorrimento, con locali fenomeni di intima cataclasizzazione e tettonizzazione (Cretacico sup.)
Calcari marnosi bianco-lattei, sottilmente stratificati, con qualche intercalazione di liste e noduli di selce, spesso tormentati e contorti, generalmente molto fratturati, con passaggio piuttosto graduale alla formazione sottostante (Cretacico sup.)
Calcari marnosi e marne, di colore grigio o grigio-scuro, sottilmente stratificati, con qualche intercalazione di liste e noduli di selce e sottili straterelli argillosi generalmente molto tettonizzati, spesso tormentati e contorti, con piani di frizione e di scorrimento (Cretacico inf.)
Calcari compatti biancastri o giallicci, talvolta lievemente dolomitici, che sfumano in basso in calcari rossastri all'inizio leggermente marnosi ma poi puri e di tipo marmoreo (Giurassico s.l.)
Sondaggio
Pozzo
Cunicolo
Piani di faglia o di scorrimento (accertati e loro probabile prosecuzione)
Zone molto tettonizzate con fenomeni di intense fratturazioni e laminazioni
Strati raddrizzati (> 75°)
Strati molto inclinati (> 50°)
Strati inclinati fino a 50°
Strati quasi orizzontali
Strati contorti
SERIE DI SEZIONI ORIENTATE S-N
NORD

Ad es., si può specificare se l'attacco dello scavo sia da un solo lato oppure da due lati, in funzione delle strade di accesso, etc.

- Stima del volume di terreno estratto, tenendo conto del coefficiente di rigonfiamento (circa 1,3 ÷ 1,5), e indicazioni sulle modalità di rimozione e stoccaggio del materiale stoccato.
- Stima preliminare dei costi (non richiesto).

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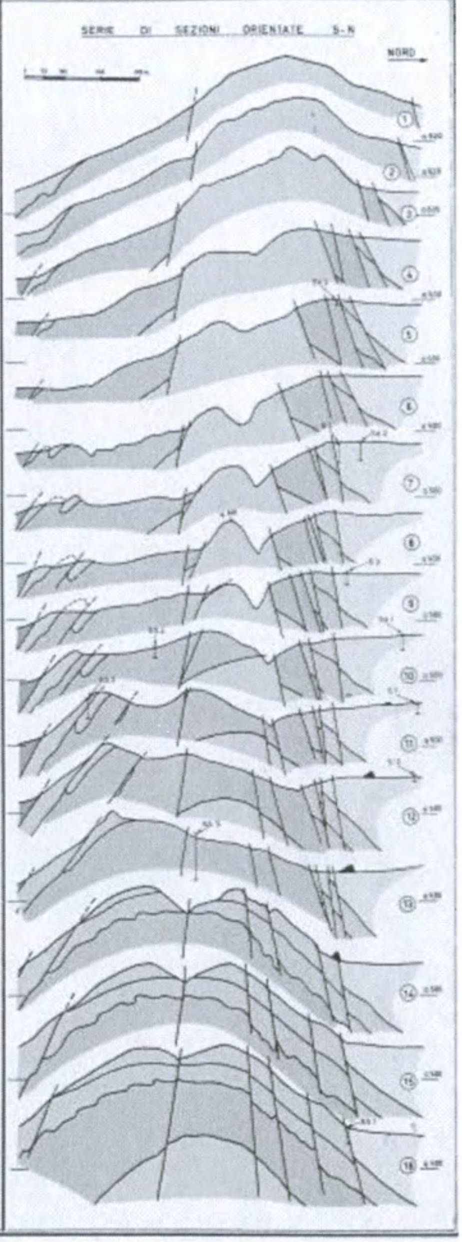
P. Nicotera

Rilevamento geologico per l'impianto idroelettrico del Guadalami

- Detrito di falda
- Argille scagliose o minutamente laminate, generalmente caotiche, con intercalate zolle di marne argillose molto tormentate (Miocene inf.)
- Argille sabbiose, compatte, grigie, ammantate da una coltre di detrito di falda e materiali rimaneggiati da frane ± stabilizzate (Miocene inf.)
- Calcari biancastri compatti, talvolta leggermente dolomitici, in grossi banchi, interessati da numerose faglie e piani di laminazione e scorrimento, con locali fenomeni di intima cataclasizzazione e tettonizzazione (Cretacico sup.)
- Calcari marnosi bianco-lattei, sottilmente stratificati, con qualche intercalazione di liste e noduli di selce, spesso tormentati e contorti, generalmente molto fratturati, con passaggio piuttosto graduale alla formazione sottostante (Cretacico sup.)
- Calcari marnosi e marne, di colore grigio o grigio-scuro, sottilmente stratificati, con qualche intercalazione di liste e noduli di selce e sottili straterelli argillosi generalmente molto tettonizzati, spesso tormentati e contorti, con piani di frizione e di scorrimento (Cretacico inf.)
- Calcari compatti biancastri o giallicci, talvolta lievemente dolomitici, che sfumano in basso in calcari rossastri all'inizio leggermente marnosi ma poi puri e di tipo marmoreo (Giurassico s.l.)
- Sondaggio
- Pozzo
- Cunicolo
- Piani di faglia o di scorrimento (accertati e loro probabile prosecuzione)
- Zone molto tettonizzate con fenomeni di intense fratturazioni e laminazioni
- Strati raddrizzati (> 75°)
- Strati molto inclinati (> 50°)
- Strati inclinati fino a 50°
- Strati quasi orizzontali
- Strati contorti



E.g., we can specify whether the attack of the excavation is from one side only or from two sides, depending on the access roads, etc.

- Estimation of soil extracted volume, taking into account the swelling coefficient (about 1.3 ÷ 1.5), and indications on how to remove and store the excavated material.
- Preliminary estimate of costs (not required).

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- Available geotechnical data, from the geological report which provides geotechnical data (summarized in a table).
- Calculations.
- Conclusions, specifying which pressure value is to be used.

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Di conseguenza, si possono adottare diverse cariche in diverse sezioni e, in questo caso, si impiegano tre cariche diverse in tre punti diversi.

La definizione della legge di sito parte da una tabella che illustra i risultati delle prove di campagna, includendo la quantità di carica, la distanza, la velocità di picco delle particelle misurata e la frequenza misurata.

Charge (kg)	Distance R (m)	Measured PPV (mm/s)	Measured frequency (Hz)
5	20	21.1	55
5	50	6.7	45
8	50	11.1	38
10	100	4.6	28
10	80	6.3	38
10	100	5.8	42
10	150	2.9	31
10	200	2.3	21
15	35	20.4	38
15	100	5.8	35
35	500	1.4	15
35	550	1.3	20

La frequenza in questo caso deve essere considerata, dal momento che bisogna decidere la velocità limite, che dipende dalla frequenza delle vibrazioni.
Il valore limite consigliato è

$$PPV = 8\, mm/s$$

ed è valida se l'esplosione non avviene vicino a un edificio.

Conoscendo il valore limite e la legge di sito, si può stimare la distanza scalata $DS_{lim} = \frac{R}{\sqrt{Q}}$ (corrispondente a una certa velocità limite) che si dovrebbe seguire per rispettare le prescrizioni.

In realtà, ci si trova di fronte a diversi elementi sensibili a diverse distanze. Dal momento che non si è in grado di valutare molte distanze, si esegue la valutazione soltanto di 3 ÷ 4 diverse distanze – distanze minime –: per ogni distanza, conoscendo il valore limite DS_{lim}, si ottiene la massima carica che si può impiegare.

Naturalmente, più grande è la distanza, più grande sarà la carica che si può utilizzare. D'altra parte, in caso di rischio di danno, sarà possibile ridurre lo sfondo, al fine di rispettare i limiti, ricordando tuttavia che l'obiettivo è quello di progettare sfondi più lunghi possibile.

Charge (kg)	Distance R (m)	Measured PPV (mm/s)	Measured frequency (Hz)
5	20	21.1	55
5	50	6.7	45
8	50	11.1	38
10	100	4.6	28
10	80	6.3	38
10	100	5.8	42
10	150	2.9	31
10	200	2.3	21
15	35	20.4	38
15	100	5.8	35
35	500	1.4	15
35	550	1.3	20

The frequency should be considered in this case, since we have to decide the limit velocity, which depends on the frequency of vibrations.
The suggested limit value is

$$PPV = 8\,mm/s$$

and it is valid if the blast is not close to a building.
Knowing the limit value and the site law, we can estimate the scaled distance $DS_{lim} = \frac{R}{\sqrt{Q}}$ (corresponding to the limit velocity) we should adopt to respect the prescriptions.

Actually, we have to face with different sensitive elements at different distances. Since we can not evaluate many distances, we will perform the evaluation just on 3 ÷ 4 different distances – minimum distances –: for each distance, knowing the limit value DS_{lim}, we obtain the maximum charge we can adopt.

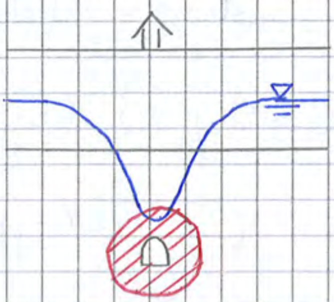
Of course, the biggest is the distance, the biggest will be the charge we can use. On the other side, in case of risk of damage, we will reduce the pool in order to respect the limits, remembering yet that the aim is to design pools as longest as possible.

In order to make a correct use of auxiliary methods and of the supports, we have to answer to two key questions

1) What is the goal for risk reduction? ⇒ It gives the engineering design

2) How to achieve this goal? ⇒ It gives the technology to achieve the goal

Ex (WATER TABLE)



RISK: lowering of the water table

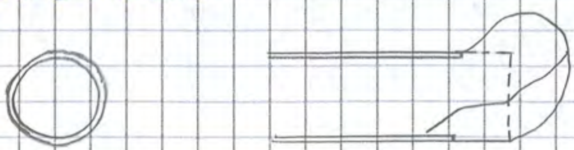
RISK ASSESSMENT: no lowering of the water table (otherwise there would be settlement)

Knowing the assessment, we can do something, e.g. grouting.
The engineering computation consists on the evaluation of the thickness of the anulus.

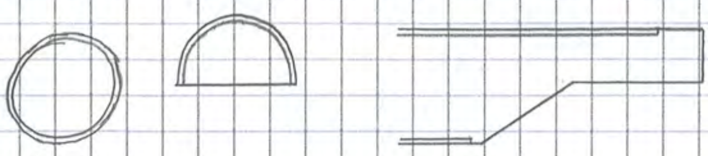
TECHNOLOGY: type of mortar, ratio a/c, thickness
↳ how to reach the aim in design

Ex (FULL FACE ADVANCEMENT)

HAZARD: collapse of the tunnel face, since the face may be not able to remain stable



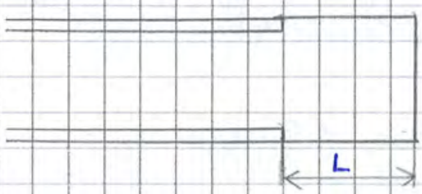
MEASURE (possible): reduction of the height of the face, through head & bench excavation - excavation is performed into two steps



Self supporting time and free span

Tunnel excavation process is a 3D process that occurs close to the face.

This aspect is useful because advancement step requires to advance of certain length and this zone should be stable.



Thanks to the 3D stress state at the face, we have a certain length of advancement which is stable and does not collapse in the time we install the supports.

Of course, the advancement length should be stable because, inside there, we have to install the support, i.e. procedures and materials to guarantee immediate and long term stability.

This is an issue of conventional tunnelling, in which we have alternance of advancement - e.g. blast round - and support installation. For instance, we may make advancement steps of 1 m and install a support of

→ steel arches - also called steel ribs -, which are bended profilates of steel

→ shotcrete, which is concrete projected against the tunnel wall, using compressed air.

The installation is done with a boom having a roof over there, in order to protect the workers. Indeed, the zone is not already protected from local detachment and advancement step should guarantee only the global stability until we install the support, which stabilizes the tunnel.

2 In tunnel excavation, we may face two problems.

→ radial collapse: Free span or self supporting time is too much short

→ face stability: the face is instable

For instance, if we excavate a big tunnel in sand, self supporting time and free span are null and we have problems of face stability.

If the situation brings unacceptable values and we have hazard regarding industrial production, which remedial measures we should adopt? With which properties? when install them?

↳ the typical queries in tunnelling are

1) WHAT TYPE OF SUPPORT?

2) WHICH PROPERTIES SHOULD THE SUPPORT HAVE?
It corresponds to the section, which is derived from the structural computation

3) WHEN SHOULD WE INSTALL THE SUPPORT?

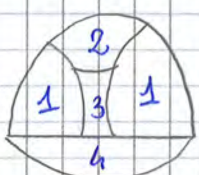
3 Remedial measures

In case of face instability or radial collapse, we can adopt different remedial measures.

I Reduction of the excavation section

We subdivide the tunnel section in smaller excavated areas, based on the principle that the smallest is the tunnel, the easiest will be the stability control.
In Italy, generally we are used to subdivide the section in head, bench and invert.
In Austria, we use the New Austrian Tunnelling Method (NATM), where we reduce the excavated section in small areas and we support them with light steel elements and bolts. The approach is very useful in weak rock.
Actually, NATM is not a method, but just a sequence of procedures following the concept of partialization and a trademark.

Italy



NATM

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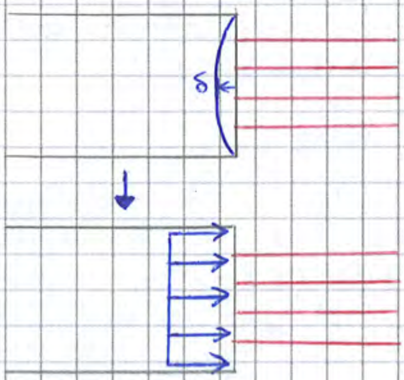
(II) Application of a face pressure

The solution is useful in case of problems of face stability.

In case of MECHANIZED EXCAVATION, the machine is able to apply a pressure at face, which should stabilize the soil and counterbalance the water pressure and keep the water out of the excavation, if the tunnel is under the water table.
To stabilize the soil, we should apply the ACTIVE PRESSURE, in order to be sure that nothing will occur. We may apply the pressure with k_0, but it would be too big!



In case of CONVENTIONAL EXCAVATION, in which excavation is performed in a cyclic process, we install fiberglass bars fully grouted on the face.
The tunnel face tends to extrude towards the excavation and, when the soil tends to move towards the tunnel, the elements tend to resist.
Of course, these bars are a passive device which reacts tending to prevent displacement.
They can be modelled as a pressure applied at face.

(III) Improvement of ground properties

If we excavate the tunnel in weak conditions (e.g. sand), we may change the properties of the material and make it more resistant.
In this way, we will excavate the tunnel in a mass different from the original one and in better conditions.

The method improves, <u>from the engineering point of view</u>, the properties of the rock mass.
Indeed, the kind of method depends on the problem we face

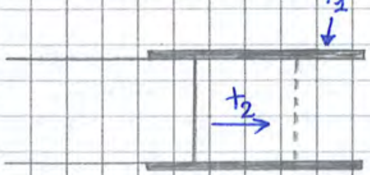
→ problem of stress - strain behaviour: we improve the mechanical behaviour.

→ problem of water: we should just make the system impervious, ignoring the mechanical aspect. For instance, we grout the cracks to prevent the water coming.
↘ it is not like grouting soil to improve it

IV Pre-support

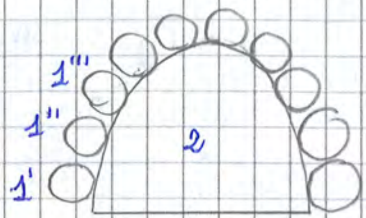
The presupports are methods which use the insertion, in the rock mass, of structural elements ahead the tunnel face, with the purpouse to create a presupport before the excavation.

If we are not able to keep the soil stable and guarantee the free span and self supporting time, we launch something ahead of the face in order to create an arch able to support the loads induced by the excavation close to the face and excavate in safe conditions.
Of course, we create the support ahead of the face and then we excavate and the advancement length will be supported by the element.



The presupport can be also obtained by using a COMBINATION OF IMPROVEMENT TECHNIQUES

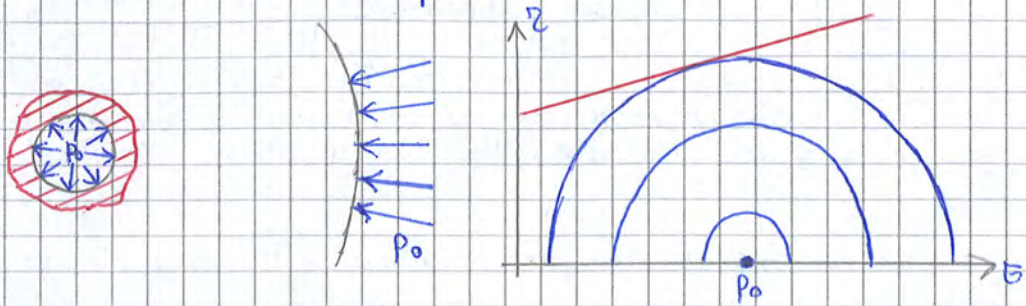
→ mechanical precut - not used anymore
→ forepoling
→ permeation grouting
→ jet grouting arch ("arco con iniezione ad alta pressione")
→ reinforcement with fiber glass elements
→ arch of microtunnels



Microtunnels are realized through small remote-controlled machines able to excavate small tunnels.
The microtunnels are very stable and, if we create an arch of microtunnels connected together and filled with concrete, we can then excavate a big tunnel under the protection of this roof.
The technique is expensive and adopted only in case of serious problems.

If we plot the Mohr-Coulombe circles, we start from a point at p_0 - hydrostatic state of stress - and then the state of stress in the generic point in the contour is growing due to the reduction of internal pressure.

⇒ we start to move in the elastic domain and, at certain moment, due to the redistribution of stresses in the medium, the stress state touches the yield curve.



In the convergence-confinement curve, the relationship displacement-internal pressure is linear during elasticity.
When we start to have plastic behaviour, the link stress state-displacement is not more described by a linear behaviour and the equation changes. In particular, we have a greater increase of displacement than in simply elastic behaviour.
In plastic behaviour, we can notice the formation of a ring of plasticized material and the smallest is the internal pressure, the largest will be the ring.

We can notice that, if rock mass is weak, the curve will be more flat. Furthermore, from the mathematical point of view, the curve goes to infinite; physically, after certain displacement we have the collapse of the tunnel.

When the face goes closer to the section, we move along the curve. The representative point B can be before or after the yielding point, depending on the rock mass quality.
The important aspect is that, from the diagram, we notice that at the tunnel face we have already experienced a displacement δ_F – radial displacement at the tunnel face –, induced by the fact that tunnel behaviour is 3D.

When the face has passed the section, we get a representative point C somewhere along the curve.

The convergence - confinement approach is good because, in the same diagram, we can combine the behaviour of the mass and the behaviour of the SUPPORT:

in this case, the support is a circular ring due to the hypothesis of axisymmetric geometry – it is a stressed ring.

How it works?

Soil wants to radially move but the support, with its stiffness, starts to prevent this movement.
By consequence, there is a mutual interaction between soil and support, in which the soil tends to move but the support reacts against the movement, due to its own stiffness.
The reaction law of the support is related to its stiffness.

Moreover, we are able to install the support only in the free span, i.e. the excavated span, and it is going to work when we advance with the face. In this way, the support starts to work from δ_F and then it reacts against displacements following the reaction law.

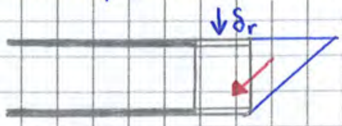
⇒ Supports and other stabilizing elements do not start at nil displacement but after a certain displacement at face

The displacement at face can not be controlled from inside the tunnel, but only before.

The intersection between the convergence - confinement curve and the reaction law defines the equilibrium point, in which we reach the equilibrium.
Here, the tunnel has experienced certain displacement and the support is subjected to certain stress state and these quantities depend on the stiffness of the support.

Face stability design in soils

1 We know that the excavation process is characterised by a 3D problem, since we have



→ problem of face stability

→ problem in advancement step

2 In soil, we can have two problems of face stability

→ collapse of a zone which is confined and close to the excavation

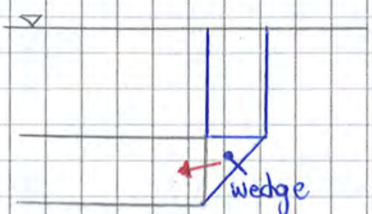
→ in shallow tunnels (or low overburden tunnels), we have CHIMNEY CAVING ("fornello"), i.e. collapse of a zone which reaches the surface.

The possible tools we can use to analyse the face stability are

→ NUMERICAL 3D CALCULATIONS

→ ANALYTICAL CALCULATIONS:

they are widely used and they are based on the limit equilibrium models, taking into account different geometries of the sliding body.



The typical scheme is a two-body scheme, with

→ the sliding of a wedge at the face

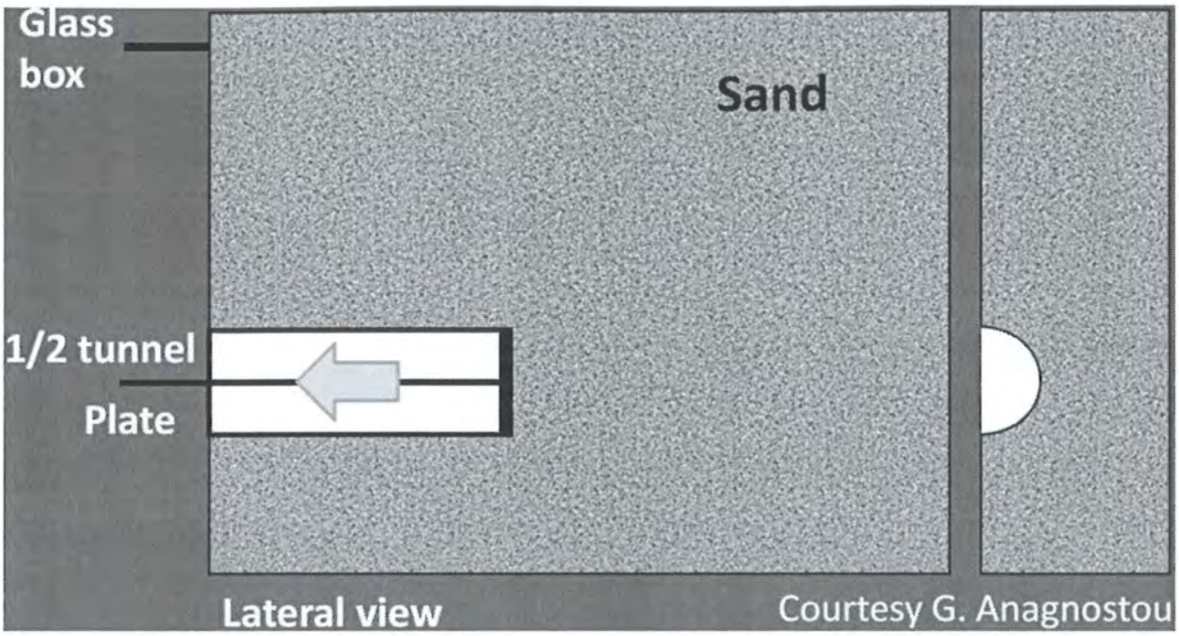
→ the sliding of a prismatic body above the wedge

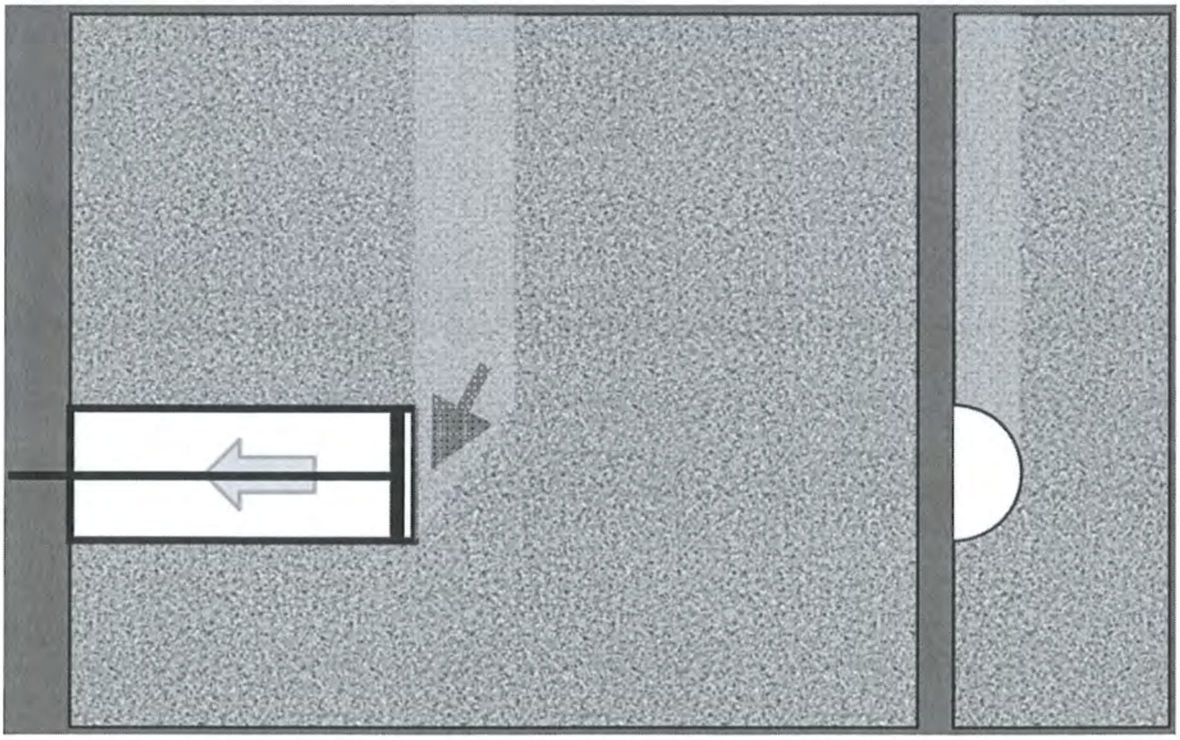
In the models, we can adopt different geometry and hypotheses.

Ignoring the single specific models, this kind of method is reasonable, as shown by Anagnostou:

he realized a simple model developped in a half-box filled with sand, where he realized a tunnel stabilized with a plate pressed against the face.
Then, the plate is moved and, moving it, we notice the formation of

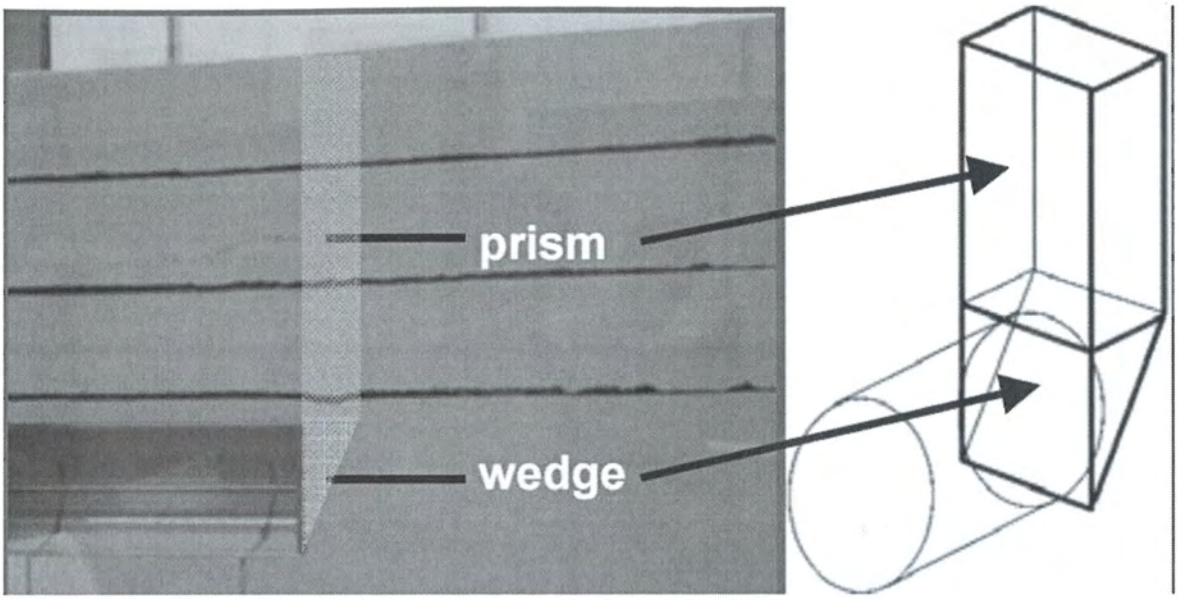
TEST ON DRY SOIL





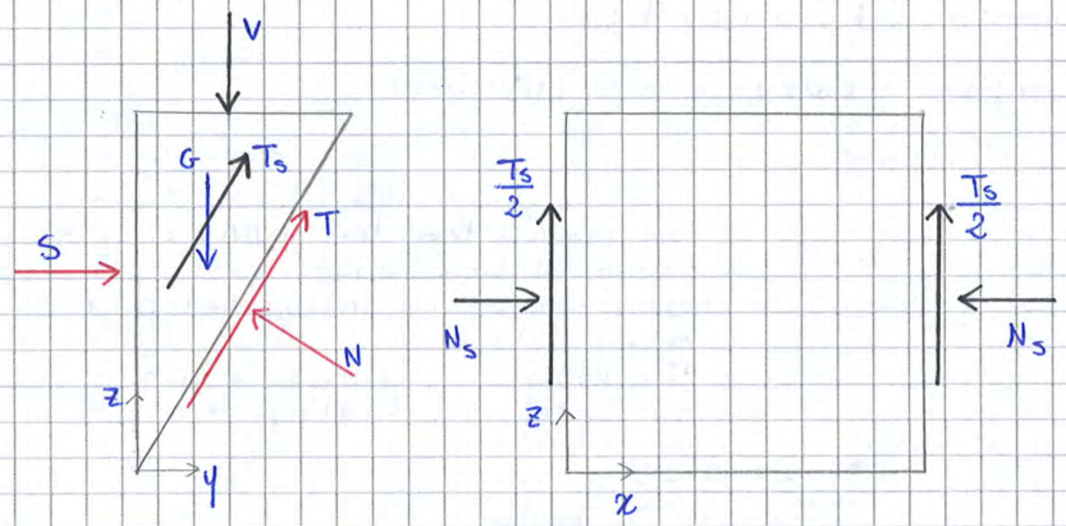
Schematic representation of the behaviour

Passage failure mechanism – computational model



→ Mohr - Coulomb yielding criterion on the sliding surfaces

We focus on the forces acting on the wedge
→ Force S applied for stabilization
→ Force V applied by the prism on the wedge
→ acting forces T, N on the sliding surface (T is against movement)
→ weight of the wedge G
→ acting forces T_s, N_s on the two lateral surfaces (T_s is against movement)

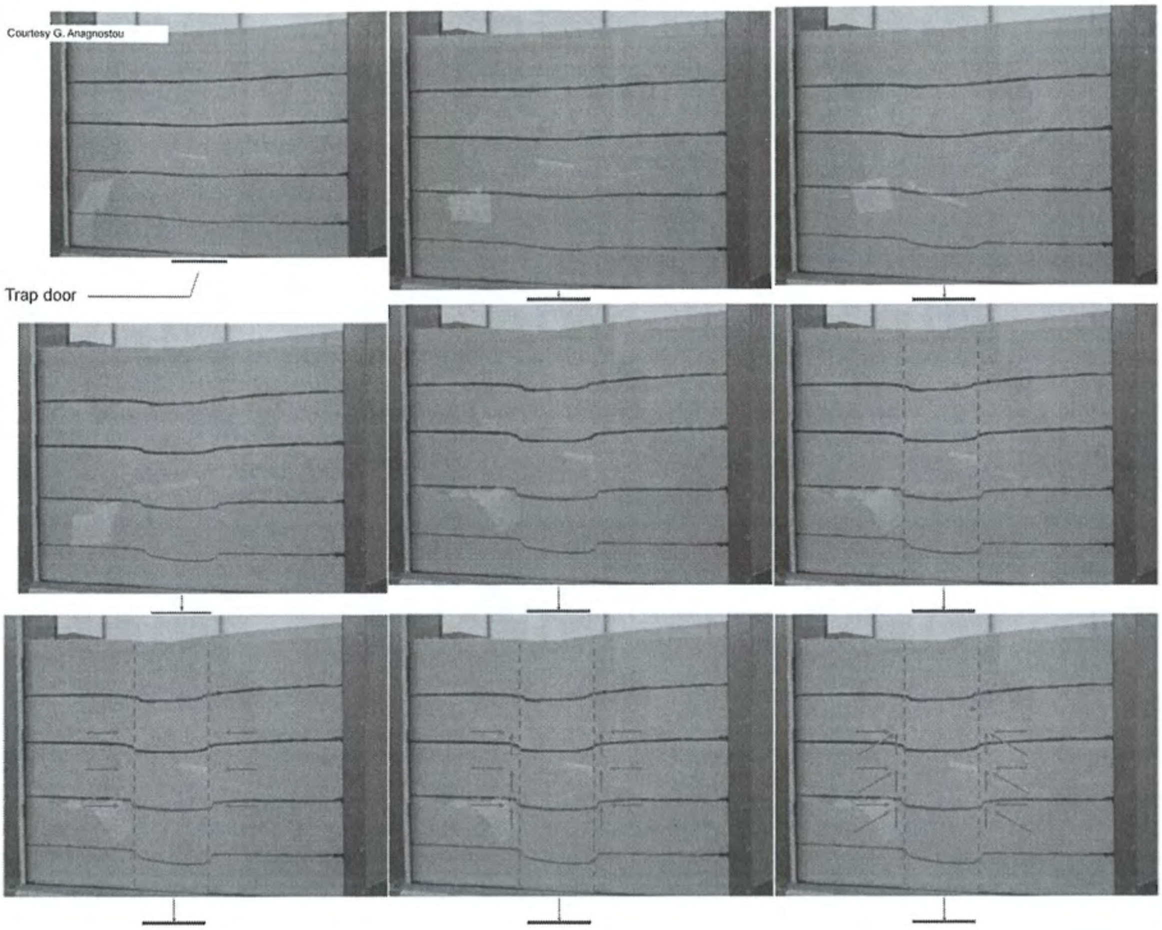


In order to evaluate the force V, we refer to silo theory:

to understand the phenomenon, Anagnostou developped an experiment called TRAP DOOR TEST.
A glass box is full of sand and present a trap door at the bottom.
When we open the trap door, we see two slip lines on the periphery of the box, which correspond to the sliding surfaces of the silo.
On these surfaces, are acting normal stresses and therefore shear stresses

⇒ the silo would like to move in certain direction but, due to normal stresses induced by soil, we have shear stresses.

As consequence, we have an ARCH ACTION.



Evolution of the trap door test

We substitute and we find a differential equation in unknown σ_z.

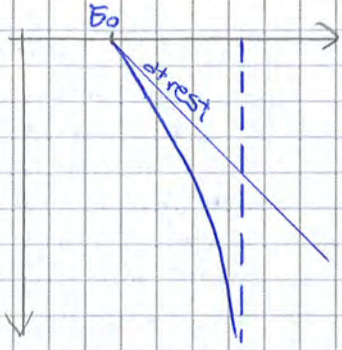
$$\frac{d\sigma_z}{dz} + k_0 R \tan\varphi' \cdot \sigma_z = \gamma \frac{c}{R}, \qquad R = \frac{BL}{2(B+L)}$$

The boundary condition is the following.

$$\sigma_z(z=0) = \sigma_0$$

The solution assumes the following general expression.

$$\sigma_z = f(z; R; \lambda; \tan\varphi'; c'; \gamma; \sigma_0)$$

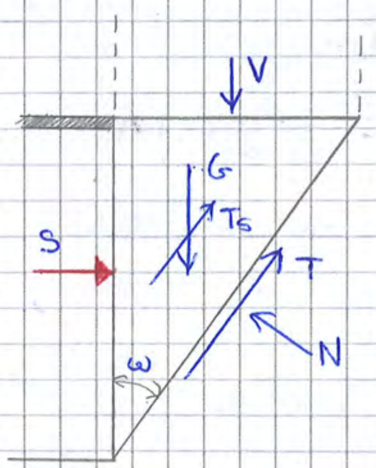


If we plot the solution, we can notice that, after certain depth, THE SILO STRESS DOES NOT INCREASE ANYMORE WITH DEPTH.

That's why it is possible to excavate very deep tunnels.

Indeed, if we made a deep hole and we had to sustain with a structure thousands of metres of overburden, it would be impossible. Adopting instead the reasonment of silo theory, we notice the creation of an arch of stress release when we have the movement - due to the movement, we have friction forces that stabilize the system (not in undisturbed conditions). This allows the reduction of the applied load.

At this point, we can consider the limit equilibrium of the wedge.



The wedge is subjected to the force V, defined from silo theory, and the force G, which is self-weight and comes from geometry.

We have 3 unknowns T, N and S and we evaluate them through

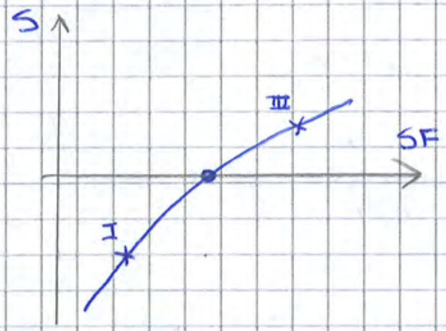
→ equilibrium in parallel direction to the sliding surface

→ equilibrium in orthogonal direction to the sliding surface

→ Coulomb condition - link T-N

In the case that the face is stable, we can evaluate the safety factor for face stability:

the safety factor physically means how far we are from collapse.
To evaluate it, we can apply a safety factor to geotechnical parameters - cohesion and tangent of friction angle - and we repeat the calculus, obtaining different curves.
We repeat the process until the curve touching the limit equilibrium and this corresponds to the safety factor related to geotechnical parameters.



If we plot the force with reference to the safety factor

→ the curve I will be below

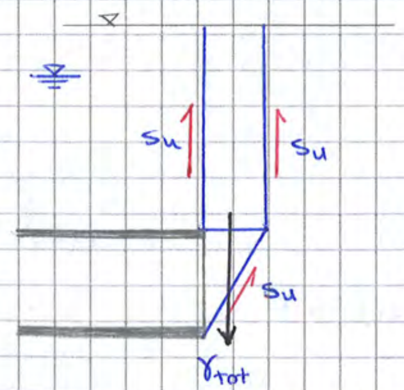
→ the curve II will be above

Actually, we can face different conditions.

→ very permeable ground above the water table:

we use the approach defined previously

→ low-permeability ground



In this case, we should refer to a SHORT-TERM ANALYSIS, before the starting of any filtration of water towards the tunnel.

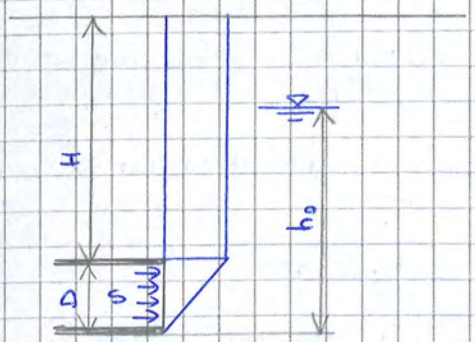
It corresponds to a total stress analysis, considering

→ undrained shear strength s_u

→ total unit weight γ_{tot}

in a certain scheme.

→ SEEPAGE FLOW



The first two members are linked to geotechnical properties, the other two to the effect of seepage flow.

$$S = F_0 \gamma' D - F_1 c + F_2 \gamma' \left(h_0 - \frac{D}{2} \right) + - F_3 c \frac{h_0 - D/2}{D}$$

→ NO SEEPAGE FLOW – hydraulic equilibrium (typical of full face machines)

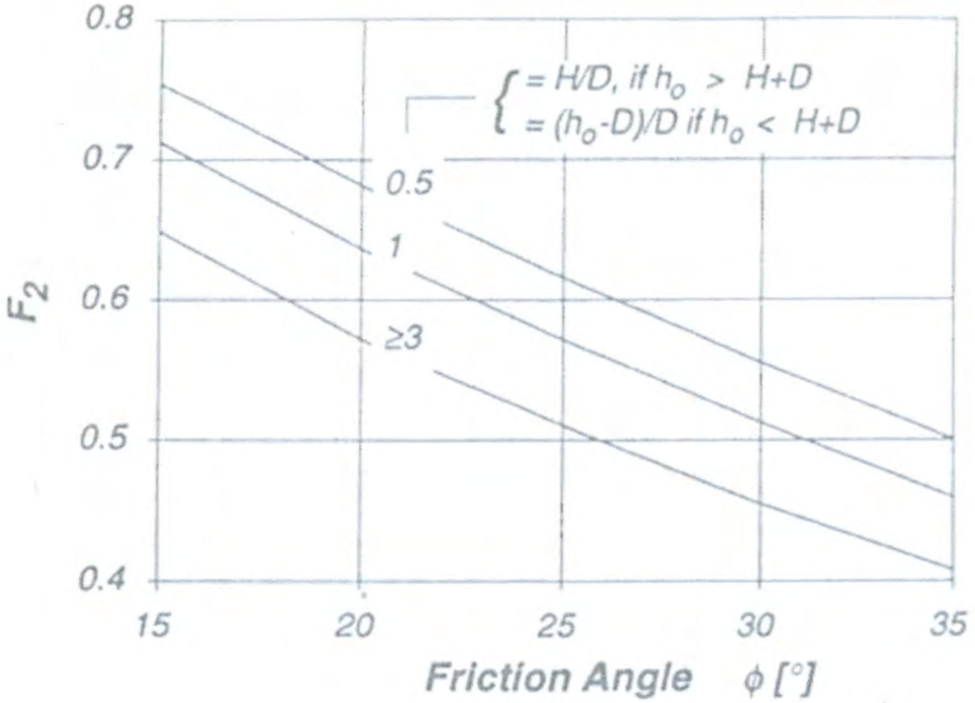
In this case, we have no filtration forces and we have just to compute water pressure.

$$S = F_0 \gamma' D - F_1 c + \text{water pressure}$$

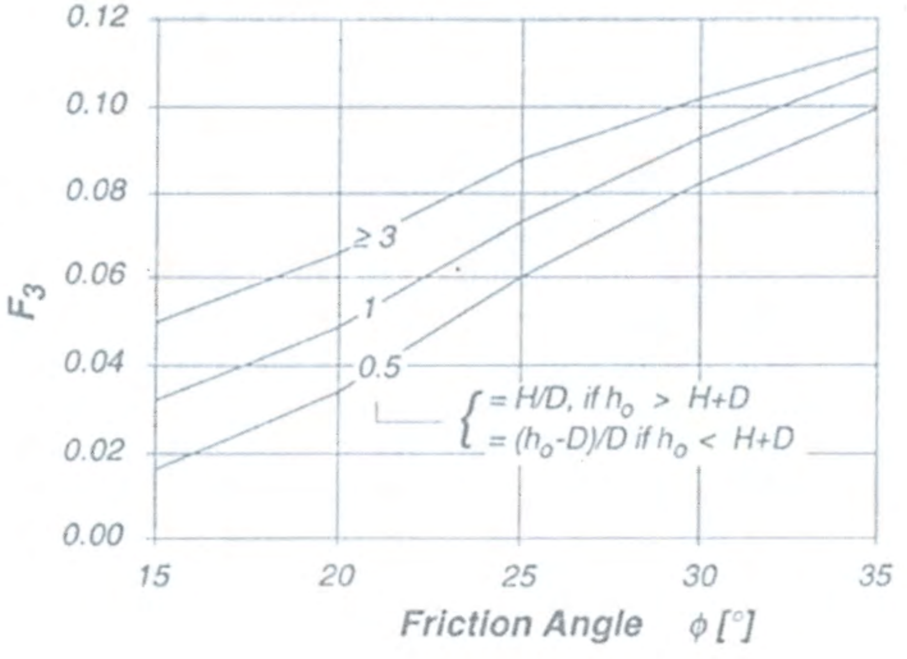
→ NO WATER

$$S = F_0 \gamma D - F_1 c$$

The parameters F_0, F_1, F_2 and F_3 are given in different curves, depending on the position of water and on friction angle of soil.



F_2 parameter



F_3 parameter

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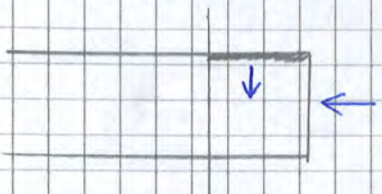
Technology of supports for conventional tunnelling.

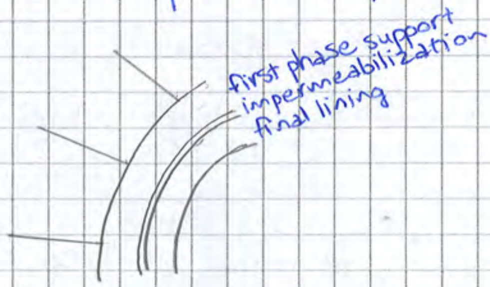
When we make a tunnel in conventional excavation, we have an advancement with certain length, which is related to stability and rock mass quality.



When we design the advancement step, we need to control displacements at face and radial displacement.
We have already seen the problem of face stability and the problem of radial stabilization of the tunnel, in the convergence-confinement method.

Now, we will see what to install in the already excavated part, in order to stabilize the tunnel.

In the supports, we find
- → first phase support
- → impermeabilization
- → final lining



The first phase support's goal is to stabilize the tunnel during excavation and give stability and safety, also for workers.
After some time, we install the final lining and, between them, we have an impermeabilization layer.

First phase support

1 The most common elements used for the first phase supports in conventional tunnelling are

→ steel ribs ("centine" - also called steel arches):

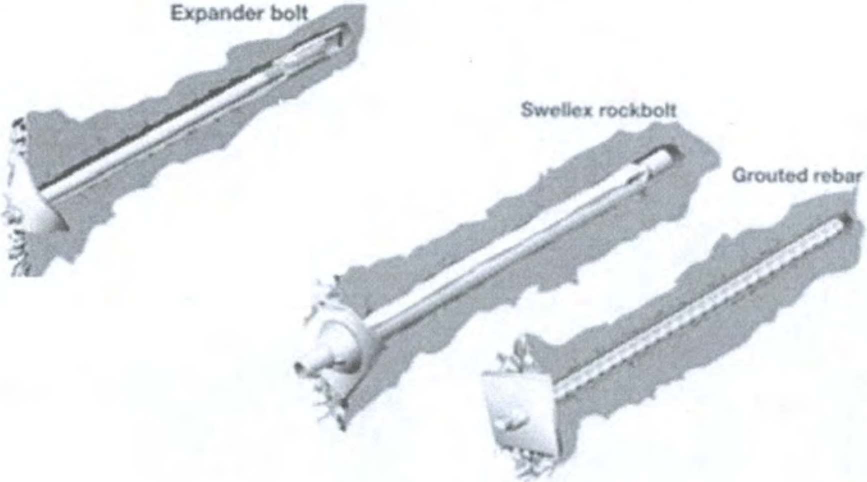
they are metallic frames bended in order to obtain a certain curvature radius.

→ Shotcrete ("calcestruzzo proiettato"):

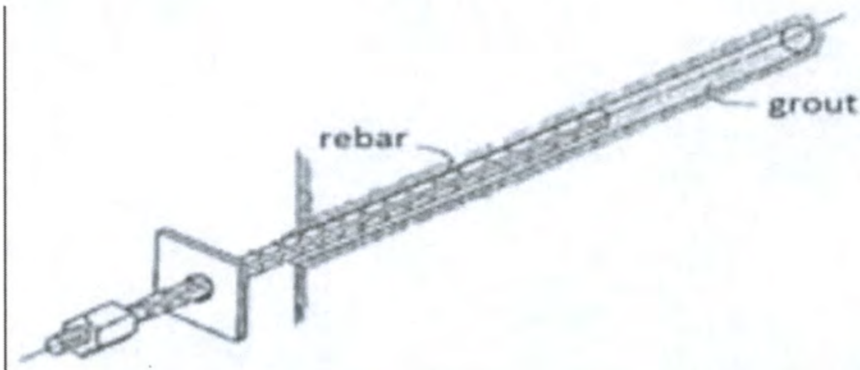
it is concrete with special grain size distribution of inert, spread against rock mass by using compressed air, in order to create layers.
It maybe reinforced with some wire-mesh layer ("rete elettrosaldata") placed between the ribs.
The shotcrete creates an arch of concrete to support the tunnel.

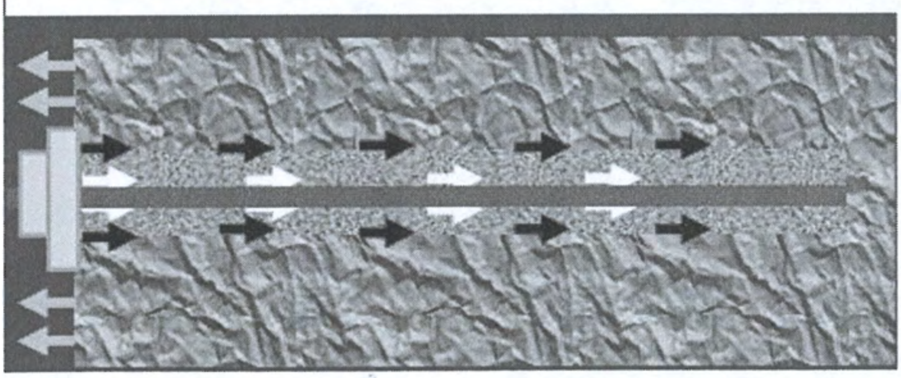
ROCK BOLTS

Rock bolts



Grouted rebars





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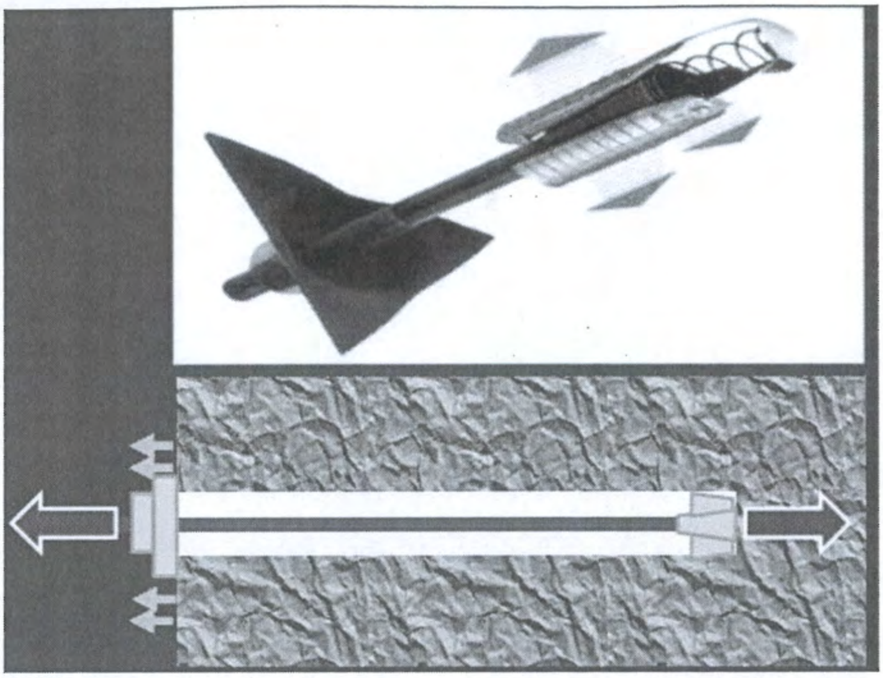
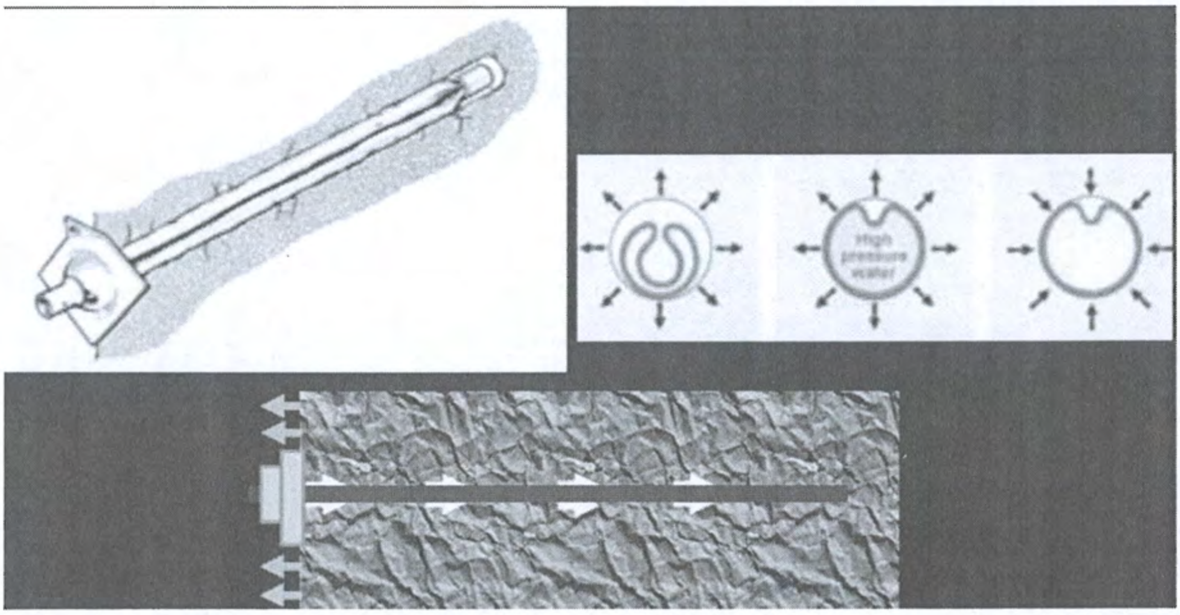
Expander bolts



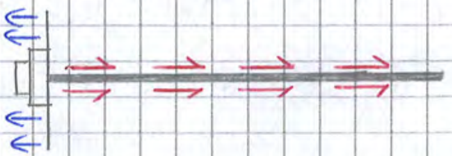
Swellex bolts



→ Swellex bolts

They are bolts made by mild steel ("acciaio dolce"), bent in certain way and closed on one end and open on the other one. They are like a sack.

When they are inserted in the hole, they are expanded by using high pressure water - more than 80 bars - inflated at the end.
The shape will deform up to a final one and, due to deformation, the steel is in deep contact with the rock mass with certain pressure and shear forces arise when rock mass tries to move.

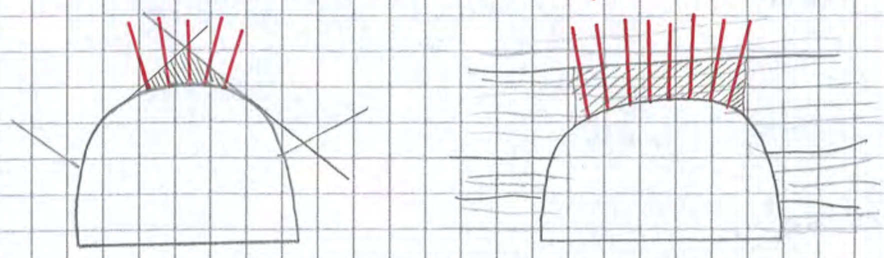


The connection is different from the one of grouted rebar, in which the linking is in the mortar. When the rockmass tends to move, shear forces arise at the interface between rockmass and the grout and at the interface between the grout and the bar. These shear forces are able to allow the bolt to apply the support action.

In swellex bolts, shear forces arise only in the connection between the steel and the rock mass.

Swellex bolts are fast and of immediate activation - in grouted bar, mortar needs some time to be active.
On the other side, they are temporary elements since they are not protected against water.

Rock bolts are employed to support unstable wedges around the tunnel and reinforce unstable layers of horizontal beddings.



3 Steel ribs

Steel ribs are metallic elements installed inside the free span - it should be stable in order to allow the installation.
They are generally covered by shotcrete and we usually install one rib per advancement.
If ock mass is good, the advancement length is bigger and we install 2 ribs.

In Italy, we use different types of steel ribs.

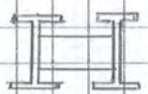
→ IPN geometry, characterised by small wings and large thickness

→ HEB geometry

→ UPN geometry, widely used in USA

→ LATTICE GIRDER ("centina reticolare"), which is very effective because it provides large bending moment with less weight, but it is expensive.
Generally, lattice girders are used only in very large span because it is easier to install at job site, since they are light.

→ IPN coupled

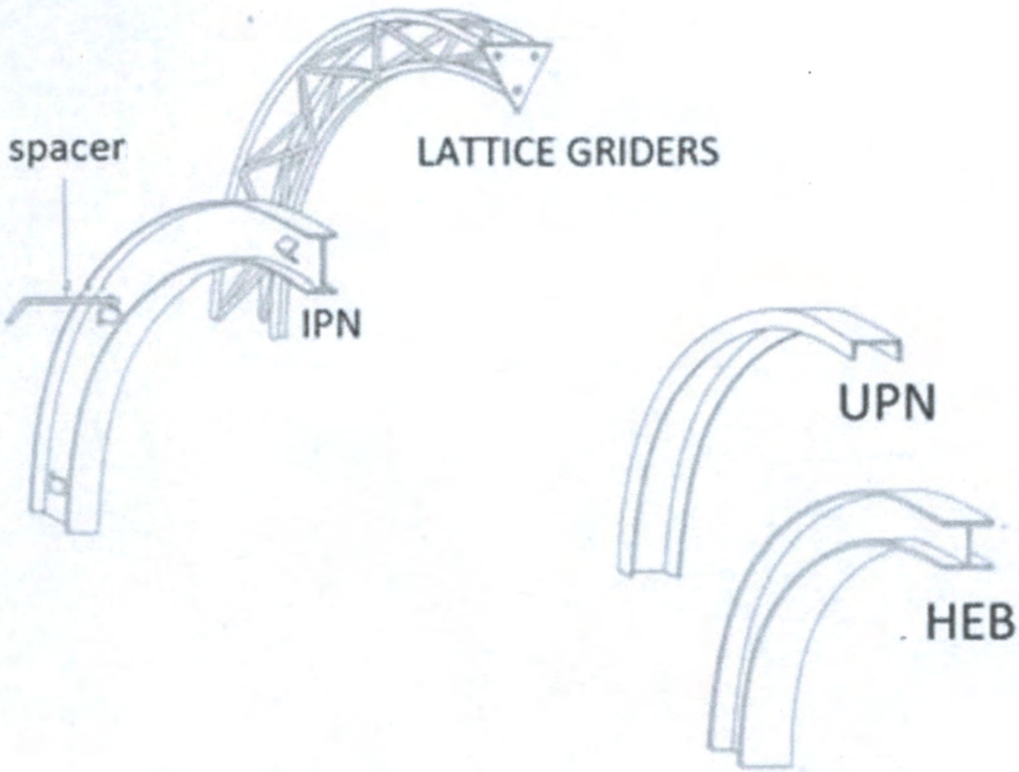


They are made of two IPNs coupled with cross stiff elements, welded to them.
They have been introduced because to bend a profile is a complex operation and bending machines can not bend elements thicker than 30 cm.

By consequence, to get high bending moment without using thick supports, we can ~~use~~ put together more elements with smaller thickness.
Furthermore, since steel ribs have to be completely covered by shotcrete, if we put with the same moment elements with smaller thickness, we save shotcrete

⇒ MAXIMUM THICKNESS IS $22 \div 24$ cm, to ensure the possibility of bending and reduce the amount of shotcrete

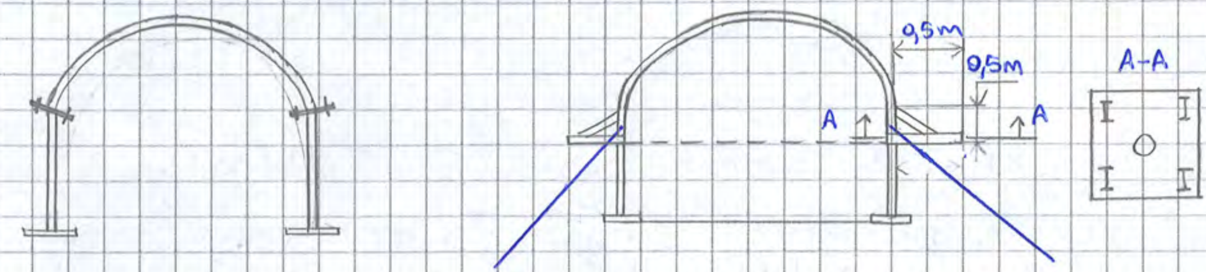
STEEL RIBS



Couplers

An important detail is the support plate, which is a steel plate placed on the lower part of the rib in order to enlarge its footing.
In this way, we reduce the stress applied to the soil and we guarantee a good foundation to soil.
Furthermore, if we adopt the head - bench excavation technique, the footing of the upper part of the rib is an enlargement used as bolting plate to place the second part of the arch. This enlargement should be only on the lateral part and can not enter inside the section.

In case of weak rock mass, we can reinforce the foundation with micro-piles which are installed through the plate.
This reinforcement is generally done to give stiffness to the system.



4 Shoterete

Shotcrete is concrete which is installed by using a spraying system ("sistema di spruzzo") with compressed air through a nosel ("ugello").

Generally, the installation system is made of a normal concrete pump and a pipe carrying concrete - it is produced in a concrete batching plant. This is connected with a pipe of compressed air and sprays the concrete.

Since concrete is not poured but sprayed, it should stick to rock and we request

→ HIGH ADHESION PROPERTIES

→ HIGH HARDENING VELOCITY

The hardening velocity is requested because we can not advance until concrete is not hardened. It is not necessary to have immediately complete hardening but just harden to a certain extent because, thanks to the transverse arch effect (3D state), the load transmitted to this portion is not the whole load.

the whole load is reached at certain distance from the excavation face

→ WATER-CEMENT RATIO $0,4 \div 0,55$

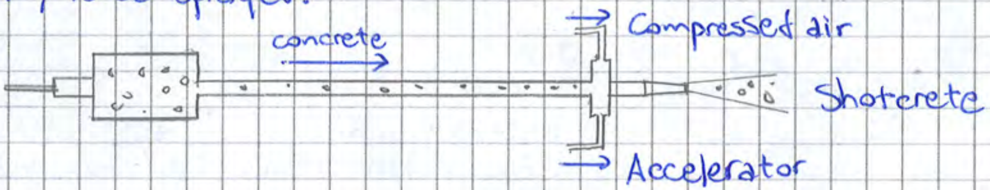
It is reasonably high because shotcrete should be fluid.

→ MAXIMUM COARSE AGGREGATE 40%

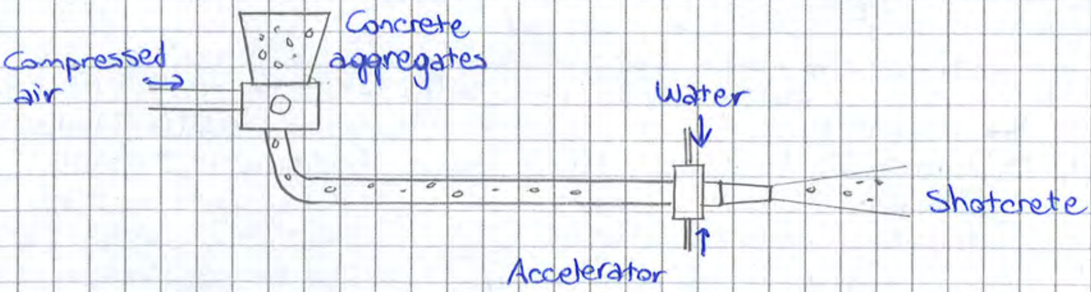
→ PLASTICIZER (for fluidity) $5 \; kg m^{-3}$

SHOTCRETE TECHNOLOGY

The most used technology is the wet-mix method ("calcestruzzo proiettato a umido").
Concrete arrives in the working place already prepared in the batching machine and mixed with water and aggregates.
Then, it is pushed and mixed with accelerators and compressed air, to be sprayed.



The dry mix method is used mainly for reparation in Italy.
In this case, concrete and aggregates arrive dry, they are expanded and then pumped by using compressed air in a dry mix.
Water and accelerator are just added in the nosel.



The dry mix method requires small machinery and the material can arrive in bags.

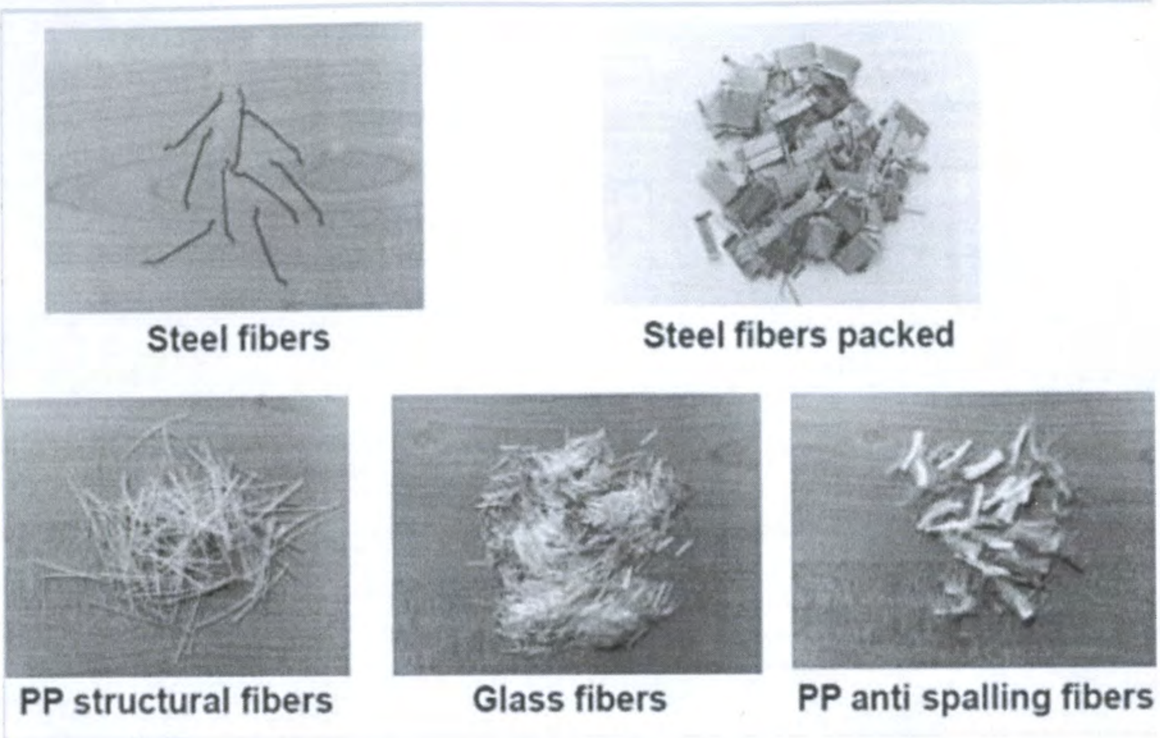
Generally, shotcrete is applied in layers, since it is difficult to apply a layer of certain thickness in an unique shot.
The number of layers is established by the operator, which should be skilled.

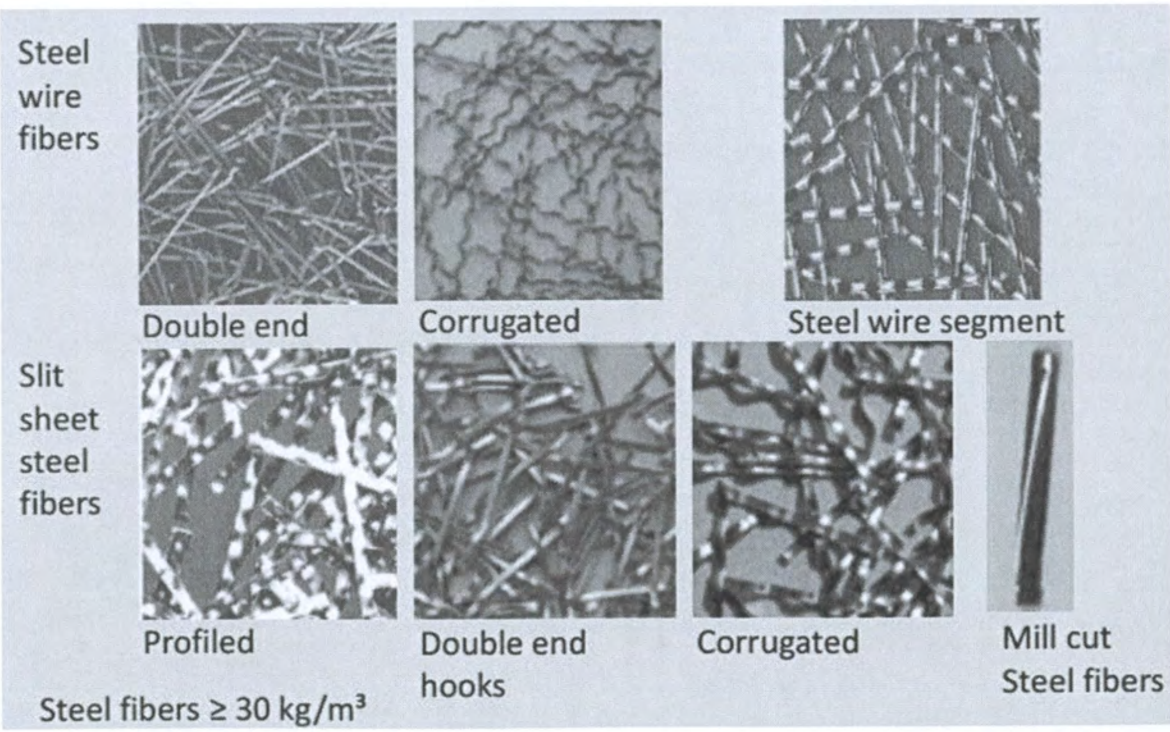
SHOTCRETE

Dry mix method VS wet mix method

Factor	Dry mix
Equipment	Lower investment than wet mix Maintenance relatively simple and infrequent
Mixing	At the jobsite or at the mixing/batching plant Premixed, dry ingredients can be used but cannot be left open in humid or wet environments Performance impaired by wet sand
Output	Rarely exceeds $5m^3/h$ in place Can be conveyed over longer distance than wet mixes
Rebound	Can be 15-40% from vertical walls; 20-50% from overhead
Quality	Higher strenght, due to lower W/C ratio Less homogeneous quality
Impact velocity	Higher – better adhesion; easier to use overhead
Additives	Powders added in mixer. Liquid at the nozzle.
Dust	Higher production of dust

Fiber reinforced shotcrete

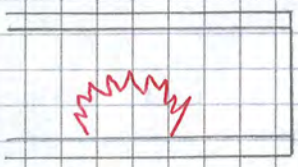




Nowadays, we tend to use polypropilene structural fibers, or plastic fibers.
They are better than steel fibers because they do not rust, but they have big ductility and, at long term, they elongate.

They are very important for ANTI SPALLING, especially applied for FIRE PROTECTION:

the aspect is not important for shotcrete but in the final lining, especially when we apply these fibers in the segments used in mechanized excavation.



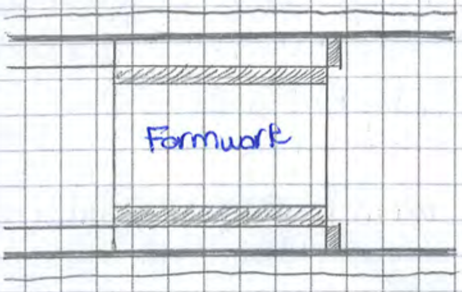
In case of fire inside the tunnel, concrete reaches a high temperature.
Then, when fire brigade arrives, they spread water which goes on a very high temperature surface.

This brings to a sudden decrease of temperature, corresponding to a sudden stress release and concrete will slab ("si sfoglietta violentemente").
This is dangerous for fire brigade - concrete pieces are projected like bullets - and brings damage to concrete.

Plastic fibers allow to give tenacity to the immediate stress release.

OBSERVATION: fibers may cause problems in mix design with water and the water-cement ratio may change due to plastic fibers.

2 Casting of the final lining



The formwork is moved inside the tunnel and it presents a steel structure, which leaves an anular gap that will be closed with concrete.
On one side, the formwork is closed by the already cast in place concrete; on the top, it is closed by the first lining and the impermeabilization; on the other side, it is closed by hand using wooden piece ("smorza").

Between the segments, there is a cold joint and, if the impermeabilization layer failed, water would lick inside the tunnel from there.

By consequence, in some tunnels, we install a WATER STOP between the elements.
It is a plastic slab embedded in two sections of the final lining and going around, in order to stop water.

Typically, the final lining is cast in place into two phases

(I) lateral part ("murette"), used as base of the formwork of the arch.

(II) arch, which is not reinforced or reinforced with a steel frame of rebars, depending on stresses.
In this case, the design approach assumes that, in long term, the first lining support effect plays no role because it is not protected and it is altered by water. Thus, all the loads considered in the design of the primary lining will go on the final lining.

MECHANIZED EXCAVATION

There is a table, prepared by French tunnelling society, which clarifies what are the tunnelling machines and how they work.

The table starts from STABILITY CONDITIONS (support):

does the excavation require some support?
In other words, is the excavation stable? And is support needed just on the cavity or also at the face?

In this way, the table separates full face machines with reference to stability problems.

→ no stability problems:

In this case we can use open TBMs ("TBM aperte"), also called "unshielded TBMs", where we do not need specific support for the tunnel.
The problem is not stability but the fact that rock mass is so robust that there is DIFFICULTY IN DEMOLISHING ROCK MASS.
By consequence, the important aspect is to design tools and machine power in order to break the rock mass.

↳ in this situation, in conventional tunnelling, we use drill & blast

→ problems in cavity stability:

this is the field of application of shielded TBM ("TBM scudata"), which is a machine protected by a steel cylinder called SHIELD.

→ problems in cavity and face stability:

this is the case of SOIL MACHINES, especially of shields ("scudi"), which are systems able to stabilize the cavity and to apply a pressure at face.

Nowadays, in tunnelling industry, soil machines and rock machines are starting to become hybrid and able to work both in rock mass and soil.

OVERVIEW OF FULL FACE MACHINES

<table>
<tr><th colspan="4">Support</th><th colspan="2">Excavation</th><th rowspan="3">Reaction Force</th><th colspan="2">Machine</th></tr>
<tr><th rowspan="2">Location</th><th colspan="3">System</th><th rowspan="2">Method</th><th rowspan="2">Tool</th><th rowspan="2">Category</th><th rowspan="2">Type</th></tr>
<tr><th>Cavity</th><th colspan="2">Face</th></tr>
<tr><td colspan="4" rowspan="2">None</td><td>Partial Face Excavating Machines (PFM)</td><td>Various</td><td>None or gripper</td><td rowspan="4">Rock Machines</td><td>Other</td></tr>
<tr><td></td><td>Cutting disk</td><td>Grippers</td><td>Special Undershielded TBM</td></tr>
<tr><td rowspan="3">Cavity</td><td rowspan="3"></td><td colspan="2" rowspan="3">None</td><td rowspan="2">Full Face Rotating Cuttings Head (TBM)</td><td>Cutting disk / Cutting bits / Cutting knieves & teeth</td><td>Thrust Jacks</td><td>(SS - TBM)</td></tr>
<tr><td>Cutting disk</td><td>Grippers and Thust Jacks</td><td>(DS - TBM)</td></tr>
<tr><td>PFM</td><td>Hod Header / Back hoe / Manual excavation</td><td>Thust Jacks</td><td></td><td>Open Shield</td></tr>
<tr><td rowspan="8">Face and cavity</td><td rowspan="8">Shield</td><td colspan="2" rowspan="2">Mechanical</td><td>TBM</td><td>Cutting bits / Cutting knieves & teeth</td><td rowspan="8">Thrust Jacks</td><td rowspan="8">Soft Ground Machines</td><td>Mechanical Supported Closed Shield</td></tr>
<tr><td>PFM</td><td>Rod Header / Back hoe</td><td>Mechanical Supported Open Shield</td></tr>
<tr><td rowspan="5">Fluid</td><td rowspan="2">Compressed Air</td><td>TBM</td><td>Cutting bits / Cutting knieves & teeth</td><td>Compressed Air Closed Shield</td></tr>
<tr><td>PFM</td><td>Rod Header / Back hoe / Manual excavation</td><td>Compressed Air Open Shield</td></tr>
<tr><td rowspan="2">Slurry</td><td>TBM</td><td>Cutting disk / Cutting bits / Cutting knieves & teeth</td><td>Hydroshield</td></tr>
<tr><td>PFM</td><td>Rod headerb/ Back hoe</td><td>Slurry Shield</td></tr>
<tr><td>Earth Pressure Balance</td><td rowspan="2">TBM</td><td rowspan="2">Cutting disk / Cutting bits / Cutting knieves & teeth</td><td>Special EPBS</td></tr>
<tr><td>None or fluid</td><td>None or Slurry or Earth Press Balance</td><td>Polishield</td></tr>
</table>

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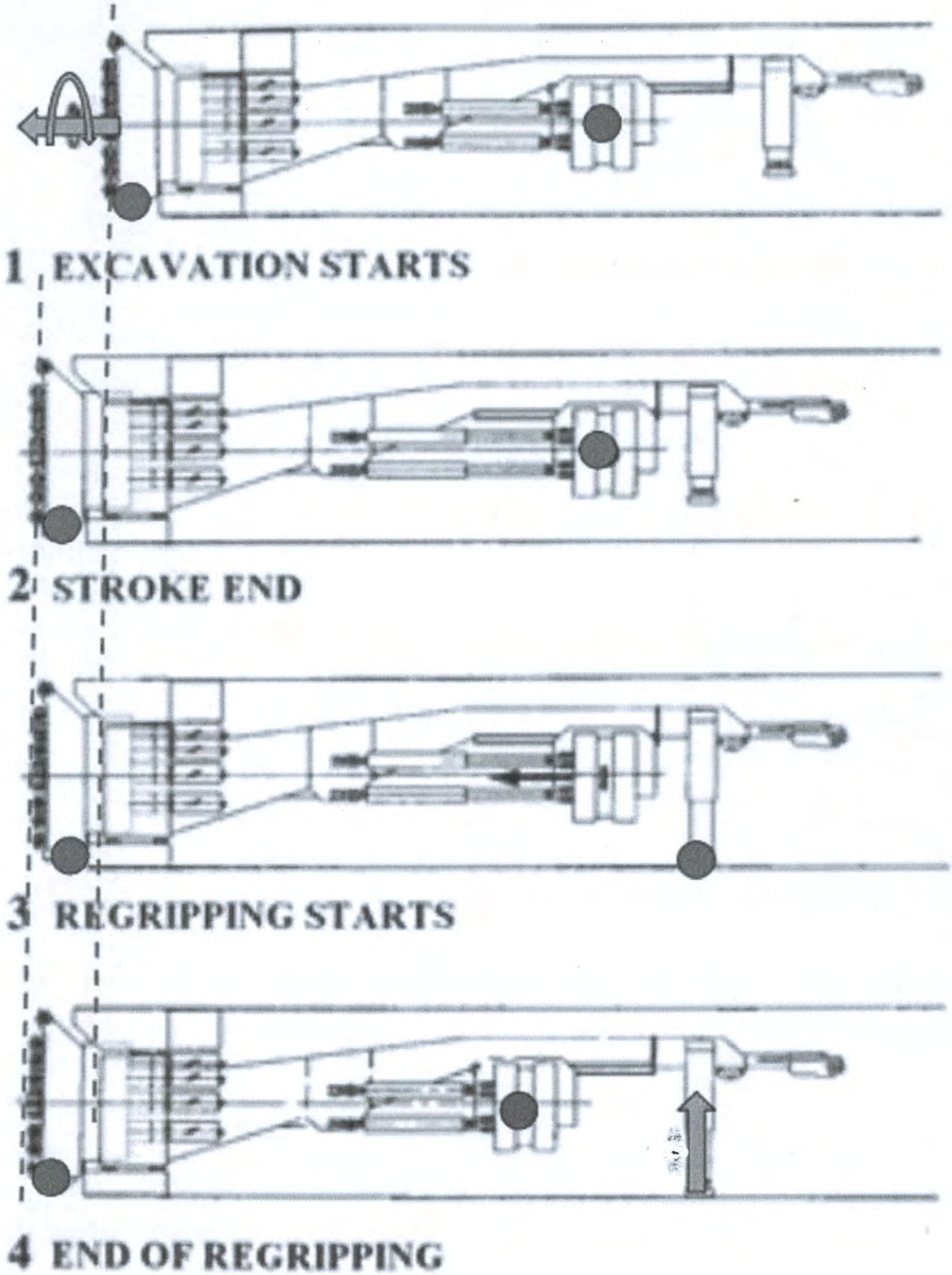
Advancement with the gripper (1)



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The charriots are assembled together and the system may reach a length up to 100 m - it can be thought as a moving factory.

Firstly, the cutter head is pushed forward; then, the back-up moves.

DEVICES

Rock TBMs sometimes present a small shield, used just to give protection from small detachments immediately close to the cutter head, since we use open TBMs in stable rock mass (no pro

↳ no problems of global stability, just local detachments

Open rock TBMs are able to apply bolts to avoid the risk of detachment of single wedges.
Bolts are the typical supports used in this case because they do not modify the section and their installation does not interfere too much with advancement.
Sometimes, these machines are also able to apply shotcrete because they have a shotcrete area in which it is applied. This solution is easy in case of big cross section.

OBSERVATION: TBM are generally taylor-sewed machines, i.e. they are projected and constructed by the producer for the specific project, starting from specific constraints

⇒ the machine is part of design and we have to be clear in mind about the specific constraints.

For instance, if we want to investigate or intervene ahead of the face (e.g. we want to introduce inclined piles to create a pre-vault), we will introduce holes in the cutter head.

Of course, we can modify the machine during the excavation, but it takes lots of time and money.