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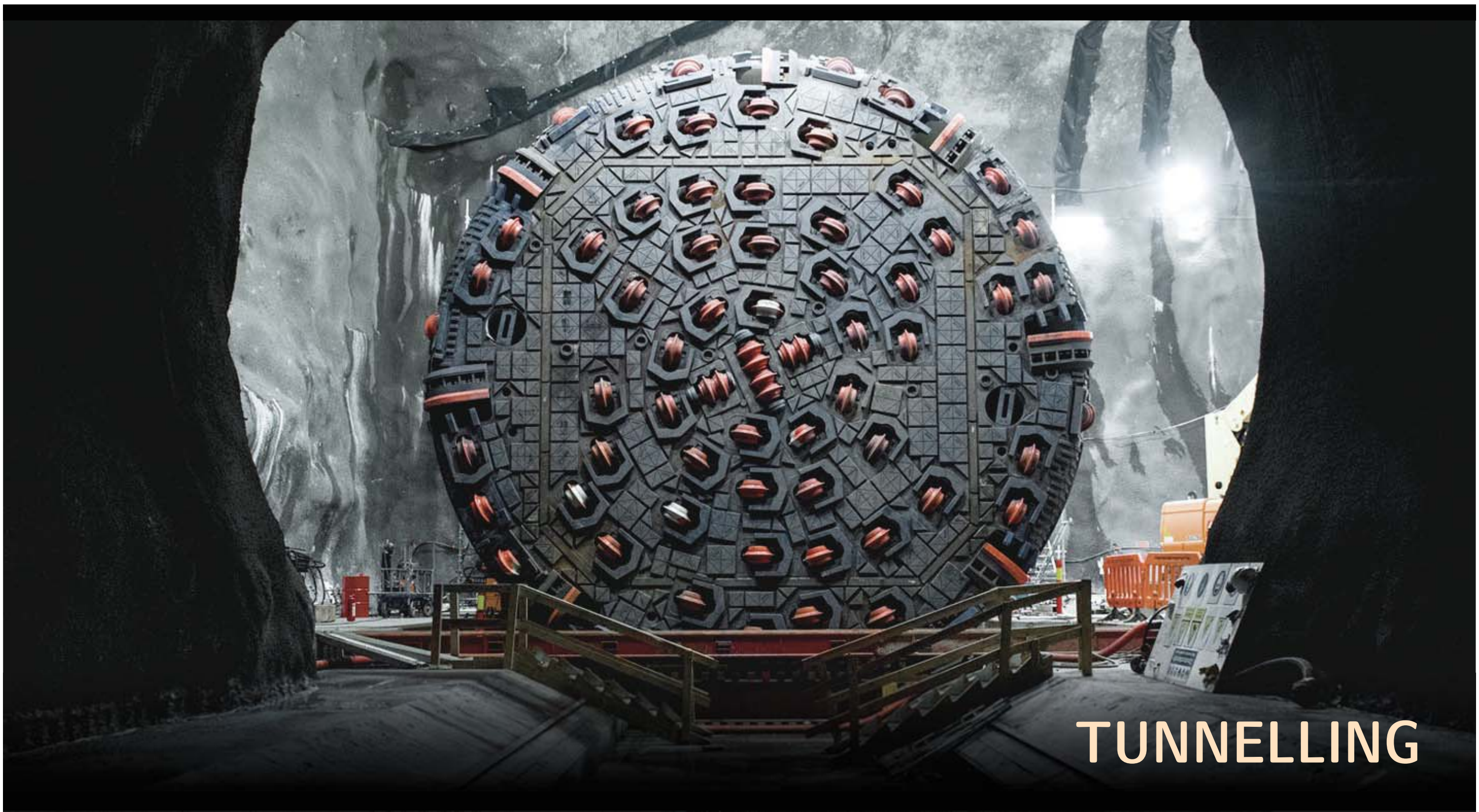

TUNNELLING

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1. GENERAL ASPECT OF TUNNELING

DIFFERENCE BETWEEN SURFACE CONSTRUCTIONS AND UNDERGROUND CONSTRUCTIONS

ABOVE GROUND CONSTRUCTIONS

- The defined properties of the construction material are guaranted during the construction process.
- The loads are well-known defined by codes, so the loads, for which the structural analyses are carried out, are mostly known or can be clearly assessed.

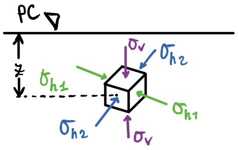
UNDERGROUND CONSTRUCTIONS

- The real construction material is the ground, and the graud is uncertain (le proprietà dei terreni sono conosciute con un certo grado di incertezza). It is difficult to change the rock mass/ground properties notwithstanding (nonostante) ground improvements techniques. If their properties are not adeguate, it's difficult to impove them even if it is possible.
- The loads can be defined only making some assumptions:
  - on the natural state of stress
  - on how our escavation changes the loads

  Infact the construction process influences the acting loads (both values and direction of application). So loads are not clearly defined but only assumed.

PRINCIPLE OF UNDERGROUND EXAVATION

The underground constructions are done by subtracting material from the under ground in order to create an hole. The material removed is already stressed.
We excavate tunnels in an already stressed body.
There is not only a single correct design of a tunnel, since we have to face to geological nature that is variable. What is a correct solution? A safe and a stable one.
We have to minimize the disturbance or the dangerous of nearby structures to minimize costs and construction time, to guarantee the health and the safety of the workers.
As we said, we escavate tunnels in an already stressed body. The state of stress is vertical and horizontal. The state of stress is expressed by 3 components:



- σ_v → VERTICAL STRESS → assumption: σ_v is lytostatic (function of only depth z)
- σ_{h1}, σ_{h2} → $\sigma_{h1} = \sigma_{h2} = \sigma_h$ → assumption that the horizontal components are the same

with:

VERTICAL STRESS: $\sigma_v = \gamma \cdot z$

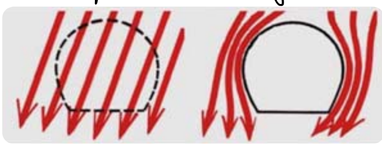
HORIZONTAL STRESS: $\sigma_h = K_0 \cdot \sigma_v$

$\sigma_h = \gamma \cdot z \Leftarrow K_0 = 1$ — STATE OF STRESS LYTHOSTATIC

$K_0 = \frac{\nu}{(1-\nu)} \cong 0.5$

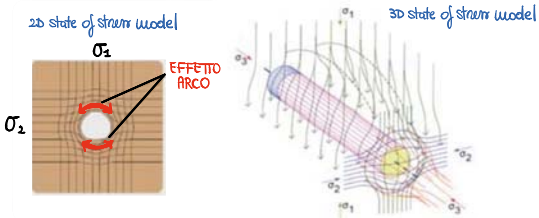
The evaluation of the natural state of stress underground is very difficult. Moreover when we excavate a tunnel we induce a redistribution of the state of stress around the tunnel. Excavation influences the previous state of stress of the ground (inside the rock mass):

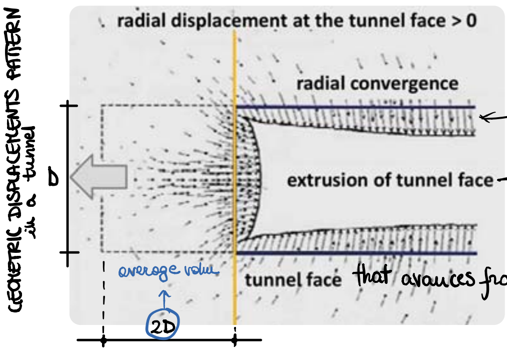
natural stress distribution before tunnel construction →



← redistribution of the natural stress inside the rock mass due to excavation of the tunnel

Because of the 3D nature of the problem, the real state of stress in front of the tunnel has a 3D geometry (La galleria influenza in maniera tridimensionale lo stato di sforzo).
For semplicity: we consider the state of stress bidimensional (2D) far from the face while 3D closer to the face.





radial displacements

extrusion of tunnel face → must be controlled → if we control the extrusion we will limit the influence out of the face. If we use a technological device able to limit extrusion we'll minimize these radial displacements
↓
This is the philosophy of the TBM

tunnel face that avances from right to left

GROUND DISTURB IN FRONT OF THE TUNNEL FACE (about 2 diameters) → 3D issue

WE HAVE SUPPORTS TO CONTROL THE BEHAVIOUR OF THE TUNNEL

Le gallerie disturbano il terreno per almeno un paio di diametri (dipende ciò dal tipo di terreno).
For this reason we start to measure displacements from the face inside the tunnel we will not measure that occours on the face, because at 2D distance we have already some displacement before excavation.

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
- UNSTABLE CONDITION
- ROCK BURST
- SQUEEZING

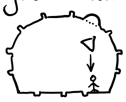
- **STABLE CONDITIONS**: in good rock mass under low natural stress state.
- **UNSTABLE CONDITIONS**: in case of fractured rock mass under low natural stress state.
- **ROCK BURST (colpo di tensione)**: in case of stiff and few fractured rock mass under high natural stress state → deep tunnels. A rock burst is a sudden release of elastic energy and gives rise a seismic effect, with the collapse of the tunnel.
- **SQUEEZING**: in the intermediate case. It consists of a layer of plastic displacements from few mm to 10 cm.

TUNNEL STABILITY

LOCAL STABILITY

Local detachment of fragments of rocks from the tunnel boundary

This doesn't affect globally the stability of the tunnel underground (il distacco di un singolo elemento non influenza la stabilità del sotterraneo). Anyway we don't want local detachments to guarantee workers safety.



Caving

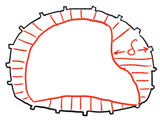
It is the detachment of some blocks around the tunnel that changes the geometry of the tunnel due to a not correct controlled underground, but without affecting globally the underground → "CHIMNEY CAVE"

↓

it can evolve to a total collapse

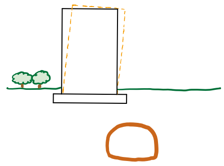
GLOBAL STABILITY

Displacements - squeezing



I have to much radial displacements not acceptable with my final design

Risk of settlements



When we excavate a tunnel, we could cause settlements from the surface

for example: if I excavate in basalt ground I don't need any support, while in ashes (ceneri vulcaniche) I need many supports.

ROCK — ROCK MASS — SOIL — MIXED PHASES

For a correct design, it is necessary the understanding of the **GROUND** behaviour and its answer to the tunneling operations, and the correct assessment of the position of the rock masses along the alignment. Generally we act in a context of great variability, of geological, geotechnical and hydrogeological conditions

⇒ **GREAT IMPORTANCE OF PRELIMINARY INVESTIGATIONS**

PARAMETERS THAT INFLUENCE THE STABILITY OF A TUNNEL

Tunneling projects can be classified as (referring to the cathegory of the ground)

(1) projects in SOFT GROUND (soil or weak rock masses) → gives stability problems, I have to stabilize the tunnel immediately
(2) projects in HARD GROUND (rock)

HARD ROCK MASSES
- basalt (basalto)
- granite (granito)
- gneiss

WEAK ROCK MASSES
- fractured hard rock
- tuff (tufo)
- limestone (calcare)
- marl (marne)
- gypsum (gesso)

Which are the parameters that influence the stability of a tunnel?

(1) ROCK MASS PROPERTIES
(2) NATURAL STATE OF STRESS
(3) SHAPE OF THE TUNNEL ⟶ concentration of stress
(4) SIZE OF THE TUNNEL SECTION ⟶ the distribution of stresses around the tunnel are bigger if we have small spans
(5) PRESENCE OF UNDERGROUND WATER
(6) CONSTRUCTION METHODS

When we make a design:

(1) I start with the design introducing some assumption → ON THE GEOLOGY; POSSIBLE PRESENCE OF WATER; ON STATE OF STRESS

(2) I start to excavate and build supports

What is the error between (1) (my design) and (2) (real life) and how can I determine it?
What's the way to tell that my design predictions (assumptions) are true?

⇓

MONITORING

it's the only tool that the designer has to check if his design assumptions were correct. Monitoring is carried out during excavation

← if the feedback is ok, I can keep excavating

→ if the feedback is bad I have to return back to my design and change it

OSS. The tunnel design and the tunnel construction are strictly linked: the link is monitoring

1. The most used monitoring system is the monitoring of displacements of the tunnel excavation boundaries, displacements of some specific points inside the rock mass, displacements of the surface.

2. Another monitoring system is the measurements of the stresses inside the supports.

3. Another monitoring system is the monitoring of the excavation machines behaviour (big datas given by machines ⇒ I have to understand them).

TUNNEL EXCAVATION METHODS

CONVENTIONAL METHOD

they are sequential procedures:
1) excavation of a certain portion of the tunnel
2) stop excavation
3) installation of support in the excavation zone

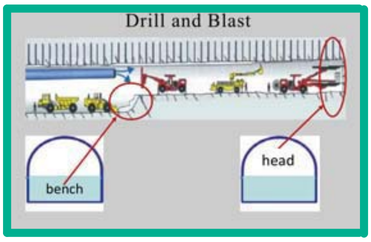
excavation can be carried out with:

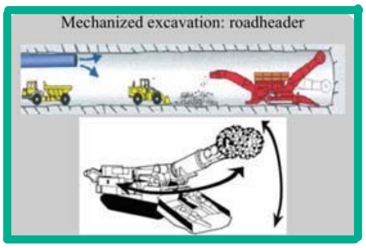
DRILL AND BLAST

drill = perforare
blast = fare esplodere } use of explosive in hard rocks

PUNCTUAL MACHINES

machines that excavate punctually to my tunnel face. (for example: roadheader = fresa puntuale). The machine demolite the rock, it creates the advancement in order to have free and sufficient space to install the supports.





FULL FACE MECHANIZED METHOD

they are machines designed in order to have:
- a cutter head (testa fresante) as big as the tunnel
- this cutter head carries the tools that demolish the rock continuosly

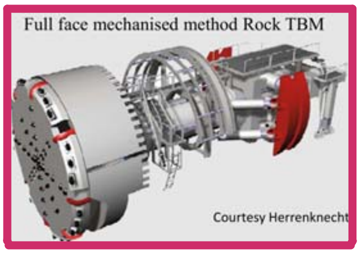
ROCK TBM

in nearly stable rock masses

SHIELD (scudo)

machines able to escavate the soil:
- cutter head that avances
- shield → installs the supports

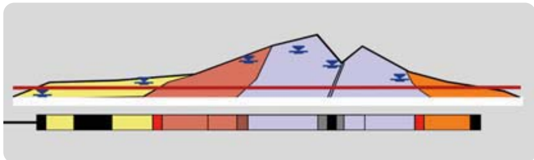
[Using a full face mechanized method brings to continuos excavation]



2. GENERAL ASPECTS OF TUNNELING SUPPORTS AND AUXILIARY METHODS

CONCEPT OF DESIGNED BASED ON RISK ASSESSMENT

An underground construction (tunnel) runs across different kind of ground, and different water level.



The design of underground structures depends on

Existing conditions
- Ground
- Hydrogeology
- Morphology → shape of the coverage above the tunnel
- Environment

For each tunnel section and for each construction phase (including also the final one) we have to:

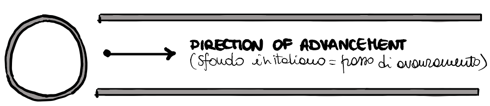
phase → SHORT-TERM (during the construction)
→ LONG-TERM (end of life)

- identify (capire quali sono) and estimate (quantizzarli) all hazards
- chose and evaluate remedial measures

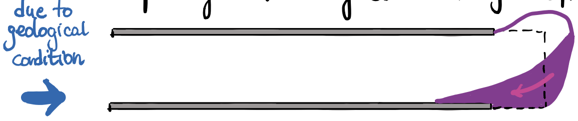
The key questions to be answered by the engineers and designers are:
* What is the goal for risk reduction? → ENGINEERING DESIGN
* How to achieve this goal? → THROUGH TECHNOLOGY

EXAMPLE

↳ TUNNEL FULL FACE ADVANCEMENT

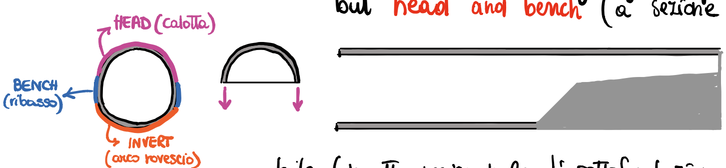


hazard due to geological condition: collapse of the tunnel face and the free span



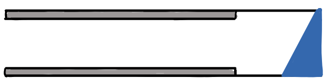
The question is now: Can I accept this hazard? If I don't wanna accept this risk, I have to do something.

POSSIBLE MEASURE → I will reduce the height of the face, so I'll not excavate full face but head and bench (a sezione parzializzata)

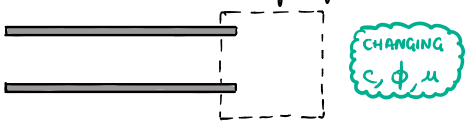


→ But now I have a new hazard: the instability of the footings of the support → I can build micro-pile (metto micropalo di sottofondazione) to guarantee the stability of the footings.

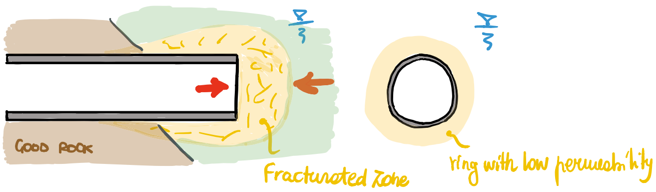
POSSIBLE MEASURE → use of shielded full face TBM with face pressure



POSSIBLE MEASURE → use of auxiliary measures ahead of the face to improve the stability condition and allow a full face excavation. Changing the properties of the soils, we get better behaviour avoiding stability problems.



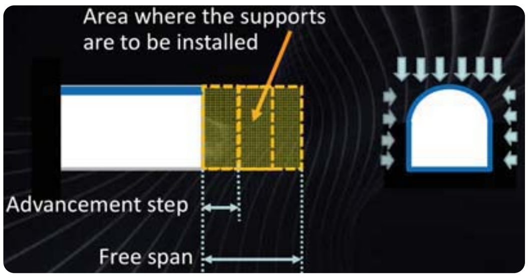
ANOTHER PROBLEM → could be the water table over the tunnel: inrush water (piovuta d'acqua)
Stability
⇓
I need to stabilize the tunnel. How? Grouting in a fault zone below water table: inject water cement mix around the tunnel to work in a stable face and without the inrushing risk. The injection fills all voids in the soil around the tunnel ⇒ permeability of the soil ↓



SELF SUPPORTING TIME AND FREE SPAN

- **SELF SUPPORTING TIME** = TEMPO DI AUTOPORTANZA → time that certain length remains stable
  - ↳ it can be also $t = \infty$
  - ↳ worst is the rock mass quality, less self supporting time will be
- **FREE SPAN** = LUCE LIBERA → maximum length can remain unsupported for a certain time

these two concepts are linked together: one has not meaning without the other.



The tunnel should remain stable with a certain advancement till the support installation. The support is used in order to guarantee the long term stability.

↑ Rock mass quality → ↑ free span, ↑ self supporting time

Self supporting time and free span are strictly linked to rock mass quality

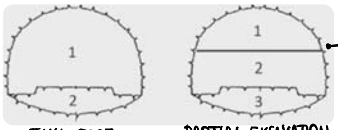
If self supporting time and free span are too short for tunnelling activity or the face is unstable → UNACCEPTABLE HAZARD

Which remedial measures?
With which properties?
When should them be installed?

→ **POSSIBLE REMEDIAL MEASURES**

1. to reduce the excavation section
2. to apply a pressure against the face
3. to improve the ground properties
4. to use pre-supports ahead the face → [for example steel elements]

1) TO REDUCE THE EXCAVATION SECTION INTO SMALLER PORTIONS (smaller sections increase stability)

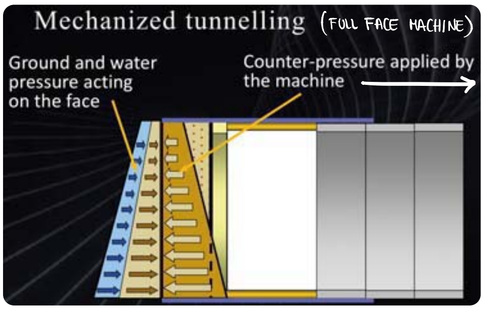


→ excavation in 3 steps

FULL FACE

PARTIAL EXCAVATION (Top heading / Bench / invert)

2) TO APPLY A FACE PREASSURE



to counterbalance ground and water pressure. It could be done also in conventional tunneling (to apply a face pressure) by using reinforcement on the face (fiber glass elements)

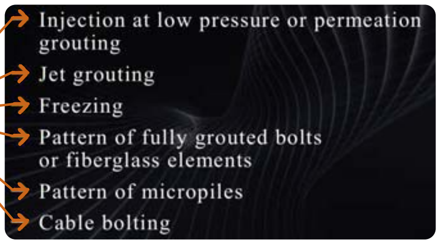
3) TO IMPROVE THE GROUND PROPERTIES

Methods that improve (from the engineering point of view) the mechanical or hydraulic properties of the rock mass.

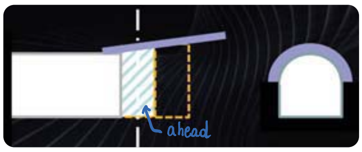
How?

- injecting fluids or freezing the fluids already present in the ground
- inserting structural elements with one dimension prevalent that will improve the quality of the rock

List of technology used →



4) TO PRE-SUPPORT THE EXCAVATION



The presupports are methods which use the insertion, in the rock mass, of structural elements ahead the tunnel face with the purpose to create a pre-support before the excavation. The presupport can be also obtained with the use of a pattern of the improvement techniques.

List of technology used:
- mechanical precut
- forepoling or steel pipe umbrella
- arch of microtunnels
- jet-grouting arch with or without reinforcement
- injection (permeation grouting) around the tunnel
- consolidation of the tunnel profile (fiber glass elements)

CONVERGENCE CONFINEMENT METHOD

INTUITIVE INTRODUCTION

What is convergence confinement?
If we consider a medium with a stress inside, and we want to evaluate the displacements (or the strain ε) in this medium, what should we have to do?
↳ I must solve some differential equations:
(if we're working in an elastic medium)

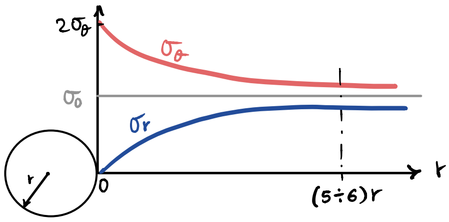
- EQUILIBRIUM EQUATIONS
- COMPATIBILITY EQUATIONS
- COSTITUTIVE LAW

If I'm not in an elastic medium (as a rock mass, as soil) there is one more set of equations:

- THE YELD BEHAVIOUR

(curva di resistenza (es. Tresca / Mohr-Coulomb))

Going to tunnels:



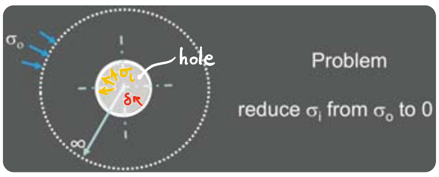
HYPOTHESIS:
(a) elastic medium
(b) hydrostatic state of stress

STRESS DISTRIBUTION IN AN ELASTIC MEDIUM AROUND THE HOLE, WITH AN HYDROSTATIC STATE OF STRESS

If I want to solve these set of equations with numerical methods (FEM), I have to make a thick mesh close to the hole, then (after $(5\div6)r$) it could be more rough.

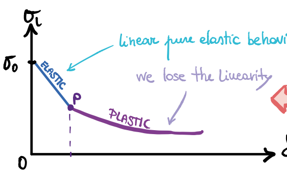
Closed form approach (solution)

Let's consider now a hole in a plate subjected to a radial pressure inside.

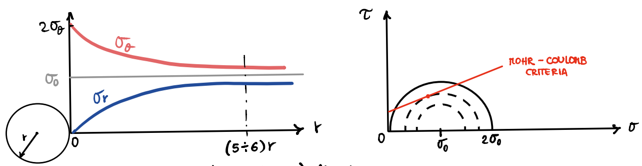


HYPOTHESIS:
- homogeneous and isotropic material
- hydrostatic state of stress
- axialsimmetric geometry

We want to obtain the radial deformation of this hole depending on the internal stress inside the hole.

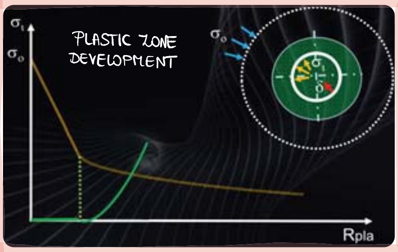


CONVERGENCE - CONFINEMENT CURVE → Reducing the internal pressure provokes a closing of the hole



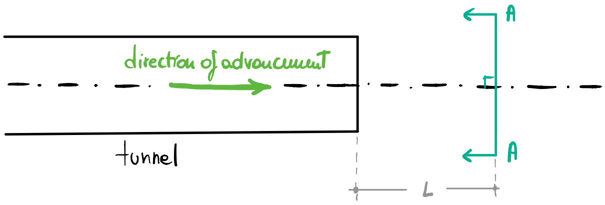
If I reduce the radial internal pressure, I'll have an increase of the tangential stress ⇒ increase of the Mohr circle. Reducing the internal pressure, that means to reduce σ_r, the circle will touch the failure curve (riducendo la pressione interna ad un certo punto il cerchio di Mohr va a toccare l'inviluppo di resistenza). In this moment we have a change in behaviour: we lose elasticity and plasticity takes place (la roccia dunque si plasticizza)

N.B.



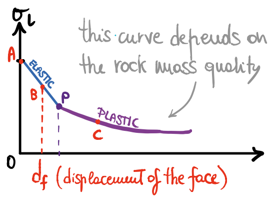
The more I break the rock the more it expands ('cause ↑ voids). Around my tunnel there is a creation of a green zone that is an area where the rocks start to be plasticize (broken rocks). These broken rocks have to controlled in the design of our support. These broken rocks are brought by supports.

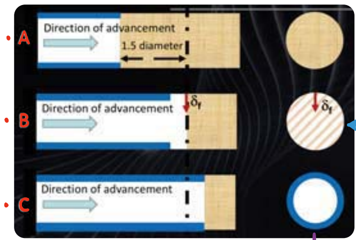
Come back to our tunnel. What does happen in cross-section A-A?
↳ it is a generic cross-section under studying



When the tunnel face is very far from the cross section A-A I'm in the elastic field. I'll start to experience the radial displacement ahead the tunnel face at 1.5 ÷ 2D = L (I'm starting to move along elastic curve ⇒ ↑ δ).

Then when I'll arrive with the face in the cross-section A-A, I could be before point P or after, it depends by the rock mass quality.

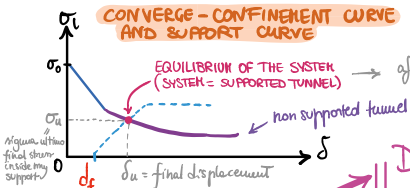


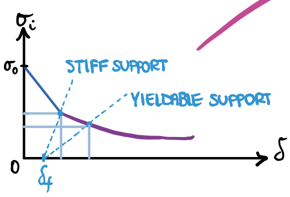


only when I arrive here I can install supports 'cause I create an hole → from this time the support curve exist!

radial ring = support (radial spring)

The ring (radial spring support) reacts to deformations imposed by the ground around. If I try to deformate this ring, it will react with a force. The two systems go in equilibrium.





Depending on the support stiffness, I could have two limit situation

- If I have a very stiff support (rigid) I'll have few deformations but a lot of stress in my tunnel.
- If I have a very yieldable support I'll have larger deformation and less stress

This is very important in the excavation of deep tunnels. In this kind of excavation I have to excavate a bigger hole than the one needed and let the tunnel close itself 'cause we have high deformation in order to reduce the high stress acting on the tunnel.

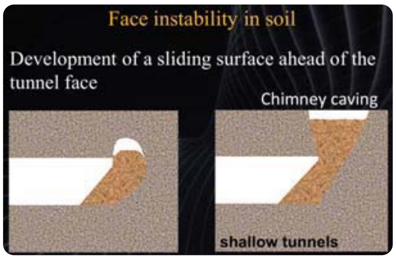
In una galleria che ha grandi deformazioni (o grande profondità), se cerco di fermare la deformazione applicando rivestimenti molto robusti avrò sforzi che non potrò sopportare né col CLS né con l'acciaio. In queste condizioni dunque devo lasciare che la struttura del tunnel si deformi accettando anche grandi deformazioni col vantaggio di ridurre gli sforzi agenti sui rivestimenti. Deformazioni più ⇒ buco più grande per rispettare i limiti geometrici a deformazione scavata

3. TUNNEL FACE STABILITY DESIGN

The tunnel face stability cannot be studied by an axialsimmetrical model because the stress distribution is not axialsimmetrical in front of the face. It should be studied by a numerical 3D analysis. The possibility to analize the stability of the face of a tunnel can be done by:

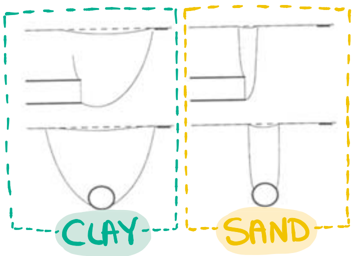
- **NUMERICAL 3D CALCULATIONS** (3D analysis)
- **ANALYTICAL CALCULATIONS** (based on the LIMIT EQUILIBRIUM MODEL with different geometries of the sliding body)

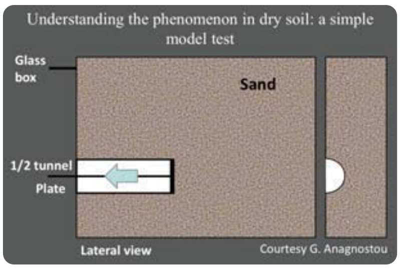
The instability problem ahead of the face (in soil) is the creation of a sliding surface that create the SINKHOLE above the tunnel ('sinkhole' = 'fornello' → se raggiunge la superficie).

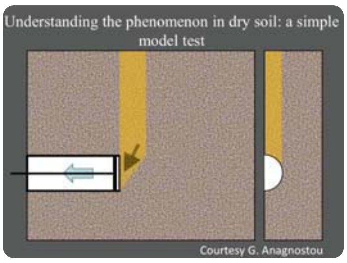


The mechanism of instability is function of the kind of the ground:
- COHESIVE MATERIAL → clay → huge settlements
- UNCOHESIVE MATERIAL → sand

↓ big different in the shape of the sinkhole depending on ground nature





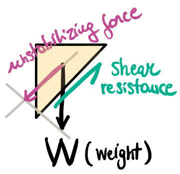


To make the evaluation of the stability condition of a tunnel face we can work with ANALYTICAL METHODS. As it said before, these methods are based on LIMIT EQUILIBRIUM ANALYSIS.

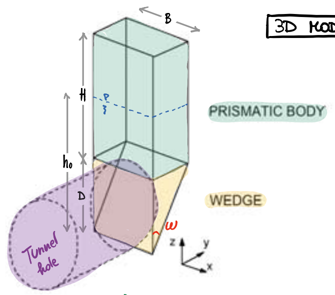
LIMIT EQUILIBRIUM CONDITION

$$\frac{\text{unstabilizing force}}{\text{stabilizing force}} \begin{cases} > 1 & \text{UNSTABLE EQUILIBRIUM} \\ < 1 & \text{STABLE EQUILIBRIUM} \end{cases}$$

if we have an element and we want to verify if it is stable on a sliding surface (for example), we will compare the unstabilizing force (stress) with the stabilizing one.



ANALYTICAL METHOD ~ Anagnostou & Kovari



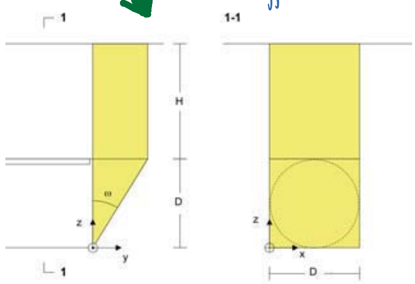
3D MODEL → Computational model of the failure mechanism. It is based on osservation in situ and by models.

HYPOTHESIS:

* homogeneous and isotropic ground
* in front of the tunnel face there is a sliding wedge
* above the wedge there is an upper body (PRISM) that applies forces on this wedge
* Mohr - Coulomb yielding criteria on the sliding surface

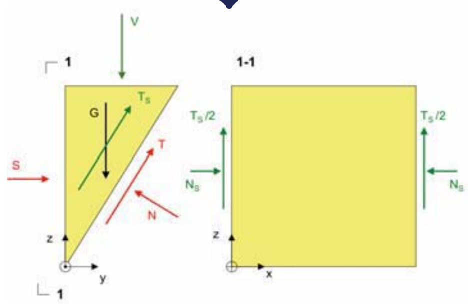
→ The base hypothesis is to make an equilibrium analysis of this geometry

In different countries there are different schemes (only in size & geometry) but all of them have in common the LIMIT EQUILIBRIUM ANALYSIS.



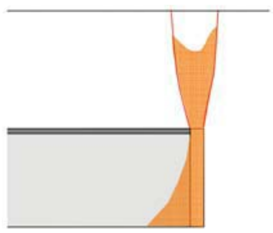
The problem is to determine the value of ω, in other words it is to determine the size of the wedge.

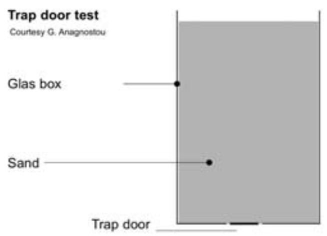
If I make an analysis of the forces acting on the wedge I get this

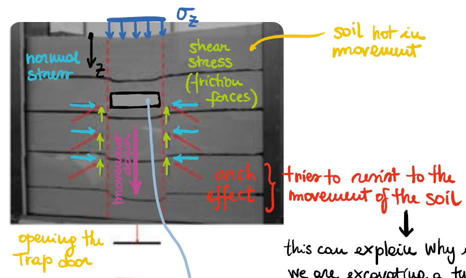


V = load that the prism applies on the wedge
G = weight of the wedge
T, N = shear and normal forces acting on the sliding surface
Ts = lateral resistance force

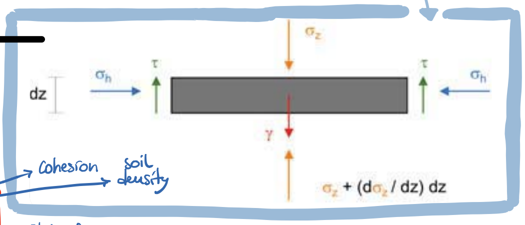
The first question is: which is the vertical force V that the prism applies on the wedge?
↳ the evaluation of this force is based on the SILO THEORY by Horn (HORN THEORY)







this can explain why if we are excavating a tunnel 1000 m deep, there is not all the 1000 m soil (up) acting on my tunnel, as soon as it will never possible to excavate (force not acceptable 'cause too heavy). When I have a very deep tunnel, due to arch effect there is only a portion of soil that it is really acting on my structure.

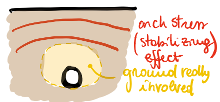


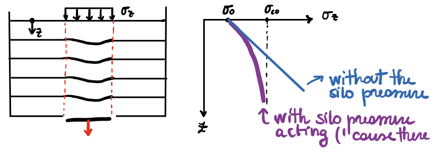
I can make a differential analysis of the equilibrium of this slice.
What I have to remember for the exam is:
(solving the equilibrium equation we obtain)

$$\sigma_z = f(z, R, \lambda, \tan\phi, c, \gamma, \sigma_0)$$

- σ_z → VERTICAL LOAD
- z → DEPTH
- R → SIZE (ratio that gives the size of the opening)
- λ → RATIO between horizontal and vertical forces (loads)
- tan φ → friction of soil
- c → cohesion
- γ → soil density
- σ_0 → state of stress on the surface

→ $\int \sigma_z \, dz = V$





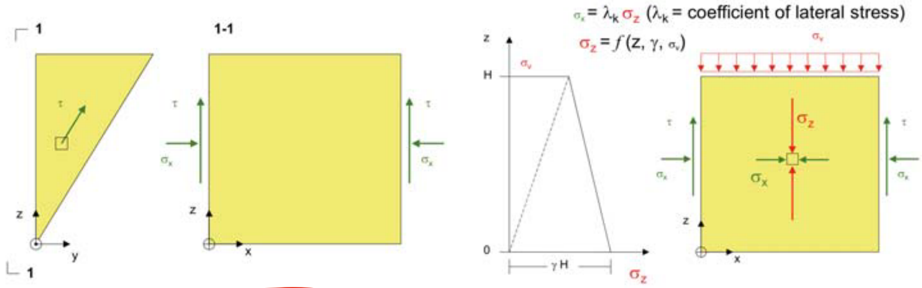
with the silo theory there is a drop of the σ_z (vertical sigma) that goes to an asintoto with the depth. Silo pressure doesn't increase anymore with the depth!

By solving the equilibrium equation, it's possible to obtain (by integration of the equation for all the slices) the value of V.
When we are tunneling with very low OVERBORDEN (ricoprimento) ($\lessapprox 1D$) this arch effect can't take place, so I have to take into account the total load.
The more depth I go, the more probability that this arch effects takes place reducing the loads.

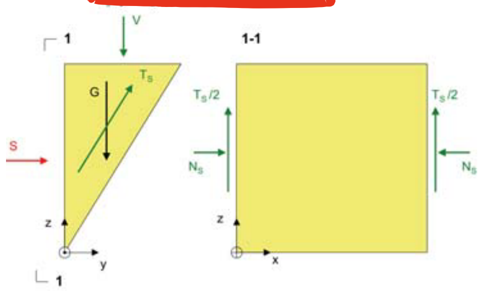
evaluation of Ts:

$\tau = c + \sigma_x \tan\phi$

$\tau = c + \sigma_x \tan\phi$
$\sigma_x = \lambda_k \sigma_z$ (λ_k = coefficient of lateral stress)
$\sigma_z = f(z, \gamma, \sigma_v)$



$\tau = c + \lambda_k \tan\phi \; f(z, \gamma, \sigma_v)$

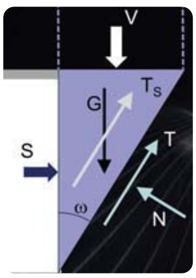


Ts is obtained by integration of τ on lateral surface

$$T_S = \int_S \tau \, dS$$

Now that we have these two parameters: $\sigma_z \rightarrow V$; $\tau \rightarrow T_S$

we can make the LIMIT EQUILIBRIUM OF THE WEDGE:



parameters that take place in equilibrium:
- V → already defined by SILO THEORY
- Ts → known
- G → known
- S, N, T → 3 unknowns: N, T unknowns on the sliding surface; S is our goal

→ the system is function of ω in fact the wedge size depends by ω.

We have 3 unknowns → so we have to write 3 equations to solve the problem:

(↗) EQUILIBRIUM PARALLEL TO THE SLIDING SURFACE

S, T, T_s, V, G

(↖) EQUILIBRIUM ORTHOGONAL TO THE SLIDING SURFACE

S, N, V, G

COULOMB EQUATION ON THE SLIDING SURFACE (Coulomb condition)

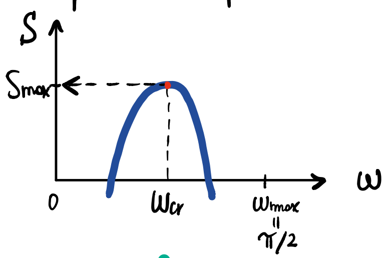
T, N

the solution of this system is:

$$S = f(\omega, D, \phi, c, G, V, T_s)$$

- S → SUPPORT FORCE
- D → tunnel diameter
- ϕ → friction angle
- c → cohesion
- G → wedge weight

It is possible to plot the SUPPORT FORCE S in function of ω:



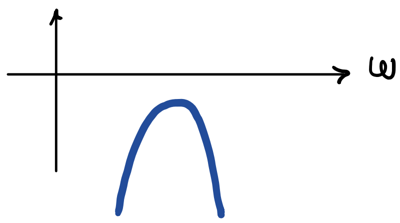
$S = f(\omega)$ → the diagram has an inverted bell shape (dry soil)

S_{max} = maximum force (evaluated using the limit equilibrium method) that I need to apply to give the safety factor of 1 ($SF = 1$) of this system (force I have to apply to the face to stabilize it in the equilibrium condition).

→ it is for example the pressure that a machine should apply to allow to excavate a tunnel without creating sinkhole in an urban area (pressure to guarantee no collapse).

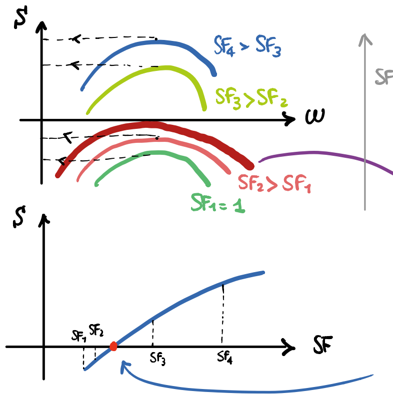
If the S I've calculated is negative ($S<0$), what does it mean physically?

↳ It means that I have to pull the wedge in order to make it collapsing. So this means that the tunnel face is stable: I have enough c, or ϕ to guarantee stability without any pressure to apply.



$S < 0$ → Tunnel face is stable

EVALUATION OF SAFETY FACTOR OF A STABLE FACE



$S = f(\omega, D, \phi, c, G, V, T_s)$

I should apply the safety factor to the: $\tan\phi$, c

$\frac{\tan\phi}{SF}$; $\frac{c}{SF}$

it corresponds to the tangent curve (S, ω)

safety factor of my tunnel face (wedge), without applying any force S!

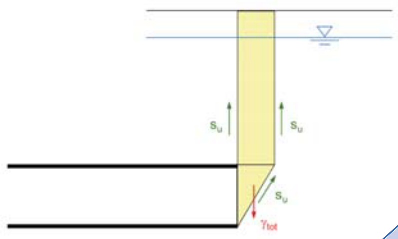
What we have discussed till now regards conditions without water pressure. Now let's consider also this case.

ANALYSIS OF FACE STABILITY BELOW WATER TABLE

Short-term stability of a low-permeability ground

Total stress analysis

GEOTECHNICAL HYPOTHESIS
- Undrained shear strength s_u ($\phi_u = 0$)
- Total unit weight γ_{tot}

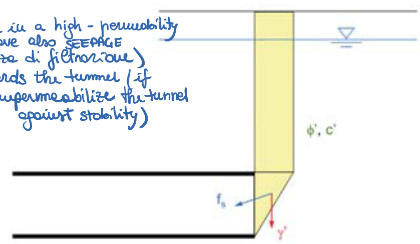


Long-term stability or high-permeability ground

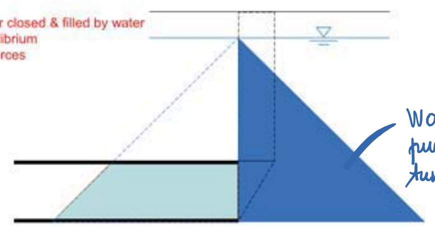
Effective stress analysis

GEOTECHNICAL HYPOTHESIS
- Effective shear strength parameters ϕ', c'
- Submerged unit weight γ'
- Seepage force f_s (depending on the hydraulic conditions)

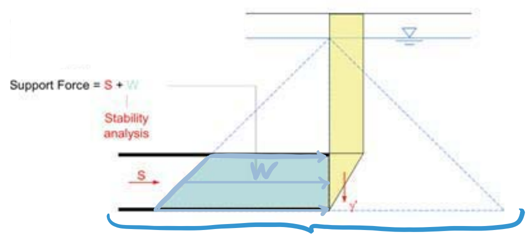
Since we are in a high-permeability ground we have also SEEPAGE FORCE f_s (forza di filtrazione) moving towards the tunnel (if we don't impermeabilize the tunnel face they are against stability)



Working chamber closed & filled by water
⇒ hydraulic equilibrium
⇒ no seepage forces



Water pressure pushing against tunnel face



If I want to eliminate seepage forces, I need to be able to have on the tunnel face a system that gives the hydraulic equilibrium. I have to invent something that applies a pressure that counterbalance water pressure. This is done by the full face machine. In conventional tunneling this is not possible, but I have to change permeability of the soil.

{ If I don't wanna apply a pressure face against water W, I have to change soil permeability property. }

So at the end I need to apply the water pressure W plus the support force S seen before. As consequence:

TOTAL SUPPORT FORCE = S + W

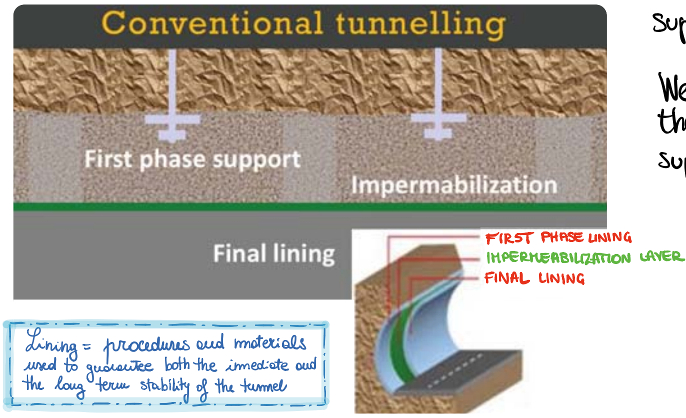
calculated in effective stress analysis

Dunque se lavoriamo con le macchine sottofalda, dobbiamo avere una macchina capace di applicare la pressione dell'acqua più la pressione che serve per stabilizzare il terreno. Se non applicami la pressione dell'acqua avrei delle forze di infiltrazione (verso il fronte) instabilizzanti.

The evolution of the condition of the behaviour of the tunnel under seepage forces is very difficult. Anagnostou developed a reserch using numerical analysis to try to evaluate these values. It's not a perfect equilibrium. He produces nomograms to get the value of the stabilizing force.

4. TUNNEL EXCAVATION METHOD

SUPPORTS USED IN CONVENTIONAL TUNNELING



Lining = procedures and materials used to guarantee both the immediate and the long term stability of the tunnel

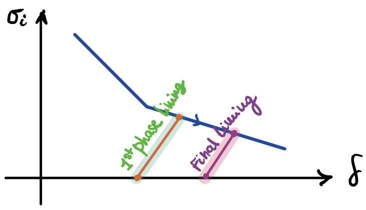
Support or Lining = sostegno o rivestimento

We divide the supports to be applied in the free span in 2 different types of support:

* FIRST PHASE LINING SUPPORT
  it is the support installed immidiately in the free span during the tunnel excavation to guarantee the stability. The goal of this lining is to stabilize temporarly the excavation till the new lining, the final one will be build. We make the hypothesis that in long term this lining disappear.

* FINAL LINING SUPPORT
  it is what we see when we drive into a conventional tunnel. It is the concrete casted in place. It can be reinforced or not: it depends on the stress level.

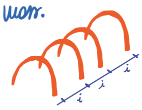
Till the first phase lining exists, Has got the final lining stress inside? (Fino a quando esiste il rivestimento di prima fase, il rivestimento finale ha carico o no?)



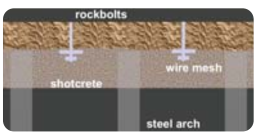
What is the answer? No, 'cause all the loads are supported by the first phase lining. Nella progettazione facciamo un'ipotesi semplificativa: il rivestimento di prima fase sparisce durante la vita utile della galleria. Why do we make this hypothesis? 'Cause there is no protection against corrosion of the first phase lining (in steel). So we are not sure what it will happen in long term. Between the two lining support there is an impermeabilization layer to controll the water table if we are below water table and to controll the dripping (gocciolio).

Which are the most used elements for first fase supports?

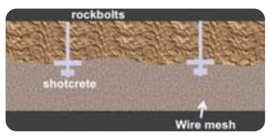
(1) ROCK BOLTS (bulloni) → elements installed inside the rock to stabilize unstable portion of rock mass.
(2) STEEL RIBS / ARCH (Rib = costola / centina) → interax between 2 steel arches i ≈ 0.5 ÷ 2 m



(3) MESH (rete elettrosaldata)
(4) SHOTCRETE (calcestruzzo proiettato) → layer of concrete applied against the rock by spraying it with compress air. The reinforcement of this shotcrete is done by wire mesh or metallic/plastic fibers.

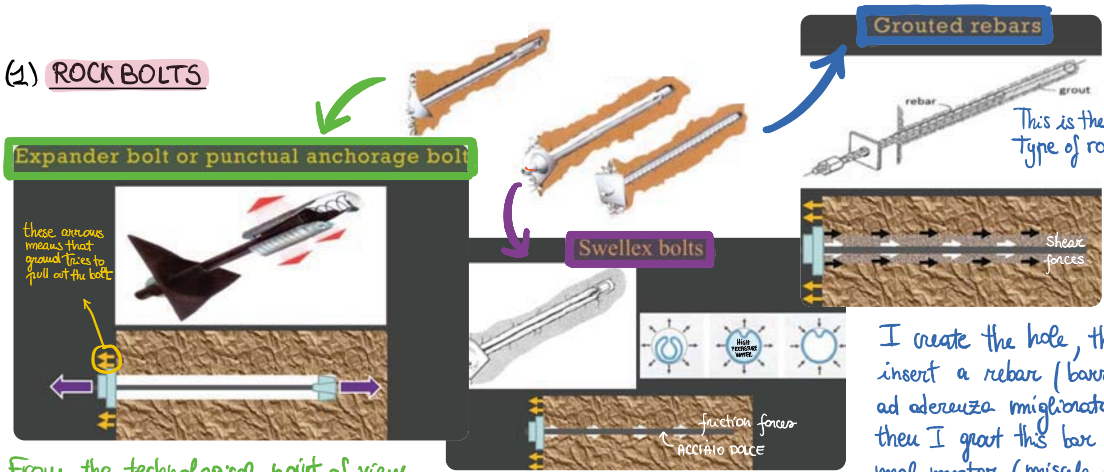


Used combo:
- STEEL ARCHES + SHOTCRETE
- ROCK BOLTS + STEEL ARCHES + SHOTCRETE
- ROCK BOLTS + SHOTCRETE



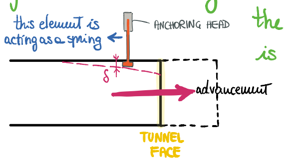
Depending on the quality of the rock the steel arch couldn't be needed, so only rock bolts and shotcrete are sufficient.
In some tunnels, mainly when we're working in soil (so there is a pre-reinforcement outside the tunnel), we haven't rock-bolts.

(1) ROCK BOLTS



This is the most used type of rockbolts

From the technological point of view, this element is made of an anchorage shell (conchiglia di ancoraggio o punto di ancoraggio) that is mechanically connected with the rock. Then we have a free bar and then a plate to fix the bolt at the surface of the tunnel. The installation technology is: • drilling (the hole must remain opened), • insertion of the bolt, • anchoring of the bolt, • closing with the nut (bullone) the connection face. It is important to highlight that I can pretensioned the bolts. The pretensioning of these elements will allow to apply immediately a pressure on the tunnel face, even if the tunnel face is not moving. During the advancement the original stress in the bolt is growing up 'cause some displacement will occur (δ).



The connection between the reinforcing element and the rock it is just done only by friction. It is empty. So if I apply inside the bolt high pressure water, the bolt will deformate. This deformation will let to born a contact force between steel and rock mass that will induce some shear resistance on the boundary. Which is the problem of this element? The steel is not protected against corrosion. For this reason this element cannot be considered as a permanent support. It is just a temporary support.

I create the hole, then I insert a rebar (barra filettata ad aderenza migliorata) and then I grout this bar with normal mortar (miscela H_2O cemento), or sometimes with chemical resines (much higher performance than mortar but too expensive). How does it work? This element is not able to apply a pressure (any confining action) against the face till the face starts to move. When the face starts to move and acts trying to pull out the bolt, I see that there are frictional forces on the boundary of the grout / steel. The bolt start to react against the yellow arrows, with a pressure. Questi elementi potrebbero essere considerati come elementi permanenti perchè l'acciaio è protetto dal getto contro la corrosione, ma non c'è nessuna garanzia che il foro sia perfettamente cementato. Se la barra è protetta da un elemento in plastica allora sì.

HOW DO THEY WORK ?

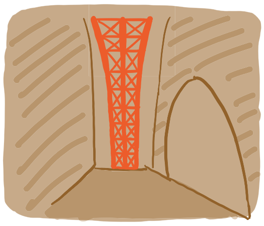
Rock bolts in rock masses are considered as elements used keeping in place unstable elements on the boundary of the excavation.

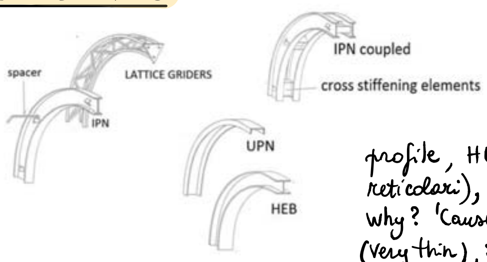
Reinforcement of rock elements free to fall under it's own weight — Reinforcement of unstable layer of horizontal beddings

Normally the calculations of the bolts in these conditions are done using LEM (limit equilibrium method)

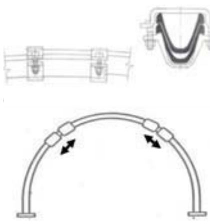
N.B. I usually install rockbolts not only in the zone where there is a wedge, but I place them using a regular pattern. Why? 'Cause we are not really aware where the irregular distribution of the joints will create the wedge. So we make the geotechnical evaluation of the maximum wedge has to be supported, and then we install the rockbolts with a regular pattern.

(2) STEEL ARCHES OR STEEL RIBS

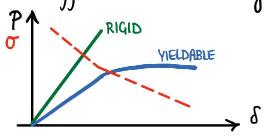


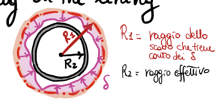


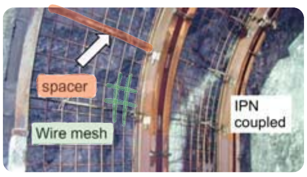
Steel arches are elements folded (piegati o calandrate) to get a cert curvature radius that will fit with the theoretical shape of our tunnel. They can be IPN profile, HEB profile, or LATTICE GRIDERS (centine reticolari), UPN profile. The most used is the IPN, why? 'Cause I can have a small wing (ala) (very thin), so the sprayed concrete will be well connected. Lattice griders are good too. Due to this shape, when I use shotcrete I will have a good connection between steel and concrete. The only problem of lattice grider is the cost.
The calandra lock (calandratrice) that is the mechanical device that folds the steel arches (for bending the steel ribs) has some limits regarding the max size of the steel profile. The max height that have been bent is between 240 mm ÷ 280 mm. For this reason often two coupled elements are used (more bending stiffness resistance).



Then there is one more kind of arch: the T-H arch (Toussaint-Heinzmann yielding steel ribs). Invented and used for coal mines. They are used for yieldable supports (supporti cedevoli). If I make a diagram of the (P, δ) of the support I can notice the difference with the rigid supports. Looking at the blue curve, we notice that at a constant pressure P we get an increase in displacement (deformation). The yieldable elements can be used when we have very deep tunnel in swelling rocks (in this case the tunnel tries to close itself). In very deep tunnel in squeezing rocks (where the deformation is important), if I try to stop the displacement of the boundary using very rigid element, I get very high value of pressure that normal structures cannot afford. So the idea is to try to use a more deformable element in order to have less pressure acting on the lining but accepting more deformation. ⇒ I have to take into account this fact in design: I excavate the tunnel with a bigger radius allowing the closing of the tunnel itself.







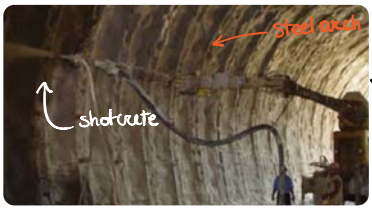
SPACER = steel bars to guarantee the correct position of steel arches. They also give longitudinal connection between steel arches.

The installation of steel ribs is done in free span → N.B. No risk for workers!
(free span: risky operation)

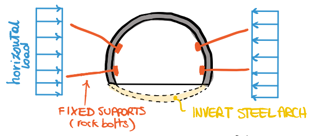
(4) SHOTCRETE

→ weigth = 30 ÷ 40 kg/m³

Shotcrete is a technology widely used in tunneling because it can be easly and fast applied in tunnel itself. The idea is to spray the concrete mix against the rock/soil, using compress air. Primary important property of the concrete mix is: to be sticky (deve avere capacità di adesione elevata in quanto una volta spruzzato non deve cadere giù). Secondly it has to have



a fast hardening to advance faster. Normally for a correct design the steel arch should be covered by the shotcrete layer. ← Here the shotcrete is not acting as a supporting shell (non come un guscio di sostegno), but it is just acting to prevent the detachment of rock/soil elements between two steel arches, without a structural effect. To guarantee a structural effect of a shotcrete layer I need to have an homogeneous covering of the steel arches.

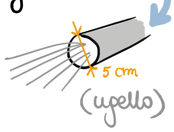


I could put some rockbolts on the steel arches to garantee much stiffness. Why do I do that? 'Cause my design assumption were wrong, so in the site I get bigger displacements than the ones I had computed in design (I see it by monitoring). I could alternatively close the arch (obtaining a ring) immediately after excavation → INVERT STEEL ARCH.

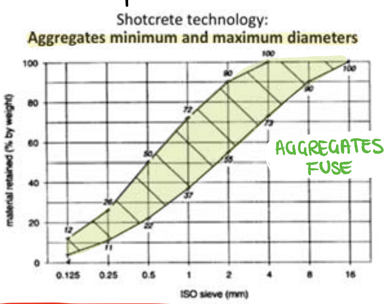
What is shotcrete made of?

Cement (42.5)	350 - 500 kg/m³
Aggregate (0-8)	~ 1700 kg/m³
Microsilica ⤷ 1mm	5 - 10 %
Accelerator	4 - 6 %
W/C ratio (water/cement)	0.4 - 0.55
Max. coarse aggr.	40 %
Plasticizer	5 kg/m³

It is similar to normal concrete, but with a different aggregate distribution. The maximum grain size is: $d_g \leq 8$ mm. Why? 'Cause it has to be sprayed against rock mass using a nozzle



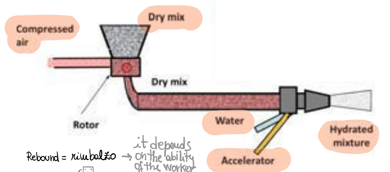
ACCELERATOR (normally is a SILICATE) makes our concrete sticky and quick hardening. The problem of silicate is that reduce the long term resistance. Too much silicate will reduce the long term resistance of the concrete, and could give some environmental problem (carbonatation). Today ALLUMINIUM BASED ACCELERATORS are used to avoid environmental issues. It is important to write in the technical specifications which type of accelerator I want to use.

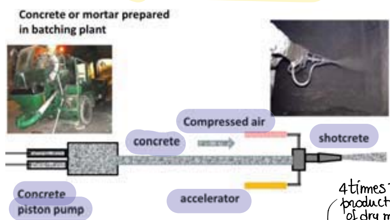


There are two technology for shotcrete application

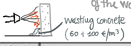
DRY-MIX METHOD

WET-MIX METHOD





Rebound = rimbalzo → it depends on the ability of the worker



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 5m³/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

Factor	Wet mix
Equipment	Less equipment at the jobsite then dry mix Usually a sprying robot is used Less wear rate in pump, hoses and nozzle Up to 60% less air consumption
Mixing	Accurate mixing at mixing/batching plant. Wet sand acceptable.
Output	Higher than similar dry machines: 2-10m³/h with hand-held nozzle, up to 20m³/h with a sprying robot
Rebound	Low rebound with correct mix, could be less than 10%.
Quality	More difficul to obtain high strenght More homogeneous quality
Impact velocity	General adquate for tunnel/mining work
Additives	Generally as liquid.
Dust	dust formed less than dry mix

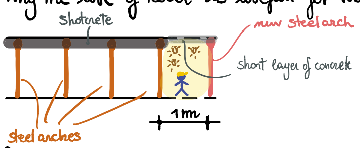
4 times the production of dry mix (Equipment)

water/cement ratio must be higher than the previous one (Mixing)

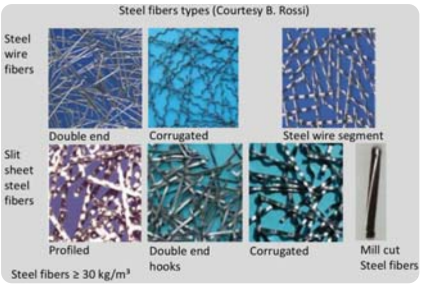
In the dry-mix method, we put a dry-mix of concrete and aggregates in a basket. The compressed air pushes the dry-mix through the nozzle. At the nozzle it is added water and accelerator. So we obtain an hydrated mixture that goes against the rock. This technology is done by using small machines. The problem is that for a short time we have a lot of dust because not all the grains of cement have been covered by water. So there is not a real complete mix between water aggregates and cement. The consequence of that is the creation of a lot of dust → problem for the workers. The advantages are: small machine, the use of premixed material also in bags (premiscelati in sacchi facili per il trasporto)

With the wet-mix design, we have concrete already prepared in a batching plant (impianto di betonaggio), pumped in a piston pump till the nozzle where we have the compressed air and the adding of accelerators. This is the techology usually used in Italy (it is the tech most used all around the world). It is anyway a dusty operation (but less than dry-mix method). The biggest advantage is that we can use a robot to spray shotcrete layer → no risk for workers

Why the use of robot is usefull for workers' safety? We send workers in an area (the yellow one) where small detachments can occour. We have to do that in order to install the next steel arch. Despite of an geotechnical design of the yellow area we cannot have 100% of security that small detachment cannot occour. So to reduce the risk, in some countries there is the tradition to create a thin layer of concrete (5 cm) on the boundary not for stability reason but to prevent these detachments. Però non tutti sono d'accordo, perchè se non applico bene il layer di cls ho pure il rischio che si stacchi pure questo layer. Today the trend is to clean the yellow boundary with the operation of SCALING = disgaggio → it is the operation that we do after the advancement to detach all possible instable elements on the boundary. The scaling is compulsory (obbligatorio) when we use DRILL & BLAST TECH.



Concrete "haven't tensile strength", so we have to install reinforcing elements:
- → I can install between 2 steel arches a layer of wire mesh (tra 2 centine metto uno strato di rete elettrosaldata), and then I spray the concrete. (10×10, 15×15)
  PROBLEM: I have to send workers inside the yellow area to install the mesh (risky operation)
- → I can add steel fibers to the wet-mix.

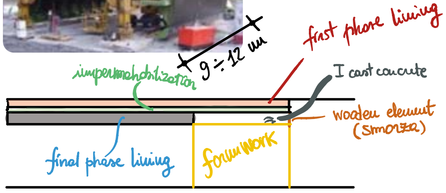


Today there is also the trend to use polypropilene fibers insted of steel ones. For the design of concrete with fiber reinforcement (only for steel) there are standars (codes).

The use of polypropilene fibers PP (very thin) for antispolling effect is very usefull. The antispolling effect is for fire protection. They are not so important in shotcrete but in segments (precast elements). In case of fire → in a tunnel T ≅ 1000 °C 'cause there is the oven effect. Spraying water to stop fire the concrete is subject to a huge ΔT. To prevent the spalling of concrete and to give a superficial resistance to the concrete against fire, the use of polypropiline fiber is very important, the mix design should be carefully studied 'cause these fibers can reduce the water content.

Final lining support

To cast in place the final lining we need the formwork



5. FOREPALING DESIGN

Pre-support measures involve spiling or grouted pipe arch canopies (or umbrella) that bridge over the unsupported excavation round (also named "unsupported length"). In this way it is possible to create a protective structure under which the excavation can be carried out in safe conditions, minimizing the surface settlement and tunnel radial movements.

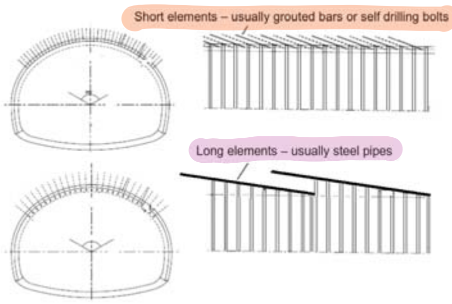
These mechanical pre-support elements are generally used to:
- increase stand-up time by preventing ground material from raveling into the tunnel opening causing potentially major over-break or tunnel instabilities;
- limit over-break;
- reduce the ground loads acting on the immediate tunnel face;
- reduce ground movements in the unsupported span and in the tunnel face and, consequently surface settlements.
- to increase the safety of the workers near the tunnel face.

3D view of steel pipe umbrella

This pre-reinforced techniques for tunnelling has been referred to in literature as: pipe roof umbrella - pipe roof support - steel pipe umbrella - umbrella arch method - pipe forepoling umbrella – forepoling - spiling - steel pipe canopy - lances

When the free span is very short one of the possible design motion is to change from one side the property of the rock or to presupport the tunnel boundary (creating some structural elements ahead of the face to be able to create a presupport able to counterbalance the trend of radial displacements can occour in the free span). Onother measure is to prereinforce the face in a way to give stability to the tunnel face.

The first technology is: the installation of STEEL PIPES UMBRELLA (ombrello di infilaggi) ahead of the face, or using ROCK BOLTS (steel pipes ≠ rock bolts. Steel pipes have big dimension than the rock bolts).



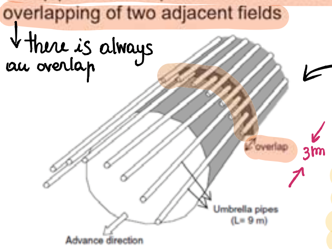
STEEL PIPE → THICKNESS → 0.6 ÷ 1 cm
STEEL PIPE → DIAMETER → 14 ÷ 20 cm
(it must be chosen to have a drilling tool able to drill the hole with this size)

When we refer to small bolts we talk about FOREPOLING, usually these bolts are installed using self drilling bolts: that are bolts with the cutterhead in front of the bolt that is able to drill an hole where install the bolt.

Steel pipe umbrella

The steel pipe umbrella or canopy is obtained by installing steel pipes ahead of the tunnel face.
The usual dip of the steel pipes is ranging between 5° and 10° (with reference to the horizontal).
The pipes form a protection vault with a truncated cone shape to allow the overlapping of two adjacent fields

↓ there is always an overlap



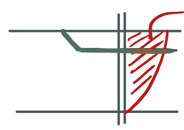
← not cylindrical shape but troncoconical shape because the umbrella cannot be drilled parallel to the tunnel excavation

These elements provide temporary overhead protection while excavation for and installation of the next set of steel set is accomplished. (excavation step by step)

!!! The length of the reinforcement depends on the avaible machine! The STEEL PIPE UMBRELLA LENGTH range is → 12 ÷ 18 m (the range of length for which the machine are avaible)

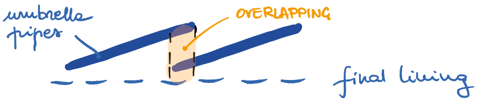
12 → most used length with an excavation of 9m and an overlapping of 3 m (average value)

(1) I excavate my arch (1m)
(2) I install the reinforcement

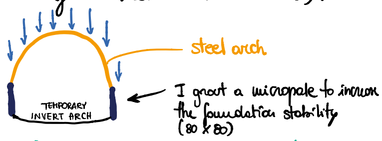


N.B. The overlap is important because the sliding surface should not exceed the residual length of the pipe when I'll stop excavation otherwise I'll have stability problems. The residual length (overlap) depends on the stability evaluation should be able to guarantee that the sliding surface ahead of the face (calculated by Anagnostou & Kovari) is stable.

The other possibility is to use FACE REINFORCEMENT on the face.
The excavation with the steel pipe umbrella and face reinforcement can be done HEAD & BENCH or FULL FACE.



it is important to use a large footing steel arch. Why? Because we excavate a bit more than we need and we enlarge the footing of the steel arch (to avoid stability problems of the steel arch: settlements).



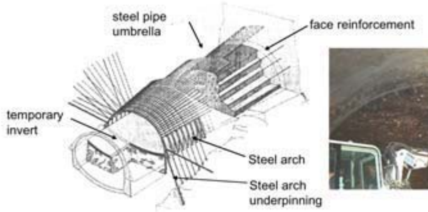
Si devono usare centine a piede allargato (o a zampa di elefante) perchè abbiamo bisogno di più capacità portante. Questo perchè tutti i carichi del tunnel vanno a finire sui piedi dell'arco (on the footing of the steel arch).

The design of the intervention with steel pipe umbrella is complex due to:

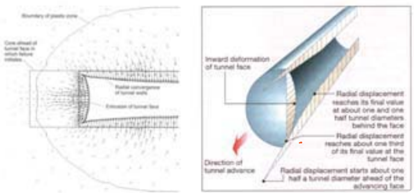
- three-dimensional geometry of the tunnel face;
- high number of involved parameters;
- tunnel face stability influence;
- three-dimensional analysis to take into account the face position;
- overlapping of the umbrellas;
- type of connection steel pipes and steel ribs;
- stiffness and foundation quality of steel ribs.

The longitudinal reinforcement elements are supported by:

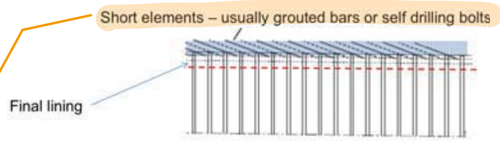
- the previously installed shotcrete lining and steel ribs behind the tunnel face
- the unexcavated ground ahead of the face.



The diameter of the steel pipes is usually ranging between 60mm up to 200mm with a thickness ranging from 4mm to 10mm.
The length of a steel pipe umbrella ranges from 9m to 15m with overlapping ranging from 3m to 6m.
The overlap guarantees the stability of the face.
The tunnel face can be reinforced usually with fiber-glass. The advancement step can be different ranging from 0.5m to 1.5m and it is supported usually with steel ribs

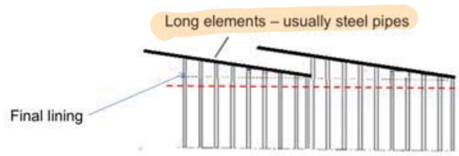


It is important to remember that the boundary displacements and the stability of a tunnel near the face is a **tri-dimensional problem** and as such it should be studied and analyzed.



FOREPALING: usually they are not long elements as the ones that we have seen till now, but they are short elements installed using grouted bar or self drilling bolts. They have much bigger overlapping and they are more inclinated.

this technology does not require a special machine (for installation of steel pipe umbrella instead we need special machine: POSIZIONATORE)



Steel pipes umbrella can be used both in fractured rock mass and in soil. When we have the faults (faglie) (they have the shear zone) they create fractures in rock mass so it is needed the use of steel pipes and the installation of steel arches.

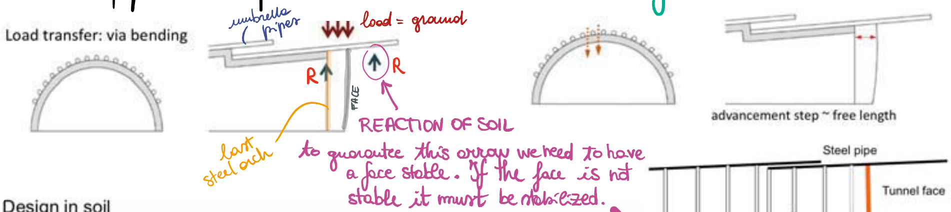
N.B. The steel pipes are not kissing one with the other, there is a gap between them. Usually in soil there are 3 steel pipes in 1 m (20 ÷ 25 cm di passo).

It is important to highlight that for this application it is foundamental that (the soil) material doesn't flow between pipes. If we're working in a clean sand (sabbia sciolta → no coesione) this technology doesn't work properly (Se lavoriamo in sabbia sciolta (senza limo e/o argilla che danno un po' di coesione) → abbiamo l'effetto clessidra)

STEEL PIPES UMBRELLA DESIGN ASPECTS

Eeach pipe is indipendent and works as a single beam. There is no trasversal arch effect.

Load transfer: via bending

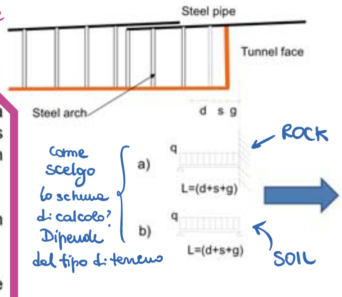


Design in soil

to determine the *required steel pipe section*, the pipe is considered to be a continuous beam on two or on multiple supports, the steel sets, that is embedded in the ground ahead of the excavation face that acts as a support an a flexible support.

The concrete which fills and surrounds the pipe is not normally considered in the calculation.

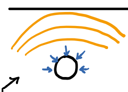
The computation is carried out for the most critical phase, which is just before the installation of the steel rib as the free span is the longest.



Evaluation of bending moments and shear stress depending of the free span of the pipe and on the acting load

How? → using elements fully grouted parallel to the tunnel axis. This elements are usually FIBER GLASS elements of reinforcement. These elements are popular 'cause they are easy to be excavated, they break when we excavate, but the strength tension of a fiber glass element is similar to the steel.

WAY TO DESIGN STEEL PIPE UMBRELLA



Analytical design

The acting load on the single pipe [q] can be evaluated, starting from the pre-defined value of the maximum vertical stress

Generally the q load is a dead load. The q load is the load that it is not bring by the arch effect but by my structure.

$q = p_v \cdot i$ — TERZAGHI FORMULA (approccio conservativo)

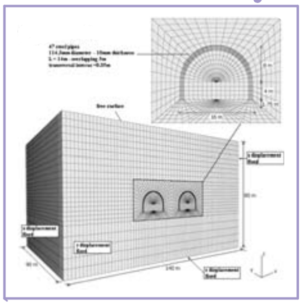
where "i" is the spacing between the pipes.

One of the problems is the evaluation of the acting vertical stress near the tunnel face.
In many cases, it is empirically assumed that p_v = 0.50-0.75 of the vertical load before excavation and the load can be is evaluated by the well known formulation of Terzaghi or (for low overburden taking into account the whole vertical load)

The *length of the steel pipe* is linked to practical reasons, that is, drillability and the maximum bore hole deviation which limit the length of 15 – 18 m (→ it depends on machines used)

The *length of the overlap* (→ 3 m (average value)) between two subsequent umbrella is controlled by the behavior of the ground ahead the tunnel face.
In recent years there have been numerous studies on tunnel face reinforcement with longitudinal pipes based on small scale laboratory tests, field tests and numerical modelling. The results of these researches suggest that the length of the overlap must not be less that 0.3 – 0.4 times the equivalent diameter of the tunnel. Analysis of the face stability can be also carried out to achieve this parameter. → An evolution of the overlap length can be done tacking into account the size of the Anagnostou & Kovari wedge.

3D numerical modelling



The *interax between the pipes* is chosen taking into account the fact that the ground must not flow between the pipes. Therefore, the natural cohesion of the ground should be able to control and prevent the occurrence of this phenomenon. Simple calculations can be carried out, considering the stability of the slice of ground onto two nearby pipes.

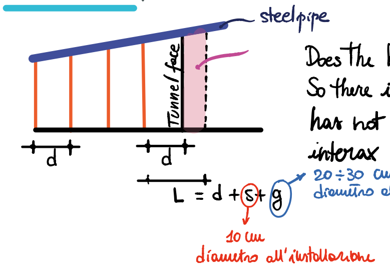
- fine material small i
- cohesive material bigger i

i:
- 2 pipes for 1 m → 0.5 m
- 3 pipes for 1 m → 0.33 m
- 5 pipes for 1 m → 0.2 m

2D numerical model (not usefull)

The only way to try to model the steel pipes umbrella in a 2D model is to simulate that as an arch (but remeber that the steel pipes umbrella doesn't work as an arch!).

PAY ATTENTION!



Does the boundary of tunnel face support the steel pipe? No, it would be unreal. So there is a certain length that depends on the rock quality and on the excavation process that has not to be considered in the design. The real length for the design is not only the interax between two steel arches (d), but L. Non posso considerare soltanto l'interasse tra le centine: mi serve dello spazio extra per posizionare la nuova centina, non posso immaginare che il terreno agisca già sul fronte (il processo di scavo, lo stesso rilascio tensionale lo altera).

Design procedure

1) Choice of a tentative interax
2) Choice of a tentative diametre and thickness of the pipes
3) Choice of a tentative interax between the steel arches
4) Choice of the steel pipe

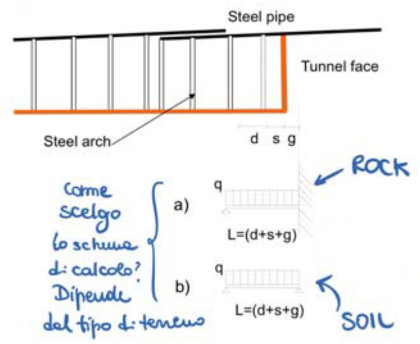
} ← Change the geometrical parameters

5) Evaluation of the design length

6) Choice of the structural schema

7) Computation of the bending moments
8) Computation of the acting stresses inside the pipe
9) Comparison between the acting stress and steel yield stress — not adequate ↑

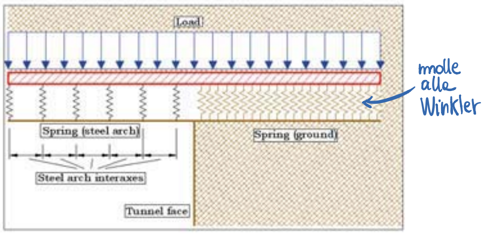
10) Definition of the steel pipe length (on the basis of available machines)
11) Definition of the excavation length
12) Design of the supports and of steel arch foundations



The described analytical design approaches are very simple and their application has been consolidated in time but they neglect some parameters which are very important for the design:
- does not consider the real stiffness of the supports (steel sets and ground)
- the effect of the ground ahead of the face

To take into account these aspects it is possible to use more complex design schemes:
- an analytical scheme based on the approach of a beam on multiple supports
- numerical models (tri-dimensional) which must be used where it is necessary to know the induced settlements

Beam on multiple supports



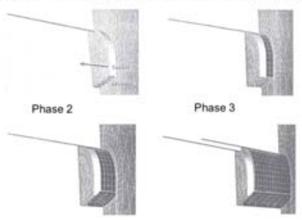
The less is the stiffness of the ground the higher the bending moment ahead the face is

The problem is the definition of the load distribution and the spring stiffness

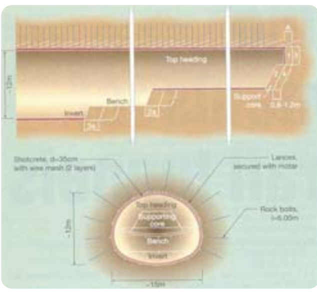
→ 3D Numerical modelling

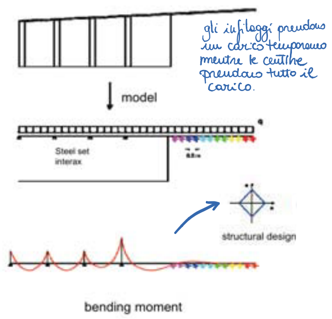
All the pipes pipes can be modelled. The advancement steps can be studied.

Problem: complexity of the numerical model and time of computation



Beam on multiple supports





→ 3D Numerical modelling vs analytical model

A difference has been observed between the analytical model results and the numerical model results with reference to the modeling of the displacements and stresses in the area immediately behind the tunnel face.
This effect is probably due to the impossibility with the analytical model to analyze the face extrusion.

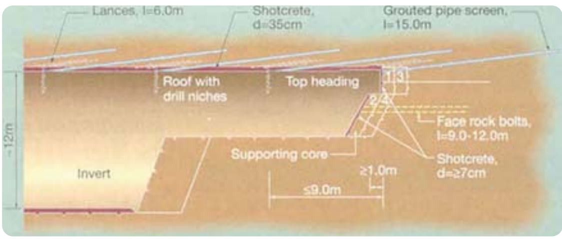
To take into account this effect it is possible to fictitiously reduce the stiffness of the springs in the last 4m behind the face.

→ 2D Numerical modelling

Some authors proposed to use a ground reinforced arch around the tunnel to model the action of the pipes also recently.

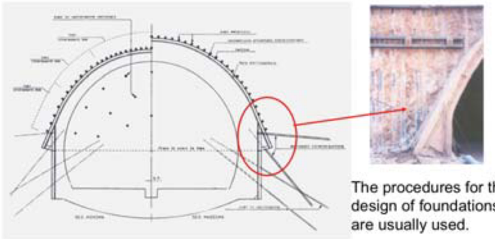
It is significant to highligth what Hoek wrote in 2001

A very crude approach is to assume that a zone of "improved" rock can be used to simulate the forepole arch. The improvement of rock mass properties is estimated by considering the weighted average (based on cross-sectional areas) of the strength and deformation properties of the steel forepoles, the grout filling and the original rock mass. While this model does not correctly represent the three-dimensional bending strength of the umbrella, it does permit the construction of a two-dimensional model that behaves well numerically



Steel arch foundation: design concepts

The stability of the steel arch foot must be designed to guarantee that there are no displacements
Enlarged foot; Foundation with micropiles or jet grouting columns



The procedures for the design of foundations are usually used.

It is important the good contact between the steel arches and the pipes.

Small displacement of the pipes in this position along the tunnel can cause the collapse of the structure or induce critical displacements of the surface.

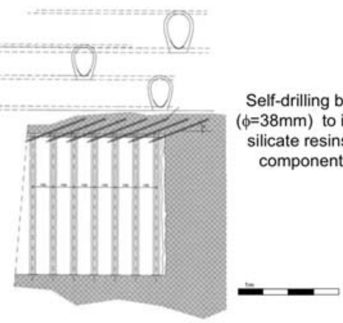
The steel sets must not move → need of a good foundation (steel sets with enlarged foot)

In special cases, it can be usefull to use the bullflex pillow to guarantee the contact beween the pipes and the steel arches.

CUTTER AND COVER METHOD (metodo Milano)

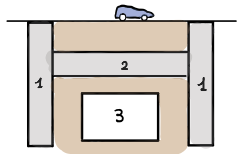
Turin metro – lot 1 – Conventional exacavation with lances (or spiling)

Steel arches
IPN 160 double
+ bullflex



Self-drilling bolts (ϕ=38mm) to inject silicate resins (2 components)

I excavate a diaphragm wall, I excavate the tunnel till certain depth with the road closed. Then I create a concrete slab, then I reinstall the surface and then I excavate the tunnel. This is the cut and cover method. It is applied in urban area where we have large roads that needs to be reinstalled as soon as possible.



(1) DIAFRAMMA
(2) PIATTAFORMA
(3) SCAVO

↓

But there is the problem of closing the road

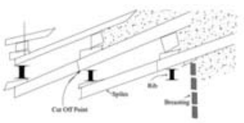
alternativa

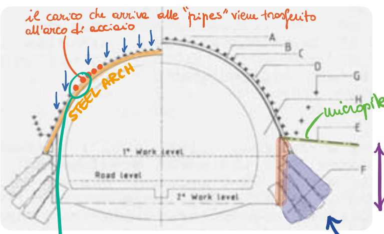
Spiles

it is used usually in rock, but thanks to the use of resins can be a good tech also in soil.

The spiles or lances are short reinforcing element usually self drilling bolts i ahead of the tunnel face.
The usual dip of the steel pipes is larger than 10° (with reference to the horizontal).
The lances form a protection vault with a truncated cone shape

→ short rock bolts to create a reinforced arch around the tunnel. But since we're working not in a rock mass but in the soil, it isn't enough adding only steel inside. We need to create an arch effect injecting silicate resins. These can expand creating the arch.





N.B. per garantire la stabilità del muro (≈ 2m) devo fare un jet grouting di colonne.

≈ 2m

problem of contact between pipes and steel arch
↳ some settlements can occur ⇒ BULLFLEX PILLOW (expansive)

they're used for a perfect control of settlements

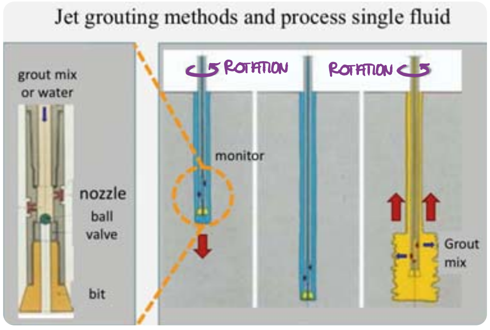
it is a solution to solve the problem. The bullflex pillow is a bag made of geotexile, that is filled using shotcrete. In this way I will confine immediately the element → "I prestress" my steel arches against the steel pipes compensating the possible settlements of the steel arches.

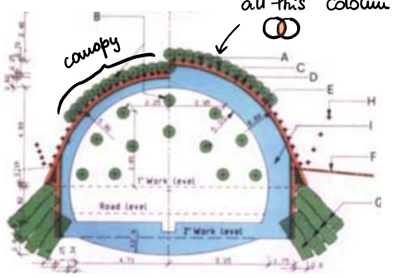
6. JET GROUTING CANOPY

Jet grouting is a technology to inject mortar at high pressure (300 ÷ 800 bars) (mortar = acqua + cemento) from some nozzles and from a drilling rod (asta di perforazione). Owing to the high pressure I'll destroy the soil structure mixing soil structure with a grouting mix. Why do we do this operation? To reinforce soil. So which are the steps of this operation?

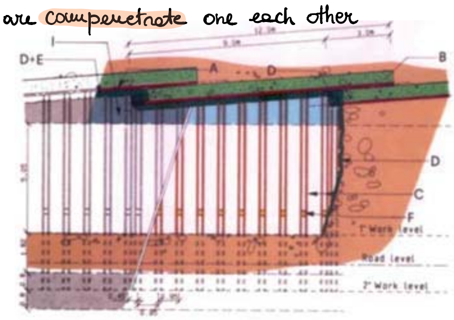
(1) Making of perforation by a drilling rod.
(2) Injection of grouting mix at high pressure.

What do we obtain? Something like "concrete using soil as aggregates".





all this colomn are compenetrate one each other



↳ the jet grouted arch around the tunnel lets the tunnel to be stable for the advancement step.

The goal is to create a canopy: making a jet grouting on the boundary of the tunnel I build an arch (canopy), a real one. Pay attention: now the structural model is not more a single beam, but it is really working as an arch!

N.B. Also when we use this kind of tech the face has to be stable → in the draw fiber glass + jet grouted colomns are used to stabilize the face
(so jet grouting can be used to stabilize the tunnel face)

GEOLOGICAL WARNING

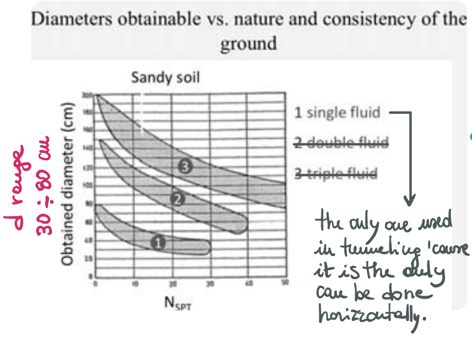
! This tech works very well in homogeneus soil: the jet grouting guarantees a final homogeneus colomn only if the soil is homogeneus. If the soil is strongly dishomogeneus, the final colomn could be not continuous → problem 'cause we lose the arch effect.

For example in a clean sand, I can't use the steel pipes 'cause the sand will go through. The sand instead is perfect to create a jet grouting. If the sand is not so clean but it has some blocks, I'll use both techs: jet grouting + steel pipes. (RICORDA: quello che comanda nella scelta è il tipo di terreno).

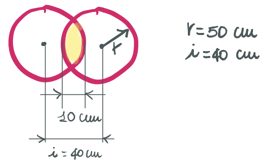
The transversal geometry is exactly the same of the steel pipes as the machine used.

How to design a jet grouting intervention? It requires: 1) to understand the properties of a jet grouted colomn.
↳ COLUMN DIAMETER d
↳ UNIAXIAL COMPRESSIVE STRENGTH σ_{vo}
(key design parameters assumed by experience (no analytical formulas))

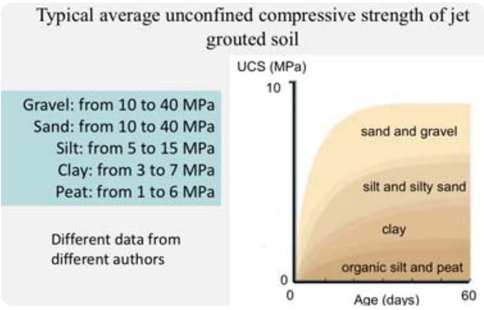
In technical literature there are many relationships linking d with the geotechnical properties of the ground. The most famous is (d, N_{SPT})



→ once we get the diameter, we can place the columns. Remember we need an overlap zone ≈ 10 cm.

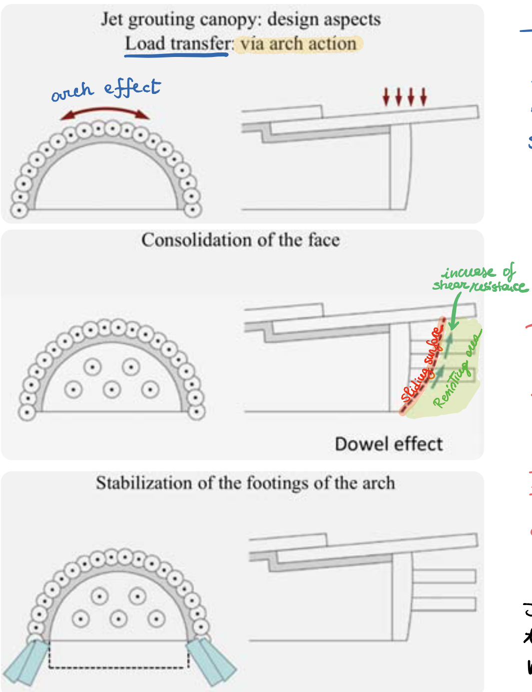


Regarding the uniaxial compressive strength we refer to technical literature:



function of the kind of soil

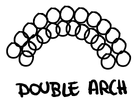
N.B. The only possibility to verify these two hypothized datas (d & σ_{v0}) is to carry out a test in situ.



The design can be done as an arch. So we have to evaluate if the stresses induced by the action of the soil will not exceed the strength of the material taking into account a certain safety factor SF.

If we use the grouting reinforcement to stabilize the tunnel face, we create some columns that intercept the sliding surface with a resistant cross section: with the limit equilibrium analysis we get a certain zone with higher shear resistance.
To make the design of the face reinforcement using jet-grouting columns we use the L.E.M. but we modify it considering that we have a bigger shear resistance in the area where the columns are placed. Practly we use the Anagnostou & Kovari equations, but in the equilibrium of the wedge we add on the sliding surface of the system an increase of shear forces due to the DOWEL EFFECT. In other words we're adding some additive cohesion artificially.

Jet grouting can be used not only to create a single arch, but if we need to have more thickness we can build more arches with the same machine just overlapping them. →



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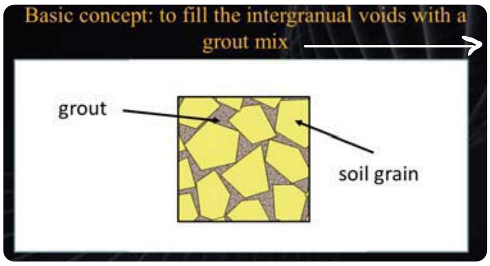
7. PERMEATION GROUTING IN SOIL

↳ impermeabilization + increase of the ground's strength and stiffness.



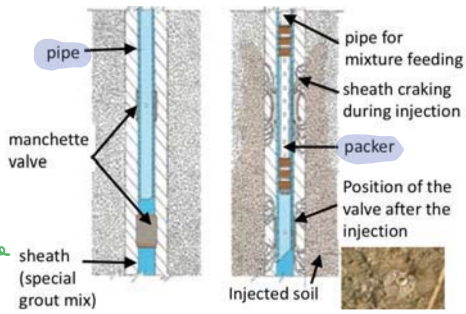
→ a material able to harden, for example a water cement mix mortar, or a chemical mix.

In this way I change the material increasing the uniaxial compressive strength (increasing of cohesion) and reducing the permeability ('cause in soil water can go through the intergranular voids, but filling them with something the water cannot pass through anymore).

So permeation grouting (≠ jet grouting) is a way to change the mechanical properties of the soil from an engineering point of view to achieve the goals for a correct tunnel: more strength and less permeability. → That's the technology I may use when I'm below the water table. This technology can be applied both in conventional tunnels and in mechanized tunnels.

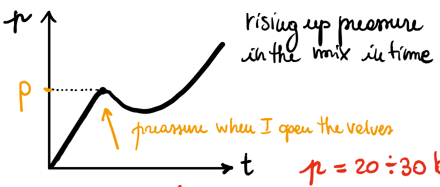
The most used technology to inject the grout exactly where I want (in soil) is **MACHETTE PIPE**

The idea is to create an hole, to insert a PVC pipe that has some holes, this hole zone is covered by rubber ring. Inside the pipe we insert the **PARKER**. When I send the mix under pressure the mix will come out my parker, will deform the rubber ring, from the hole in manchette pipe will go out and will grout the soil.



Domanda d'esame: descrivi cos'è e come funziona una machette pipe.

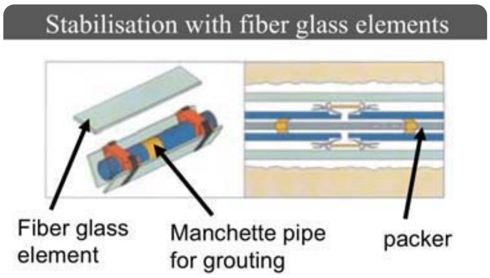
We have a PVC pipe with some holes at a certain interax (2 ÷ 3 valves per meter). It is inserted in a drilled hole. Then inside the manchette pipe, I insert an injector device (another kind of pipe) provided by the parker element. The parker is a closed pipe. When I pump the injection mix it will come out of the parker, it will fill completely inside the manchette pipe. At a certain pressure the mix will come out from the pipe's holes grouting the soil.



$p = 20 \div 30$ bar
(1 order of magnitude less than jet grouting)

(As consequence there is a certain gap between PVC pipe (manchette one) and the hole must be filled before starting the injection, contrary the mix will not enter in the soil but it will run in this gap → So it is compulsory filling this gap completely with mix before we start the real injection. This is usually a water cement mix with a lot of bentonite (5÷6 %) to make it enough stiff to fill the gap).

Si fa 1 perforazione, inserisci un tubo in PVC che ha i fori e le guaine in gomma e la canna machette. Dopo di che ci metto un tubo di iniezione che è composto dal porker. Questo ultimo consente di creare una piccola camera di pressione a cavallo della valvola perchè ha 2 sigilli in gomma che si espandono. Inietto, ad 1 certa pressione si apre la valvola e la miscela va nel terreno.

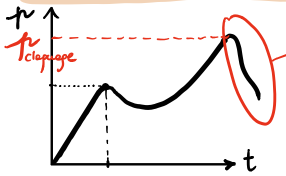


N.B. Our goal is not to demolish the soil structure. Our goal is to permeate "softly" the soil changing for this reason its properties.

MIX: water + cement + bentonite

N.B. The permeation grouting doesn't work in clay! Because the intergranular voids are so small.

CLAQUAGE PRESSURE



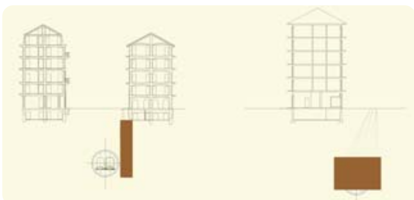
When I start to inject, the pressure in my pipe is rising up. Then I open the valves and the material starts to flow, in the soil there is a reduction of pressure (permeation time), and the grout will reach a certain radius from the valves that depends from the soil and from the mix. There are no empirical relationship that links the property of the mix and the property of the soil to determine analytically the radius of injection r. This radius can be only determined by experience.

(radius of injection → is a design parameter)

At a certain pressure the material is not able anymore to adsorbe any mix. In my soil I start to create some layers of mix that destroy the soil structure. This phenomena is called: CLAQUAGE. It is an unwanted effect that occurs if I'm keeping pushing the mix over the ability of the soil to adsorbe the mix itself. This fenomena can occur depending on another injection design parameter: the injection speed → the more fast I inject the mix, the quickest I reach this value.

The design parameters are:
- radius of injection
- acceptable value of injection speed
- acceptable pressure (claquage pressure)

N.B. The claquage phenomena could be sometimes a wanted effect: COMPENSATION GROUTING



Quando scavo non posso evitare dei cedimenti (ad esempio di fondazioni) delle strutture soprastanti → the goal is to generate displacements in soil (slowly) to create an uplift of the building compensating the settlements.

Injections are very flexible, they can be done from the surface, from the underground, to obtain many different geometries. The key design parameters are: * geometry needed for my engineering goal, * soil parameters.

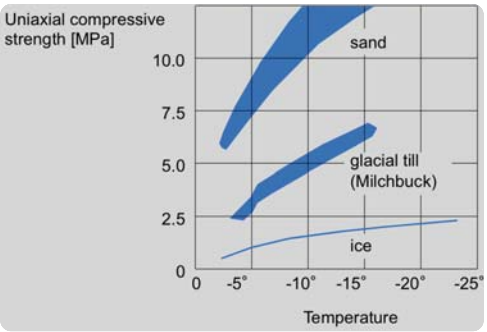
The design of grouting is done using an controlled volume and a controlled pressure.
I have to guarantee the correct overlapping of the injection volumes (overlapping 10÷50 cm)
From each valve I have a certain volume of injection: it depends on geometrical properties.

If we know the volume index of the soil, I know the amount of injected volume.

N.B. Never inject all volume in one step! (two/three steps)

8. FREEZING

A problem can occurs in permeation grouting when we fast moving of the underground water. One possibility to pass through this problem is to freeze the water in soil → we get the frozen soil.



How does it work? I create the drill hole, then I pour liquid nitrogenum into (nitrogeno liquido), and the soil starts to freeze. It takes some times to freeze completely the ground. In this way we create a reinforced arch of frozen soil. The ice has a resistance bigger than the soil.

→ the main problem is to determine which is the uniaxial compressive strength of the soil

BIG DESIGN PROBLEM

('cause it is technological difficult to carry out conventional test on the frozen soil)

What is good of frozen soil is the perfect water controll

Ice expande the water volume ⇒ when we defreeze the soil, the soil lose its natural coherion ⇒ it disturbes very much the soil. So in the structural design of the lining we have to take into account that the mechanical properties of the soil assessed before freezing the soil, have lower down.

9. DRILL & BLAST

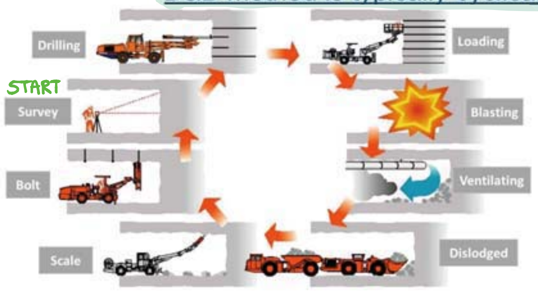
DRILL & BLAST METHOD AND BASIC CONCEPTS

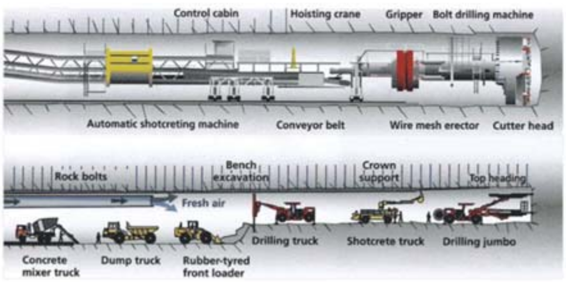
The excavation of tunnels by drilling and blasting consists in breaking and removing, in succession, cylindrical volumes of rock, whose cross section is the **excavation cross section**, and whose length is the **pull** of the round.

By **"round"** we intend a set of blast-holes cooperating in the fragmentation and displacement of said volume, that subsequently will be removed by the mucking machines, uncovering a fresh rock face, in advance with respect to the previously blasted one by the length of the pull of the round.

- Referring to the main component of the total charge, NG based explosives are the most commonly used; NG content is not high (most common types are in the 10% to 35% range); specific gravity (cartridge) over 1.3 is preferred

D&B method is typically cyclical, even though superposition is possible



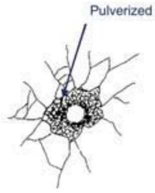


To proceed to break a rock mass by means of an explosive, a **free face** is needed

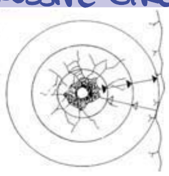
In tunnelling, excavation is more difficult than in bench blasting: **only one free surface is present and available for drilling**

There are 3 stages of blasting process

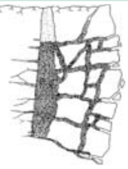
① RADIAL CRACK FORMATION



② COMPRESSIVE STRESS WAVE



③ GAS PENETRATION OF CRACK FORMATION



Il drill & blast method è usato solo nell'avanzamento in rocce dure, dove la lunghezza del tunnel è di modesta entità (1-2 Km) e quindi per scavi in cui la mobilitazione di una fresa a full face sarebbe economicamente non conveniente.

Tale metodologia prevede l'esecuzione di un determinato numero di fori in relazione alle condizioni geotecniche e di contesto, eseguiti tramite specifiche macchine dette jumbo. Negli stessi fori viene, in una seconda fase, inserito l'esplosivo che provvederà alla vera e propria frantumazione del fronte di scavo per la lunghezza e dimensioni desiderate. L'esplosivo è un materiale che brucia velocemente producendo gas ad alta pressione, che confinato dentro i fori determina la frantumazione della roccia. In questo contesto bisogna progettare la posizione dei fori, che non è casuale, e la sequenza di esplosioni, che tipicamente si susseguono con intervalli di pochi millisecondi, così da creare una superficie libera che si irradia dal centro alle estremità del fronte.

L'insieme delle mine per compiere la demolizione è detto VOLTA e si distinguono in:

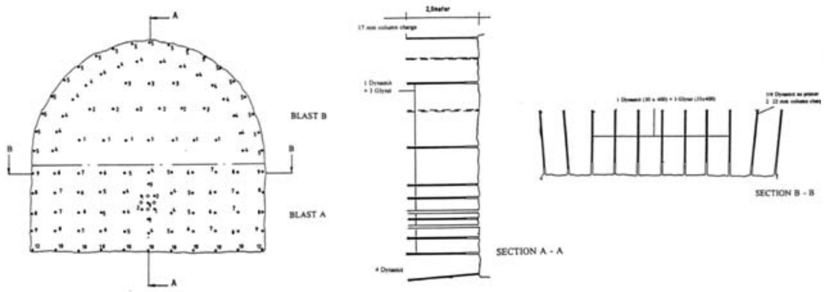
- **MINE DI RINORA**: posizionate nel centro del fronte tipicamente sono inserite in fori a formare una V, sono di intensità maggiore rispetto alle altre esplosioni in quanto devono formare la superficie libera dalla quale dipartiranno le successive esplosioni.
- **MINE DI PRODUZIONE**: a ridosso delle prime si stendono con una forma circolare e sono quelle che producono il maggior quantitativo di materiale frantumato. Al fine di lavorare ottimizzando la frantumazione necessitano di una superficie libera prima formata.
- **MINE DI CONTORNO**: di intensità notevolmente inferiore, per evitare sul contorno frantumazioni non volute, sono le ultime a brillare consentendo l'eliminazione della roccia sul perimetro della galleria.

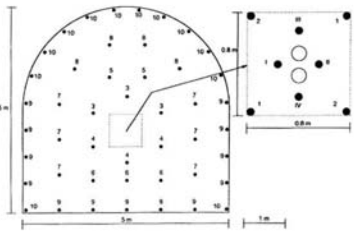
All'eliminazione dei gas dovuti all'esplosione, tramite opportuni sistemi di ventilazione e del materiale frantumato, segue lo SCALING e il supporto del contorno con centine in acciaio.

Con tale metodologia si arriva ad un avanzamento massimo del fronte di scavo di 5 metri, ma il principale deficit è legato alle fasi e tempistiche successive alla detonazione.

THE BLASTING SCHEME

- it is the **executive plan design project of the round**; it sets location, diameter, length and direction of the holes, charging and timing
- the work on rock media, inhomogeneous and poorly known in the local details, can not respect strict geometrical tolerances as metal working; however, reasonable tolerance limits must be set and respected;
- the blasting scheme comprises a quoted drawing of the round and a table containing the charging and timing data of the holes.



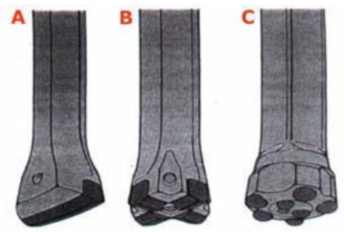


Charging table

n° - funzione mina	n° - tipo di cartucce per ogni mina	kg di esplosivo per ogni mina
8 - apertura ritardi (I,II,III,IV,1,2)	4 - GD1 + 4 - Tutagex	5.2
21 - allargo ritardi (3,4,5,6,7,8)	3 - GD1 + 4 - Tutagex	4.5
7 - rilevaggio ritardi (9,10)	4 - GD1 + 4 - Tutagex	5.2
18 - profilo ritardi (9,10)	1 - Tutagex + 5 - Emuldin + 1 - Tutagex	2.45

it depends on the kind of rock wær I'm excavating

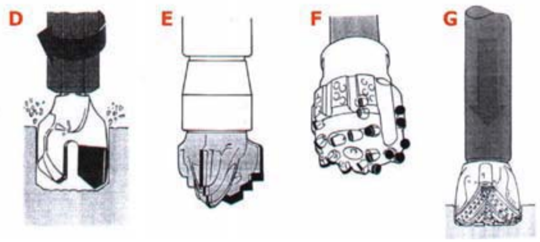
DRILLING



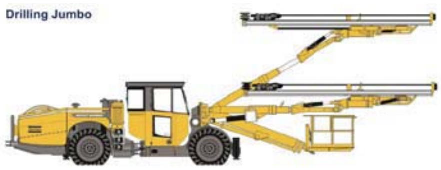
Examples of drilling integral steels and drilling bits (Sandvik, 1995). **A, B, C, D: drilling tools for small – medium diameter holes.**

A: integral single chisel steel for percussive or rotary–percussive drilling *of abrasive rocks, ϕ 30 – 40 mm;*
B: integral cross chisel steel for percussive or rotary-percussive drilling *of abrasive rocks, ϕ 30 – 50 mm;*
C: bottom bit for percussive or rotary–percussive drilling *of abrasive rocks, ϕ > 40 mm;*

D: two wings or U drag bit for rotary drilling of *softs or mildly abrasive rocks, ϕ 30 – 40 mm.*
E, F, G: drilling tools for mediun-large diameter holes. E: "pine tree" *rotary drag bit, for rotary drilling of large diameter holes in mildly abrasive rocks;* ***F: rotary bit with "nail" carbide inserts,*** *diamond coated, for rotary drilling of abrasive rocks;*
G: tri-cone bit *for rotary drilling by rolling tool, suitable to hard and tough rock drilling (only for very large diameter > 200 mm)*
The examples shown cover only a very small part of the commercial tools types.



Drilling Jumbo



EXPLOSIVE

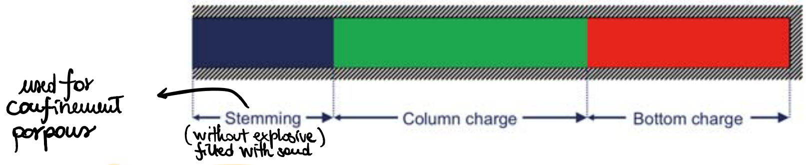
Ingredients: Decreasing cost ↓; Increasing ammonium nitrate ↓; Decreasing nitroglycerin content ↓

Non-gelatinous	Gelatinous
Nitroglycerin	Blasting gelatin
Straight dynamite	Straight gelatin
High-density ammonia dynamite	Ammonia gelatin
Low-density ammonia dynamite	Semigelatin
Dry blasting agents	Slurries

Increasing water resistance →

Properties: Decreasing density ↓; Decreasing detonating velocity ↓; Decreasing water resistance ↓

Typical production hole composition



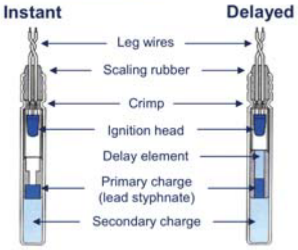
Types:
- Packaged in cartridges
- Bulk

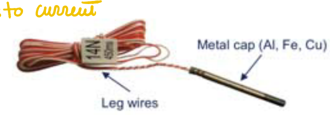
Originally, tunnels and rock caverns were almost exclusively blasted with explosives in packaged form. Lately the tendency is to use bulk ANFO using boreholes of 45 mm and larger. Slurries have been then introduced to solve the problems linked to the water leakages (ANFO is lighter then water).
Bulk explosives are preferable because the components (which alone are not explosives) are mixed directly in the holes having a safer working environment and filling in a better way the hole.
Bottom charges are usually dynamite cartridges.

INITIATION SYSTEM

The most common blasting works do not consist in firing single charges, or in simultaneously firing many charges, rather in firing several charges according to a predetermined sequence, which means explosion timing.

Electric detonators → work thanks to current





Blasting machine

Diameter (mm)	Cross section (mm²)	Copper wire resistance (each, Ω)	Iron wire resistance (each, Ω)
0.5	0.196	8.9	-
0.6	0.283	6.1	42.0
0.7	0.385	4.5	30.9
0.8	0.503	3.4	23.7
0.9	0.636	2.7	18.7
1.0	0.785	2.2	15.2
1.2	1.131	1.5	10.5
1.4	1.539	1.1	7.7

Electric requirements	Low intensity	High intensity
No ignition with 5' exposure to direct current of	0.2 A	4 A
Ignition in less than 0.01 s with a direct current of	0.5 A	7 A
No ignition when the electric impulse is lower than	0.8 mWs/Ω	1100 mWs/Ω
Ignition with electric impulses in the range	0.8-3 mWs/Ω	1100…2500 mWs/Ω
Electric resistance of the ignition filament of	1.4-1.7 Ω	0.05 Ω
Firing of a series circuit requires a current of	1.2 A	30 A

Environmental conditions	
Presence of water	Suitable (connections insulated, especially for HI and MI)
EM fields/stray currents	LI/MI not suitable, HI suffers limitations

Operative conditions	
Time gap between ignition and explosion	Practically nil
Possibility of sequential explosions	Max 52 explosion times, danger of misfires for insufficient current
Max number of detonators in a blast	2000 for LI; 160 for HI
Control of the blasting circuit before blast	Visual and electric with approved ohmmeter
Ease and celerity of use	Good
Accuracy of timing	Good
Space required	Minimal

$i = \frac{V}{R}$ → 1ª LEGGE DI OHM per calcolare l'intensità necessaria

NONEL detonators (shock tube)

funzionano senza elettricità

1) **Aluminum cap** of variable length for the different delay times.

2) **Base charge** of high explosive, giving a strength number 8.

3) **Primary charge**, sensitive to flame.

4) The desired **delay** is given by an Al tube filled with a pyrotechnic mixture (a part is directly pressed in the shell).

5) The detonator is crimped on a **sealing rubber tube**, a part of which protrudes to protect the Nonel tube against wear.

6) A **Nonel tube** of determined length, sealed at the opposite end.

Shock tube is a small diameter (3mm outer diameter) laminated plastic tube internally coated with approximately 14.86 grams of reactive material per km. This tube transmits a low energy signal from the point of initiation to the delay cap at approximately 2,100 m/s. The detonation is sustained by such a small quantity of reactive material that the outer surface of the tube remains intact during and after functioning.

Structure in 3 layers (generally):

1. Innermost → good adhesive properties
2. Middle → good tensile and radial properties
3. Outermost → good abrasive resistance and coloured in different ways depending on the delay

NONEL UNIDET

Unit	Detonators		Connector blocks			
Code	UB 475	UB 500	UB 0	U 17	UB 25	UB 42
Delay (ms)	475	500	0	17	25	42
Colour	Yellow/red	Yellow/green	Yellow	Blue	Red	Green
Symbol			▷---	▷---	▶---	▷---

NONEL GT/MS

Period	Delay (ms)	Interval (ms)
3…20	75…500	25

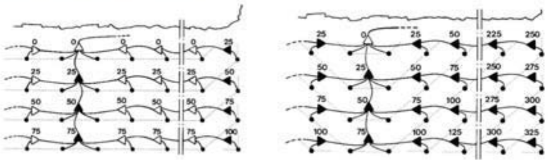
NONEL GT/T

Period	Delay (ms)	Interval (ms)
0	75…500	-
1…12	100…1200	100
14, 16, 18, 20	1400…2000	200
25, 30, 35, 40, 45, 50, 55, 60	2500…6000	500

Underground use: NONEL LP detonators

Period/Delay (ms)	Delay tag colour	Period/Delay (ms)	Delay tag colour
0 / 0	Pink	10 / 3500	Green
1 / 500	White	11 / 3900	Yellow
2 / 800	Light blue	12 / 4400	Red
3 / 1100	Orange	13 / 4900	White
4 / 1400	Green	14 / 5400	Light blue
5 / 1700	Yellow	15 / 5900	Orange
6 / 2000	Red	16 / 6500	Green
7 / 2300	White	17 / 7200	Yellow
8 / 2700	Light blue	18 / 8000	Red
9 / 3100	Orange		

Blasting/firing patterns (open pit)



Underground use

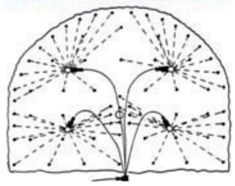
The hook-up of NONEL LP in tunnelling is quickest and most easily done with what is known as **bunch connectors**.

Nonel tubings from the loaded boreholes are collected in bunches with maximum 20 (minimum 5) tubing per bunch. A bunch connector is fastened in each bunch by the affixed loop of detonating cord. Then the tubings of the bunch connectors are coupled together in a snapline 0 and the blast is ready for firing.

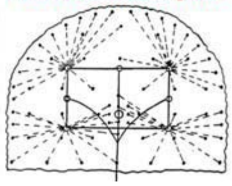
Bunch connector delays (Shockstar)

Typical blast rounds in tunnel

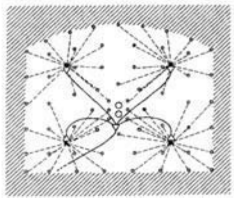
NONEL LP + bunch connectors

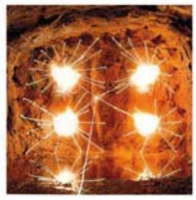


NONEL LP + detonating cord

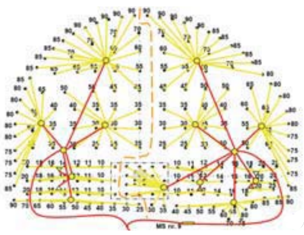


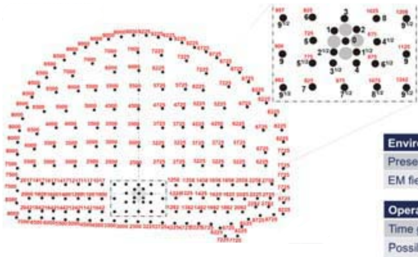
Blasting/firing pattern Tunnel





BLASTING SCHEME





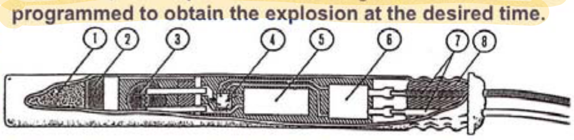
Environmental conditions	
Presence of water	Suitable
EM fields/stray currents	Suitable

Operative conditions	
Time gap between ignition and explosion	Practically nil
Possibility of sequential explosions	Infinite sequential explosion times
Max number of detonators in a blast	Infinite
Control of the blasting circuit before blast	Just visual
Ease and celerity of use	Good
Accuracy of timing	Good
Space required	Good

Electronic detonators

Precise but high cost!

The detonator includes, besides the charge and the ignition head, a **capacitance** storing the electric energy needed to obtain the ignition, and an **electronic timer**, which activates the discharge of the capacitance of the ignition head at an accurately (± 1 ms) programmed time. **Upon charging of the blast hole, the capacitance is charged and the timer is programmed to obtain the explosion at the desired time.**



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

Environmental conditions	
Presence of water	Suitable
EM fields/stray currents	Suitable

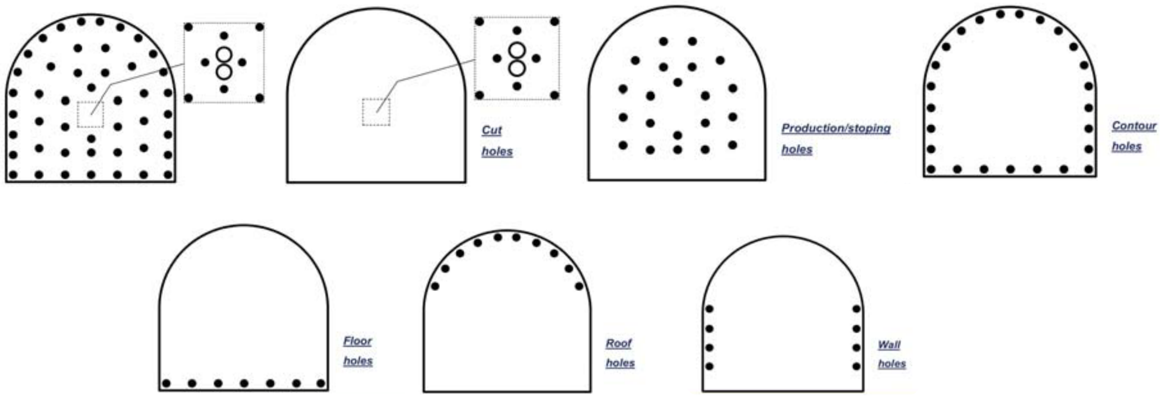
Operative conditions	
Time gap between ignition and explosion	Practically nil
Possibility of sequential explosions	Up to 8000 explosion times, with 1 ms interval time
Max number of detonators in a blast	2000
Control of the blasting circuit before blast	Visual and electric with approved ohmmeter
Ease and celerity of use	Very good
Accuracy of timing	Excellent
Space required	Minimal

OSS. the CHARGING is the operation in which the charge is inserted in each hole

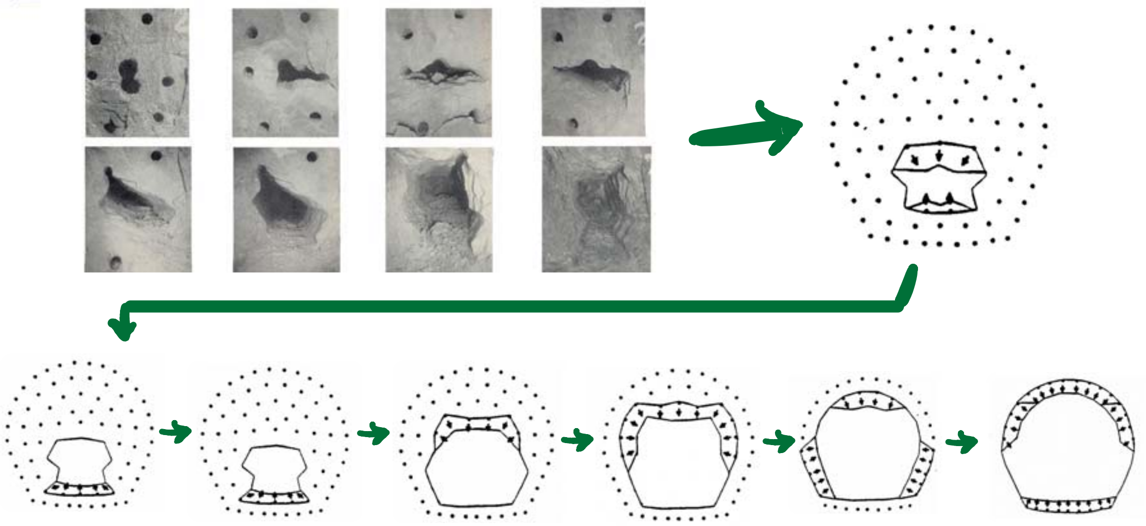
TUNNEL EXCAVATION BY D&B

PHASES
1) Charging
2) Smoke clearing
3) Mucking
4) Scaling
5) Bolting
6) Marking the next round

Nomenclature



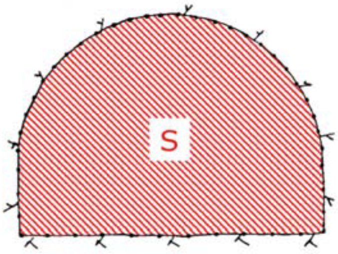
Cut



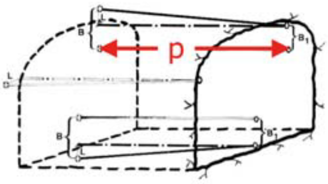
Tunnel driving by drilling and blasting consists of breaking and removing in succession roughly cylindrical volumes of rock.
The cross section of removed volumes is the excavation cross section, and the length is the pull of the round.

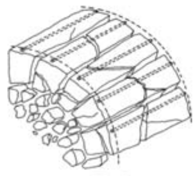
The volume to be blasted is the product of the cross section S by the pull s.

$$V = S \times p$$



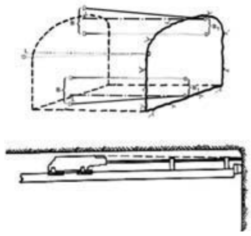
A round is a multiplicity of charged holes, cooperation to the aim of breaking and dislodging (blasting) said volume, that later will be removed by the mucking machinery, making free a new excavation face, displaced by the length of the pull in advance (with respect to the previously drilled and blasted one) in the tunnel driving direction.

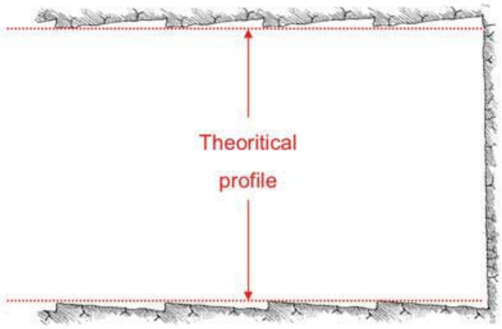




Look-out

The look–out of the contour holes allows to keep constant the excavation cross section in the successive round

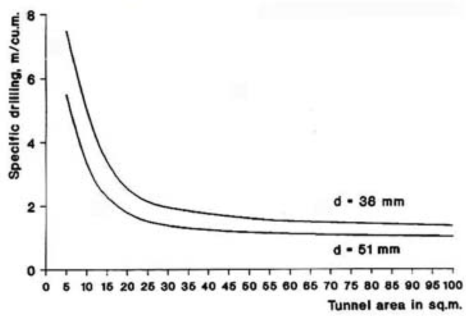




Some essential parameters

- **Powder Factor** (specific consumption of explosive) → $PF = \frac{Q}{V}$ (kg/m³) — *quantity of explosive*
- **Drilling density** → $D_1 = \frac{n_{holes}}{S}$ (holes/m²)
- **Specific drilling** → $SD = \frac{L_{TOT\_dr}}{V}$ (drm/m³) — *lunghezza totale di drilling*

The overall drilled length for a round is approximately the product of drilling density (holes/m²) by the blasted volume.

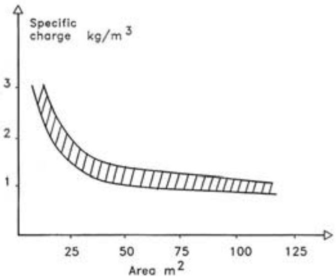


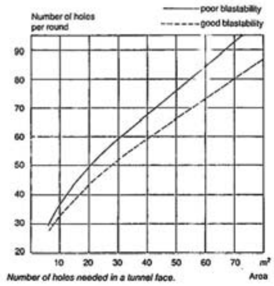
Being, approximately, the average length of the holes equivalent to the pull p, the total charge of the round is:

$$Q_{TOT} = PF \cdot S \cdot p$$

formule alternative

The P.F. is greater than in a surface blasting operation, and depends on the **excavation cross section**, more markedly than on the rock properties: the smaller is the cross section and greater the is the specific explosive consumption, with the rock mass being the same.

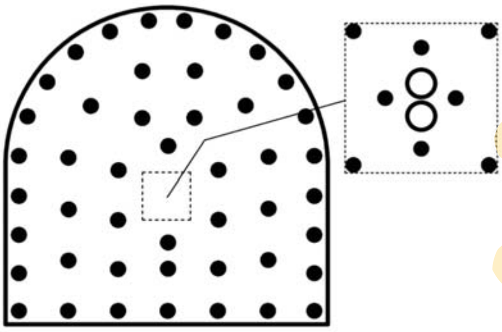




$$PF = 0.8 + \frac{12}{S}$$

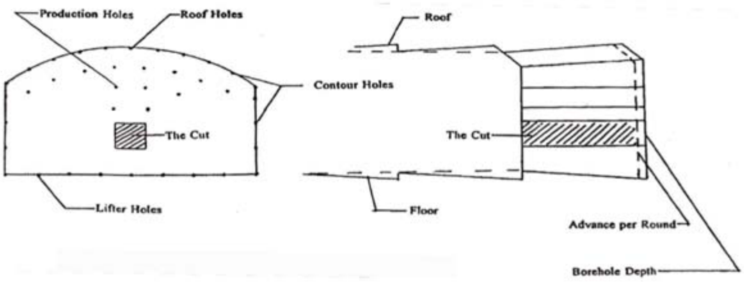
$$D_1 = 1.2 + \frac{25}{S}$$

THE CUT



The face is seen by the charge as a single free surface; it is necessary, therefore, to create further free surfaces, for efficient blasting: this is the **attack stage** of the blast. Then, the cavity obtained must be progressively enlarged by further charges, favorably located with respect to the free surfaces obtained in the previous stage, until the desired contour is reached: this is the **detachment stage** of the blast.

The selection of the cut type is dictated by the available **drilling machinery**, by the **tunnel width** and by the **intended pull** (advance per round)

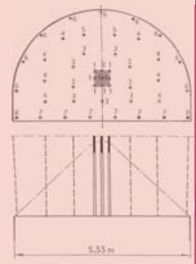


Cut types

- **Parallel holes cut**: can provide a longer pull in small-medium cross section tunnels
- **Inclined holes cut**: the double, 3 and 4 V cuts are effective, especially in large excavation cross sections

In tunnel driving, the group of holes producing the initial opening (**cut**) is necessarily very tight: it has to be considered the possibility of unwanted simultaneous firing of charges in different holes by flashover, that means that the charge does not explode thanks to its ignition device, at the correct time, rather under the impulse transmitted through the rock by a nearby exploding charge.

The charges **must fire in sequence**, not simultaneously: simultaneous explosions can "freeze" the blast (rock compaction without dislodging)

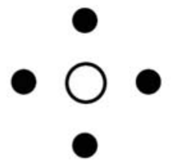


The type of round has to be chosen on the basis of available machinery, but more important is the cross section.
Crater cut (also termed "no cut round"). It is seldom used, and only for small cross section tunnels. The initial opening is a conical crater, with horizontal axis, created by one or more charges simultaneously detonated. In order to obtain a long pull, a great amount of explosive has to be used.

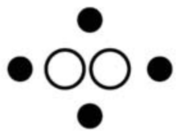
- ***Parallel holes cut, with dummy holes (CANADIAN CUT; SWEDISH CUT etc.)***

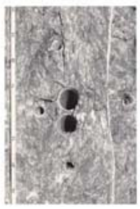
  Used mainly for narrow tunnels.

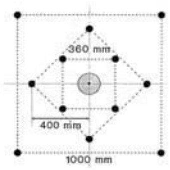
  One or more charged holes are placed at a small distance from **uncharged** (dummy) holes, to crush the rock and eject it axially, producing a cylindrical cavity, whose length equals the pull.

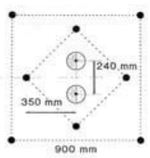


  The **pull** obtainable depends practically only on the **diameter of the dummy hole**. Moreover, the larger is the diameter of the dummy hole, the greater the allowed **distance** to the charged holes, and lower the number of holes to be drilled. It is always necessary to take into account the volume increase of the blasted rock: each charged hole, when detonated, must find enough free volume (laterally) to accommodate for the volume increase of the broken rock.
  The **vibratory disturbance is quite high**, being the free surface very small at the beginning of the round detonation.

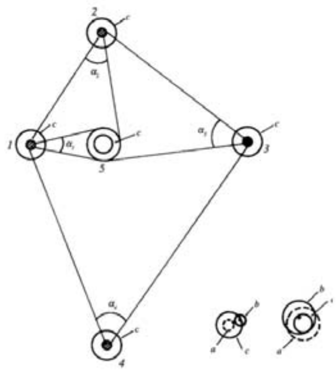








- ***Spiral cut***



- ***Inclined holes cut*** → holes drilled at a certain angles