Tunnel construction and ground reinforcement: general aspects

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The excavation of the tunnel causes a redistribution of the natural stresses inside the rock mass around the hole.
The instability conditions that can occur in and around a tunnel depend on the type of soil or rock in which the excavation is being done and on the natural (preexisting) state of stress in the rock mass.

### Examples of collapse
The boundary displacements and the stability of a tunnel is always a tri-dimensional problem and as such it should always be studied and analyzed.

Hoek, 2002

The place inside of the tunnel, where it is necessary to act to obtain the stability of the void, is always the last section of the tunnel that has just been excavated and where neither the walls nor the face have still been supported.

Bieniawski, 1987
The just excavated section of the tunnel must remain stable for the time necessary to carry out all the operations subsequent to blasting, which means mucking (removal of blasted rock), scaling and support installation works.

This stretch of the tunnel is usually known as “unsupported span” and the corresponding stability time is known as “characteristic time” or “self-bearing time”.

The self-bearing time is a necessity of the excavation of a tunnel: the natural stability of the unsupported span is requested in order to have enough time for mucking, scaling and support excavation.

The stability conditions of a tunnel are controlled by

✓ Ground geotechnical properties
✓ Natural (pre-existing) state of stress
✓ Shape of the tunnel section
✓ Size of the tunnel section
✓ Construction methods
✓ Underground water
Ground geotechnical properties

The stability conditions are defined both by the strength of the rock in comparison with the natural and induced stresses around the void and by the material deformability, that can show an elastic, brittle, plastic, soft strengthening, etc. behavior.

Natural state of stress

The in-situ stresses locked in the rock mass can depend from the geostatic and gravity forces (nature of the material and depth of the excavation, i.e. overburden); tectonic; gestructural-morphological conditions.
The opposition to the rock mass tension undertaken by the supports determines the stability conditions for the tunnel.

Shape of the tunnel section

This influences the rock mass tension concentration around the void (edges are points where the tension concentration is maximum). The excavation section shape can vary from rectangular (best use of the free section in road and railway tunnels) to circular (more homogeneous induced tensions), up to polycentric (in order to get the best advantages from both the approaches).

Size of the tunnel section

This influences the ask for resistance by the supports, both short-term and long-term ones.
**Construction method**

This influences the scheduling for the support installation or reinforcement execution (that can be prior or in concomitance of the excavation), that hence influence the convergence (that has to be mastered) and therefore the ask for resistance by the supports (short-term or long-term ones).

**Underground water**

It is a fundamental parameter for the tunnel construction, that has to be studied in a deep detail. The interference of the excavations with the underground water shows many aspects: natural and induced tension field modification, soil alteration, nasty interferences on the working operations and environmental impact.

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**Ground geotechnical quality**

(Hoek and Brown, 1980)

- Overburden soil or heavily weathered rock
  - squeezing and flowing ground, short stand-up time.

- Blooky jointed rock partially weathered
  - gravity falls of blocks from roof and sidewalls.

- Massive rock with few unweathered joints
  - no serious stability problems.

- Massive rock at great depth
  - stress induced failures, spalling and popping with possible rockbursts.
The tunnel must be stabilized using supports generally stiff and continuous.

Often presupports and/or ground improvements techniques are necessary.

Overburden soil or heavily weathered rock
Squeezing and flowing ground, short stand-up time.

Fractured rock mass with rock blocks not stable.
Systematic supports are necessary.

Sometimes presupports and/or ground improvements techniques are necessary.
Massive rock with few unweathered joints
no serious stability problems.

The tunnel is stable.
Local supports may be necessary.
Massive rock at great depth
stress induced failures, spalling and popping with possible rockbursts.

Continuous supports to control the displacements of the tunnel are necessary.

Apart from surface tunnels in loose soils whose behavior is determined by cohesion and internal friction of the ground, the most frequent and complex case that presents a great variety of aspects is that of tunnels in materials which undergo a plastic conditions due to the stress re-distribution around the tunnel after the opening of the void.

The opening of the tunnel determines the movement of the rock mass towards the void and the excavation and stabilization methods are to be designed in such a way as to put the deformations under control.
Excavation methods can be divided in four main groups

- **Conventional cyclic method using Drill and Blast.** The excavation can be carried out full face or parzialized face sub-dividing the face in smaller separate attacks, with or without preventive reinforcement of the ground;

- **Conventional cyclic method using punctual excavating machines** (roadheader, high energy impact hammer, mechanical excavator, etc.). The excavation can be carried out full face or parzialized excavation sub-dividing the face in smaller separate attacks, with or without preventive reinforcement of the ground;

- **Full face mechanised continuous excavation method using Rock TBM** for the excavation of tunnels in rock. The main problem is to break the rock;

- **Full face mechanised continuous excavation method**, using mechanised shields and with counter-pressure against the face for the excavation of tunnels in soil above and below the water table. The main problem is the stability of the tunnel as well as the control of the groundwater.

When the unsupported length is too short it can be changed through the following four methods:

1) to reduce of the excavation section into smaller portions for parzialised face;

2) to apply a counter pressure against the face;

3) to improve or reinforce the ground properties;

4) to use preventive supports of the rock mass installed ahead of the face.
1) To reduce of the excavation section into smaller portions for parzialized face.

In smaller face the control of the stability is easier, the characteristic time is shorter, smaller amount of waste rock is to be removed and fewer supports have to be installed.

![Diagram of excavation head and bench with steel pipe umbrella](image)

Extraction head and bench with steel pipe umbrella
1) To reduce the excavation section into smaller portions for parzialized face.

Classification between the various possible multiple headings excavation and the rock mass quality

Hoek, 2000
2) To apply a counter pressure at the face.

- mechanized tunnelling

Ground and water pressure acting on the face =

Counter-pressure inside the bulk chamber

Pressure transducers inside the bulk chamber

Maidl et al., 1994

- conventional tunnelling

BO-ST High speed railway line (Italy) - CAVET Brossure

Full face excavation and face reinforcement
3) and 4) Improving the ground properties and pre-supporting the tunnel free span.

The main problem, when tunneling through difficult geotechnical conditions with conventional methods, is the control of deformations. Without support or reinforcement the ground plasticizes and tends to move towards the opening:

- fall of ground from the upper part of the tunnel face;
- displacement of tunnel boundary;
- tunnel face extrusion and failure.

To prevent this phenomena of the ground around the tunnel, it is necessary to use “pre-confinement technique” (defined as any action that favours the formation of an arch effect in the ground ahead the tunnel face) or to improve the ground properties.

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**DESIGN APPROACH**

- Clear identification of project requirements
- Special studies to resolve particular problems related to the unique features of a project
- Do risk analysis – assess initial and residual risks
- Selection of the optimum construction solution with responsible-reliable estimates of cost and duration
- Use systematic risk management to allow for safe, cheap and reliable technical solutions

Grasso, 2010
Tunnel construction and ground reinforcement: various technologies

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The following main types of reinforcement can be identified with reference to the carrying out techniques:

- IMPROVEMENTS
- REINFORCEMENTS
- PRE-SUPPORTS
- DRAINAGE SYSTEMS
IMPROVEMENT

Methods which improve (from the engineering point of view) the mechanical or hydraulic properties of the rock mass: injecting fluids or freezing the fluids already present in the ground.

- injection (at low pressure)
- jet grouting
- freezing
The most applied grouting methods are:

- in rock - ascending and descending stage methods;
  - multipacker method;
- in soil - sleeved pipe system (tubes à manchette).
Naples metro – Fractured tuff

**REINFORCEMENT**

Methods which use the insertion, inside the rock mass, of structural elements with one dimension prevalent

- Bolts
- Micropiles
- Cable bolting

Serena Tunnel, Italy
Ground reinforcement with bolts and tendons

Borzoli Cavern (Italy) (Grasso and Pelizza, 1998)

Ground reinforcement with micropiles from the surface
PRE-SUPPORT

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 is carried out.

- mechanical precut
- pretunnel
- steel pipe umbrella
- arch of microtunnels

Courtesy Geodata S.p.A., Torino

PRE-SUPPORT

If the improvement techniques are used to create a reinforced zone around the tunnel and ahead the tunnel face

The global action must be considered as the most important for tunnelling. Therefore this intervention can be classified as a pre-support.

- jet-grouting arch with or without reinforcement
- consolidation of the tunnel profile: reinforced with VTR elements
- low pressure injection of the ground around the tunnel
MECHANICAL PRECUT

STEEL PIPE UMBRELLA
ARCH OF MICROTUNNELS

Sottopasso della Highway 285 (Atlanta)

Stazione ferroviaria Almaeda (Lisbona)

JET – GROUTING ARCH WITH OR WITHOUT REINFORCEMENT OF THE COLUMNS
LOW PRESSURE INJECTION OF THE GROUND AROUND THE TUNNEL

Seikan Tunnel grouting scheme to cross fault zones (Hashimoto and Tanabe, 1986)

DRAINAGE

Technologies which take away water from the rock mass or the ground in a controlled way.
Key: ● Applicable. ○ Applicable with special intervention: 1 – chemical grout; 2 - two or three-fluid jet grouting; 3 - steel rebar or pipe reinforced jet grouting; 4 – active dewatering (vacuum pump required); 5 – additional grouting; 6 – high resistance element; 7 – additional grouting.

The interventions listed in this table can be combined in order to guarantee safe tunnelling conditions in almost all geotechnical conditions. Grouting, jet-grouting, freezing and dewatering can be normally be applicable also when tunnelling under water table. The other interventions when the tunnel is under the water table must be combined with impermeabilization techniques.
IMPROVEMENT OF THE CONVERGENCE – CONFINEMENT CURVE

METHODS WHICH MODIFY THE CONVERGENCE – CONFINEMENT CURVE

Radial displacement

IMPROVEMENT OF THE CONVERGENCE–CONFINEMENT CURVE

METHODS WHICH MODIFY THE RADIAL DISPLACEMENT AT THE FACE

Radial displacement

INTERNAL PRESSURE

REACTION LINE OF THE SUPPORTS

REDUCTION OF THE DISPLACEMENT AT THE TUNNEL FACE

INTERNAL PRESSURE

u₀

u₀

METHODS WHICH GUARANTEE THE STABILITY OF THE FREE SPAN AND OF THE FACE

Tunnel construction and ground reinforcement: steel pipe umbrella

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Steel pipe umbrella or forepoling is a pre-reinforcement technique which is obtained by installing steel pipes ahead of the tunnel face.

The steel pipes usually have a dip of 5°-10° (with reference to the horizontal) in a way as to form an umbrella with a truncated cone shape and which allows the overlapping of two adjacent fields.

3D view of steel pipes
It is possible to cover advance lengths of 12-15 metres, of which 9-12 metres are of excavation, account being taken of the necessary overlap between two sets of pipe umbrella to guarantee the stability of the face.
Steel pipe umbrella

- DO NOT MODIFY the convergence-confinement curve
- DO NOT MODIFY the position of the reaction line of the supports

Guarantee the stability of the tunnel free span

Steel pipe umbrella: Example of application

Cross-section of Lonato tunnel (Italy) where steel pipe umbrella has been used in a morain. The excavation has been carried out heading and bench.

Key: A: steel pipes; B: twin steel arches; C: shotcrete; D: electro-welded mesh; E: micropile; F: jet grouting column; G: drainage pipe; H: concrete lining
Some Italian examples

Example of Steel pipe umbrella in a morenic debris
Torino – Freus Tunnel Highway (Italy)
Example of Steel pipe umbrella in a morenic debris
Aosta-Mont Blanc highway (Italy)

Example of steel pipe umbrella to cross a fault in rock at great depth
Water conveyance tunnel - Pont Ventoux Hydroelectric power plant (Italy)
Example of steel pipe umbrella in a very fractured rock mass at low depth
Cossato road tunnel (South entrance) - Italy

Example of steel pipe umbrella in a strong clay (argillite) with rock blocks
Drainage tunnel below San Lorenzo Tunnel (Italy)
Example of steel pipe umbrella in a sand and gravel
Torino Metro (Italy)

Example of steel pipe umbrella and jet grouting face reinforcement in sand
Cossato Tunnel (north entrance), Italy
Example of steel pipe umbrella and face reinforcement with VTR in clay at low depth Milieu-Gaurain Tunnel (Belgio)

Portal of Doria Tunnel (Genova, Italy) (Courtesy Geodata S.p.A.)
Example of steel pipe umbrella with Rock TBM tunnelling
Abdalajis Tunnel (Grandori e Romualdi, 2004)

Example of steel pipe umbrella with Rock TBM tunnelling
Stariano water conveyance tunnel (Italy)
FOREPOLING
In many applications the pre-support is made using bolts or steel bars. Many applications have been carried out using self-drilling bolts.
FOREPOLING

(Germany)

Simplified design approaches

To determine the required steel pipe section, the pipe is considered to be a continuous beam on two or on multiple supports (steel arches) embedded in the ground ahead of the excavation face.

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.
The acting load on the pipe \([q]\) can be evaluated, starting from the value of the maximum vertical stress
\[
q = p_v i
\]
where "\(i\)" is the spacing between the pipes.

One of the problems is the evaluation of the vertical stress that is acting near the face. In many cases, it is empirically assumed that \(p_v = 0.50-0.75\) of the total vertical load before excavation and the load is evaluated by the well known formulation of Terzaghi.
The length ahead of the tunnel face which is not acting as support of the pipes (g) is usually empirically chosen and very often the value of 0.5 m is assumed. This value is, obviously, directly linked to the geomechanical properties of the ground and to the presence of tunnel face reinforcements. More detailed research must be developed for a complete and final definition of this length.

Considering that the support action of the pipes must be developed for a short period of time before the tunnel support are installed (steel arches and shotcrete), the admissible working stress of the steel of the pipes can be close to its field stress (1.5>F_s>1.1). Knowing the acting stresses it is possible to chose the type of pipe.

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;

The length of the overlap between two subsequent umbrella is controlled by the behaviour 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.

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.
The described empirical 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 arches and ground);
- the effect of the ground ahead of the face;
- the own bending stiffness of the steel pipe.

To take into account these aspects it is possible to use more complex design schemes:
- a model based on the approach of a beam on multiple supports.
- numerical models (tri-dimensional) which must be used in very complex problems (where for example it is necessary to know the induced settlements exactly)
Beam on multiple supports

The less rigid it is the stiffness of the ground the higher the moment near the face is

i=1m ; E = 2500 kg/cm²

Analysis of the influence of the stiffness of the ground on the bending moments of the steel pipes

The less rigid it is the stiffness of the ground the higher the moment near the face is
Analysis of the influence of the stiffness of the ground on the displacement of the steel pipes

From the previous presented results it can be put in evidence that is important the good contact between the steel arches and the pipes: Few centimeters of displacement of the pipes can cause the collapse of the structure or induce critical displacements of the surface

The steel arches must not move ⟷ need of a good foundation.

In special cases it is suggested to use the bullflex pillow to guarantee the contact between the pipes and the steel arches

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 design of foundation procedures are usually used
BULLFLEX SYSTEM

Problem of contact between the steel arch and the pipes: Bullflex pillow
Filled with shotcrete mix
DESIGN ASPECTS: Numerical modelling

3D numerical modelling where the pipes are singularly modelled
- problem
- complexity of the numerical model and time of computation

2D numerical modelling
some authors proposed to use a ground reinforced arch around the tunnel
to model the action of the pipes

Problems
- it is difficult (or impossible ?) to correctly define the improved parameters
- it is difficult (or impossible ?) to correctly define the correct stress release
- does the pipes act as an arch ?
Numerical modelling

Tunnel construction and ground reinforcement: jet grouting arch

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Documents pédagogiques internes au Mastère TOS

1. drilging with casing
2. hole cased to depth
3. insertion downhole of jet-grouting tool and retrieval of casing
4. commencement of jet-grouting
5. rotary drill rod extraction while jet-grouting
6. completed grouted soil column

JET COLUMNS FOR
- ARCH
- CONSOLIDATION
- JET COLUMNS FOR HEADING STABILITY
Turin railway junction

Soil improvement for a tunnel. Most critical section. (Manassero, 1993)

In this case were used jet grouting umbrella for the excavation of the drift, a permeation grouting to reinforce the ground around the tunnel, a steel pipe umbrella and jet-grouting columns to reinforce the tunnel wall.
Design of a jet grouting intervention

- the static design of the structure as a whole with the analysis of tunnel stability. This step should lead to the design of the geometry of the treated ground.

- design of the injection parameters, of the type of grout and of the operational parameters;

- design of an experimental test site and the definition of the control parameters.

Design of operational parameters

**Preliminary site investigation and testing**

The preliminary study of any geotechnical problem demands a thorough site investigation to enable the most convenient solution to be reached. If a stabilizing treatment is necessary, an accurate design may require supplementary testing that is specifically related to the solution proposed. In comparison with conventional grouting requirements, the factors that affect the feasibility and the selection of jet grouting parameters can be assessed by a less engaging experimental program, which may be summarized as follows:

- detailed soil profiles and general hydro geological information
- simple in-situ tests such as cone penetration or SPT, to estimate soil consistency or relative density
- simple laboratory tests on representative soil samples, to evaluate grain size distribution of cohesionless materials and water content, bulk density, properties of cohesive formations
- laboratory tests on trial grout and soil-grout mixtures, to be defined according to the importance and specific requirements of the work
- in-situ jet grouting tests to check the operational parameters and to a larger extent, if necessary, to provide more detailed information for the final design.
**Design of operational parameters**

**Geometry of treatment**

The great flexibility of the jet grouting procedures allows various problems to be solved by suitable geometrical patterns such as:

- continuous strip treatment by one or more rows of vertical overlapping elements to form cut-off walls for ground water control or earth-retaining structures. Such barriers may have a circular or elliptic form, for instance when required to protect deep shaft excavation.
- block treatment by vertically staggered columns to increase bearing capacity of foundations or to improve mechanical properties of soils in tunnelling problems; if conditions permit the treatment is done from the surface around the periphery of a planned tunnel or extended to the entire area to be excavated.
- sub-horizontal treatment ahead of the excavation face in deep tunnelling, when operations from the surface are impossible or not convenient.

**Selection of the grout**

The grout mix constituents and composition can be varied to meet the specific requirements for strength and permeability with quite different and less restrictive criteria in comparison with conventional injection. As regards the initial rheological properties, viscosity and rigidity should be fairly low in any case to allow a uniform treatment to the greatest extent.

Where strength is the main design criterion a simple cement slurry is employed, with a cement/water ratio C/W mostly ranging between 0.5 and 1.0 to be selected according to various factors besides the required strength, such as:

- the type of soil as regards grain size and permeability in general and water content in cohesive formations.
- the estimated amount of grout per unit volume of treated soil.

In permeable granular formations a considerable amount of water may be drained out both from the soil and the grout, while in a cohesive soil of low permeability the final water content may nearly equal the sum of the two original contents.
Design of operational parameters

Selection of jet-grouting parameters

The influence of nozzle diameter, pressure, type and quantity of grout, monitor rotation and withdrawal speed have been widely investigated in various soils and hydrological conditions.

According to previous experience on job-sites and in field trials, the orders of magnitude of the main parameters for the monofluid are:

- pressure: 30 to 50 MPa
- nozzle diameter: 1.8 to 3 mm
- rod rotation speed: 10 to 20 rpm
- rod drawing up speed: 20 to 70 cm/min.

A suitable selection of these parameters requires practical experience and may demand site trials.

Influence of water contents of grout and soil on the required quantities of grout and cement (Perelli Cippo e Tornaghi, 1984).
Design of operational parameters

<table>
<thead>
<tr>
<th>System</th>
<th>Fluid</th>
<th>( V_1 ) (m/min)</th>
<th>( V ) (m³/min)</th>
<th>( Q ) (m³/h)</th>
<th>( P ) (MPa)</th>
<th>( E ) (MJ/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>single fluid</td>
<td>mix</td>
<td>15</td>
<td>35</td>
<td>0.2</td>
<td>0.3</td>
<td>3.6</td>
</tr>
<tr>
<td>two fluids</td>
<td>mix</td>
<td>4</td>
<td>18</td>
<td>0.5</td>
<td>1.5</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>air</td>
<td>10</td>
<td>100</td>
<td>6.5</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>three fluids</td>
<td>mix</td>
<td>10</td>
<td>100</td>
<td>6.5</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>water</td>
<td>10</td>
<td>100</td>
<td>6.5</td>
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<tr>
<td></td>
<td>air</td>
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<td>100</td>
<td>6.5</td>
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</tr>
</tbody>
</table>

\( V_1 \) = withdrawal speed  
\( V \) = mix volume  
\( Q \) = grout mix flowrate  
\( P \) = grout mix pressure  
\( E \) = grouting energy

Properties of treated soil

The results of any treatment, in terms of uniformity and mechanical properties, depends on a number of interconnected factors concerning the soil and the jet-grouting parameters.
Design of operational parameters

Column diameter for the monofluid (T1) bifluid (T2) jet grouting procedure data (Botto, 1985).

Tunnel construction and ground reinforcement: face reinforcement

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Tunnel face ground reinforcement is obtained by installing on the tunnel face fibre glass elements (such as pipes or plaquettes) fully grouted and therefore connected with the soil/rock mass.
Tunnel face reinforcement elements

- YTR element
- Manchette pipe for grouting

Tunnel face reinforcement elements

- T SOLD ORD
- "T" SHAPE ROD
- PLAT STRUCUTRAL ELEMENTS TO BE ACCORDING TO DIFFERENT POSSIBLE TYPOLLOEGS
- resistant area
- spacer
- grouting pipe
Ground reinforcement used for the construction of the Bo-Fi highspeed railway (Italy)
Example of some scheme of ground reinforcement using longitudinal fiber glass reinforcement in Tartaiguille tunnel (France) (Lunardi, 2000)

Example of Tartaiguille tunnel - (France) (Lunardi, 2000)
Example of Tartaiuguille tunnel - (France) (Lunardi, 2000)

Example of reinforcement scheme that was adopted to cross the Daj Khad shear zone in the Headrace tunnel of Nathpa Jhakri Hydroelectric Project - India (courtesy Geodata S.p.A.)
Example of Reinforcement scheme that was adopted to cross the Daj Khad shear zone in the Headrace tunnel of Nathpa Jhakri Hydroelectric Project - India (courtesy Geodata S.p.A.)

Tunnel near the surface - collapse mechanism: sliding of the ground

Failure bulbs for different ratio diameter/depth (Chambon and Corté, 1990)
Influence of tunnel unlined length on extent of failure mechanism for C/D=4 (Chambon and Corté, 1990)

Tunnel near the surface - collapse mechanism: sliding of the ground

Deep tunnel - collapse mechanism: estrusion
Deep tunnel - collapse mechanism: estrusion

Lunardi, 2000

Tunnel face below the prereinforcement - collapse mechanism: sliding of the ground
**DESIGN PROCEDURES**

- Modeling of all the reinforcement element in a 3D numerical model
- Modeling the action of the face reinforcement as an applied pressure
  - Numerical model (3D, Axi-symmetric)
  - Analytical model (sliding body)
- Modeling the action of the face reinforcement as an improvement of the geotechnical properties of the ground of the core
  - Numerical model (3D, Axi-symmetric)
- Modeling the action of the face reinforcement as an element intercepting a sliding surface and acting thanks to its shear properties
  - Analytical model (sliding body)

**Surface and deep tunnels**

**Surface and deep tunnels**

**Surface and deep tunnels**

**Surface tunnels**

face stability below the pre-support

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**Evaluation of face reinforcement action**

Temporary support pressure on the face

\[
\sigma_1 = \min \left[ \frac{4N_b A \sigma_b}{\pi D^2}; \frac{4N_b S \tau_a}{\pi D^2} \right]
\]

Pells, 1994

where:

\( Nb \) = number of VTR pipe; \( A \) = cross section of the VTR pipe; \( S l \) lateral surface of the pipes, \( \tau_a \) = shear stress on the lateral surface of the pipe; \( \sigma_b \) = yielding stress of the pipe material
Evaluation of face reinforcement action

Improvement of cohesion of the ground

\[ c^* = c + \frac{1}{2} \cdot \cos \phi \cdot \left( \frac{\sin \phi}{\cos \phi} \right) \]

\[ \Delta \sigma = \frac{n \cdot T_{\text{max}}}{S} \]

\( T_{\text{max}} \) = max force of sliding between the reinforcement and the ground

FLAC 3D

Cohesion increment

Rock reinforcement (a)

Applied pressure (b)

Modelling of the single pipes (c)
Horn model (1961) which was assumed by Anagnostou and Kovari (1994,1996) as the base for the stability analysis of the face ahead of Slurry Shield and EPB machines but can be used for surface tunnel design.

Evaluation of the forces in the nails based on Soil nailing approach (Raccomendation Clutèrre, 1991)
Example of 3D numerical computation with modeling of the elements

Contour lines of total displacement for overburden of H/D=0.5 for different length of reinforcement (Schweiger and Mayer, 2004)

Tunnel construction and ground reinforcement: example of a reinforcement ahead the face with self-drilling bolts in a complex urban environment

Sebastiano Pelizza - Daniele Peila
POLITECNICO DI TORINO
Grouting from the tunnel in urban area
(Turin metro – lot 1 – Conventional excavation)

Self-drilling bolts ($\varphi = 38$ mm) to inject silicate resins (2 components)

Steel arches
IPN 160 double
+ bullflex

Documents pédagogiques internes au Mastère TOS
Drilling and installation of self drilling bolts

Excavation with HEIH
Tunnel construction and ground reinforcement: innovative soil reinforcement method in unstable silty sand and clay

Sebastiano Pelizza – Claudia Mignelli - Daniele Peila
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The Serra dell’Ospedale Tunnel

Double-tube motorway tunnel located in the South of Italy on the motorway Salerno-Reggio Calabria
The Serra dell’Ospedale Tunnel

North bound 1038 m
South bound 938 m

- existing tubes
- new tubes
- artificial tunnels

GEOLOGICAL MAP

- PL1: Sandy silts
- PL2: Silty sands and fine sands
- PL3: Silty clays and clayey silts
Longitudinal geotechnical profile, Serra dell’ Ospedale Tunnel

Typical geotechnical soil layout

Debris
SAL Silty sands
SA-SAG-SAF Fine sands
LIM Sandy silts
ARG Silty clays and clayey silts

Sands Complex

SAF
SAL (yellow)
SAL (yellow-gray)
SAG (light yellow)
- the sands show a rather different behaviour, even in presence of similar granulometrical mixtures

- the sands, if present at the face inside the layers or lenses in the clayey-silt, constitute a somewhat precarious support foundation for the clayey formations, which often result instable because of lack of support

- sands with silty intercalations, have an unpredictable behavior

- the silt shows good holding characteristics

- the silty-clayey soil has a role that is connected to the deformability of the excavation face

This situation is made worse by the particular and unforeseeable behaviour of the water infiltrations at the face

SOUTH BOUND – SOUTH PORTAL (Reggio Calabria)
**Main technical soil characteristics**

- spallings of the face
- collapse of the inadequately supported pieces of cohesive layers
- repeated collapse, because of the flow of sand between the umbrella pipes, with the formation of chimneys some of which reached the ground surface

**Soil reinforcement technical measures**

- self drilling steel pipes on the back jet grouted with swelling cement mortar
- self drilling steel pipes on the face jet grouted with cement mortar

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**Consolidation of the excavation face**

- a transversal section and longitudinal profile of the “umbrella” at the back face and the consolidation of the excavation face

**The novelty of this new method:**

all the pipes on the back and at the face are **self-drilling steel pipes**
self-drilling pipes with a simple drilling bit, suitable for sand and clay, in which the cement mortar injection nozzles were inserted.
a ground column jet grouted on the face at 200 bar through the 60.3 mm diameter self-drilling pipe

Ground reinforcement and stabilisation system

“umbrella” at the back face: the 110.6 mm self-drilling pipes jet grouted with swelling cement mortar

the face reinforcement: 60.3 mm self-drilling pipes jet grouted with cement mortar
The drilling pipes were first bent by the excavator and then cut off with a robust scissor supported by a trucked carrier and then carried away.

<table>
<thead>
<tr>
<th>Operation</th>
<th>Dimensions</th>
<th>Quantity per cycle</th>
<th>Execution time (*)</th>
<th>**</th>
<th>**</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pre-lining of the back face (one self-drilling casting)</td>
<td>110,6/8 12 78 48 (**</td>
<td>)</td>
<td></td>
<td></td>
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<tr>
<td>Reinforcement of the face (self-drilling microcasting)</td>
<td>60,3/8 18 80 72 (**</td>
<td>)</td>
<td></td>
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</tr>
<tr>
<td>Excavation and support erection on excavation soil (6 m, 1 m step by step) steel ribs (1 m spaced) shotcrete</td>
<td>150 m/m 2 IPN 180/1.0 m 25 cm (7.5 m³/m)</td>
<td>54</td>
<td></td>
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<tr>
<td>Excavation of the invert (to 6 m) and concrete casting</td>
<td>15 m³/m 15 m³/m</td>
<td>12 12</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**TOTAL TIME PER CYCLE** | **198** |