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Tunneling Through the Rock-Soil Interface

Werner BILFINGER[a,1]
[a] *Vecttor Projetos Ltda*

Abstract. Constructive methods, tunnel lining, and soil treatment and conditioning depend fundamentally on ground and hydro-geological conditions. Regardless of specific difficulties, construction in homogeneous ground becomes, after adequate definition of the parameters above and the initial learning curve, a repetitive and uniformly controlled process, either through conventional or mechanical tunneling methods. Difficulties often arise when varying ground conditions are encountered along the tunnel alignment, especially if ground behavior presents significant contrasts in deformability, shear strength and permeability. In geological environments where soft ground overlays rock and tunnels have to be built crossing this interface, the above-mentioned contrasts normally occur at the same location. The most significant recent tunnel failures in Brazil occurred close to rock soil interface, showing the necessity of a review of current design and construction practice This paper intends to discuss main challenges associated to the rock-soil interface in the light of recent tunnel failures and present suggestions for robust design and construction methods.

Keywords. Soil Treatment, Rock-Soil Interface, Constructive Methods, Design

1. Introduction

In tunnel construction, geological-geotechnical contrasts along tunnel alignment are among the main challenges. Changes in material behavior, shear strength, deformability and permeability have often led to problems, including reduction of production rates, need for additional and unexpected treatments, and failures.

This paper presents the main topics associated to design and construction, focusing on the soil-rock interface. Although most of the discussed topics are known and available in the literature, it is important to revisit these topics in the light of recent tunnel failures.

The paper is focused on tunnels built using the so-called NATM or SEM (Sequential Excavation Method); however, the discussed concepts may also be used for mechanically excavated tunnels.

2. Background

Unfortunately, few publications discuss failures and their causes in tunneling. Some compilations with case histories are presented in [1], [2], [3], [4] and [5]. When analyzing these databases, it becomes clear that a significant part of the published failures is associated to singularities and unexpected ground conditions. The soil-rock interface is

[1] Corresponding Author, Vecttor Projetos Ltda, Rua Cardeal Arcoverde 1749 cj. 103, São Paulo, Brazil; E-mail: werner@vecttor.com.br

one of these singularities, where often some critical conditions are encountered at the same place:

- great variability of materials (deformability, shear strength and permeability), with non-regular geometry;
- concentration of high permeability layers, leading to high water inflow and its associated problems;
- Necessity to change the constructive method: at the interface the constructive method has to be changed often from conventional excavation of soil (using excavators or road headers), to the necessity to excavate rock (or vice-versa), using drill and blast.

The shape and position of the soil-rock interface is often difficult to determine precisely prior to tunnel excavation. Figure 1 below present data from [2], showing the soil-rock interface as presented in the final design documents, as well as a re-interpretation based on additional site investigations performed after a failure at the Lausanne Metro in 2005. The failure, according to [2], is related to the difference in ground conditions between the design profile and the real site conditions, including a "pocket filled with water."

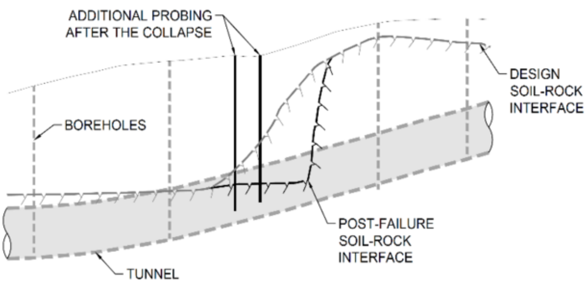


Figure 1. Longitudinal geological profile with soil-rock interface as presented in final design and after post-failure site investigations [2].

An analogous problem occurred during the construction of Salvador Metro: during the design phase, the soil-rock interface was analyzed using a number of boreholes. However, between two boreholes, an undetected depression reduced significantly the rock cover above tunnel crown, which led to a failure that, fortunately, did not progress to the surface.

Figure 2 presents an example of a soil-rock interface, reproduced from [6], as interpreted after the forensic evaluations of the failure of the Pinheiros Station, in São Paulo. It becomes clear that the often idealized straight line used to identify the soil-rock interface can be misleading and has to be adequately interpreted.

The difficulties of mapping the soil-rock interface is not limited to tunneling projects: in [7], a case history is presented where, due to an undetected paleo-channel, the design of a dam had to be changed from an earth-rock fill dam, to a concrete structure.

The different characteristics of the geo-materials normally lead to the need of using additional measures, when compared to the routine, to ensure tunnel stability, like installation of (additional?) soil treatments, reduction of excavation sections and groundwater lowering. The efficiency of different stabilizing measures has to be carefully evaluated. For example, a jet-grouted pre-lining, including a "plug", closing the pre-lining, could be considered a safe solution at first glance. However, minor defects in the pre-lining can lead to almost catastrophic situations, with significant water ingress into the tunnel, piping of loose material and surface settlement velocity of more than 120mm/day, as described in [8].

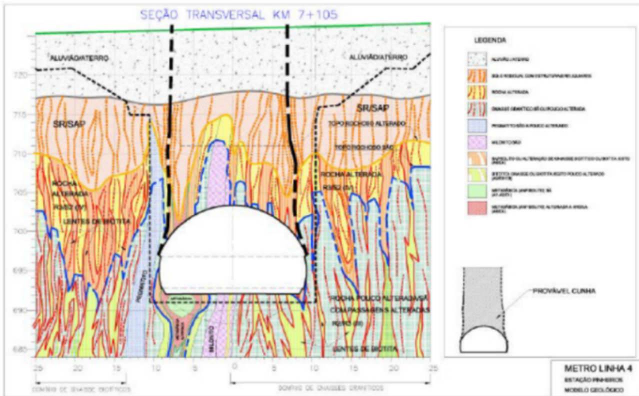


Figure 2. Geological cross section of the Pinheiros Station, reproduced from [6].

Therefore, the difficulties associated to the soil-rock interface can be divided into at least three different aspects:

- Mapping its position and shape;
- Identifying its characteristics;
- Identifying possible treatments and their limitations.

3. Site Investigations – State of Practice and Limitations

Site investigations are the tool to predict the geotechnical profile and geomechanical properties of its materials. Without the proper site investigations, the risk of not identifying important geological-geotechnical features increases and, therefore, of facing non-predicted behavior during and after tunnel excavation. Following general rules regarding site investigations are proposed in [9] and discussed by Parker [10]:

- Between 1.5 and 2.25 % of the construction cost should be spent with site investigations, and a contingency of up to 3% should be foreseen;
- For every meter of tunnel, between 0.75 and 1.2 m of boreholes should be foreseen. It is interesting to mention that, historically, in the Metro of São Paulo, the meter of borehole / meter of tunnel ratio is 1, in line with this recommendation.

These quantities have to be seen as broad guidelines, bearing in mind that every project has its particularities. Fookes, in [11], states very properly that "if you do not know what you are looking for in a site investigation you are not likely to find much of value."

3.1. Site investigation sequence

Site investigations influence tunneling projects in all its phases, starting at the feasibility studies, until actual tunnel construction. Site investigations are influenced by several factors, including, geology, hydrogeology and geomorphology, project characteristics and use, construction method and environmental considerations [12].

Usual site investigation types consist of:

- desk studies;
- field mapping;
- field investigations, including direct (trial pits, borings, in situ tests) and indirect investigations (geophysical methods), surveys and monitoring;
- laboratory tests, and
- exploratory investigations.

A suggestion of strategy to develop site investigations is presented in [12]. This strategy divides the tunnel design in three phases and associates 3 respective phases of site investigations:

- feasibility studies – first campaign, to assess the main ground conditions and identify major risks. Table 1 presents a summary of expected results and investigation means.
- preliminary design – second campaign, to quantitatively assess soil and rock behavior, and validate methods and sequence. Table 2 presents a summary of expected results and investigation means.
- detailed design – identify properties of ground units and reduce uncertainties. Table 3 presents a summary of expected results and investigation means.

The names of the phases may not be the same in different parts of the world, but the 3 phases approach can be considered conventional.

Table 1. Site Investigations for feasibility studies, based on ITA recommendations [12].

Expected results	Investigation means
Geological and hydrogeological maps	Regional topographic, geological, hydrogeological / groundwater, seismic hazard map
Natural risk maps, when appropriate	Information from field surveys and/or adjacent similar projects
Longitudinal geological profile	Geophysics may provide useful information.
Longitudinal geotechnical and geomechanical profile and identification of major hazards	Limited site investigations to confirm extremely critical geological or groundwater conditions
Preparation of risk register	

Table 2. Site Investigations for preliminary design, based on ITA recommendations [12].

Expected results	Investigation means
Longitudinal geological profile (1:5000 to 1:2000)	Geophysics and boreholes at portals and shafts
Longitudinal geotechnical-geomechanical profile (1:5000 to 1:2000) with ground behavior classes	Boreholes along the alignment
Geological and geotechnical cross sections at the portals (1:500 to 1:200)	Water sources and groundwater monitoring
Geological and geotechnical cross sections at access and ventilation shafts	Laboratory tests
Preliminary characterization of the hydrogeological regime	Outcrop and surface mapping
Update of risk register	In situ measurements and permeability tests, when appropriate
	Exploratory galleries / shafts, if needed

Table 3. Site Investigations for detailed design, based on ITA recommendations [12].

Expected results	Investigation means
Longitudinal geological profile (1:2000 to 1:1000)	Additional boreholes at portals and along alignment
Longitudinal geotechnical-geomechanical profile (1:2000 to 1:1000) with ground behavior classes	Laboratory and field tests
Geological and geotechnical cross sections at the portals and shafts (1:200 to 1:100)	In specific cases / locations, geophysics may provide useful information
Definition of detailed set of design parameters and their variability	Excavation of experimental sections along tunnel alignment, if needed
Detailed characterization of the hydrogeological regime	Continue the monitoring of water sources and groundwater
Update of risk register	

3.2. Desktop studies and field mapping

Desktop studies and field mapping are activities performed normally in the initial site investigation phases. The main objective is identifying materials that will be excavated, geomechanical classes and possible hazards that will be encountered along tunnel alignment. This information is obtained by the analyses of regional and local geology data, mapping of faults, shear zones and other possible discontinuities. Data about structural geology are also obtained. All this information is obtained from literature and /or in field mapping activities.

During these initial phases the soil-rock interface is interpreted normally with the main objective of defining the tunnel length to be excavated in different materials and to estimate quantities.

3.3. Direct investigations

Direct investigations evaluate type of material and measure soil and rock properties by the insertion of some type of tool into the ground. Material may be extracted or not. The most common direct investigations in soil are ([13], [14], [15]):

- Trial pits;
- Standard penetration test (SPT);
- Cone and Piezocone Test (CPT / CPTU);
- Vane test;
- Pressuremeter tests;
- Flat Dilatometer test (DMT).

Interpretation of the results of these tests can provide geomechanical parameters using different theoretical or empirical approaches [13]. Permeability may also be measured using constant head (Le Franc) or variable head tests in boreholes.

In rock, tests are usually associated to core drilling, complemented by other tests:

- Discontinuity measurements, with borehole video or acoustic televiewing. In the past the impression packer was used for this purpose, but the televiewing technique replaced it almost completely;
- Permeability measurements, using Lugeon tests with packers;
- Uniaxial compressive strength performed on recovered specimens.

Special tests, like hydraulic fracturing, may also be performed on boreholes, but are less common. A complete list of ISRM standardized – SM Suggested Methods – can be found in the “Blue Book” [16] and in the more recent “Orange Book” [17].

It can be seen that the direct investigations in soil and in rock are different and, exactly at the boundary between these materials, difficulties may arise in identifying and characterizing materials: SPT and CPT/CPTU tests may reach refusal to penetration before rock is actually reached. On the other hand, core drilling, when drilling close to the soil-rock interface, may not recover material and, therefore, no information may be obtained in this region.

3.4. Indirect investigations - geophysics

Indirect investigations aim to identify different materials and its properties of the subsoil profile without the necessity of drilling boreholes or driving probes. Most of the geophysical methods have the advantage of generation sections, in contrast to direct investigations, where only a vertical “line” is investigated.

Main geophysical investigation methods are ([18], [19]):

- Seismic – reflection, refraction, borehole seismic, surface waves;
- Gravity;
- Magnetics;
- Geoelectrics;
- Electromagnetics;
- Radar.

Geophysical methods focus mainly on profiling – preparation of geological profiles and geomechanical parameters are generally not obtained. Seismic tests measure seismic velocity of the different layers, which then can be correlated to stiffness and other geomechanical parameters.

In the experience of the author, for usual tunnel projects, seismic refraction and electrical resistivity tests are commonly used. To obtain more accurate dynamic soil parameters, cross hole or down the hole tests are performed. The use of the seismic refraction technique may be limited by external sound sources, typical in urban environment.

In [20] case histories are discussed, where different geophysical methods are combined to map the geology along tunnel alignment.

3.5. Discussion

The recommendations regarding site investigations presented in 3.1, as well as the available techniques, direct and indirect, show that there are several tools to investigate soils and rocks. However, the interface, which is normally not the idealized straight line, is often very difficult to investigate. Figures 2 and 5 present examples of more realistic shapes of the soil-rock interface. To investigate the interface, in the opinion of the author, a combination of methods is necessary:

- Initial geological assessment, to evaluate possible shapes, thicknesses, existence of boulders, geohydrological conditions, orientation of discontinuities, etc.
- Borings, focusing on maximum core recovery (large diameter, careful boring – limiting the use of water circulation). Orientation of boreholes should take into consideration the orientation of the discontinuities of the rock;
- Televiewing of boreholes, where boreholes are sufficiently stable;

- Geophysical investigation, calibrated using the borings, if possible, using more than 1 method.
- Recovered core samples should be inspected and tested in the laboratory.

After the site investigation campaign, which could be performed in phases, like the recommendation presented in 3.1, its interpretation should be equally careful. The interpretation should focus specially on:

- Identification of different materials;
- Identification of geohydrological conditions;
- Characterization of shear strength, deformability and permeability of the identified materials;
- Mineralogical characterization of the materials, evaluating the possibility of swelling or other deleterious properties.

It is, however, important to state that with current practices and technologies, it is not possible to preview with precision the location, dimension and particularities of the so-called soil-rock interface. In reality, the interface is not a line or plane, but a transition zone.

The use of probe-drilling during construction has been an important tool to improve the knowledge about the ground. More recent techniques, including seismic ([21],[22]) investigations from inside the tunnel are an evolution that may add information to the pre-tunneling geological-geomechanical model.

4. Lining and Support - concepts

There is no universal nomenclature that defines the system of elements that support the soil or rock mass, stabilizing the tunnel opening. In [23], primary support, also called lining, and secondary linings are associated to construction phases. The primary support is defined as having the purpose of "…to stabilize the underground opening until the final lining is installed". The usual elements of a primary support / lining are shotcrete (reinforced or not), rock bolts, steel ribs and lattice girders.

The definitions above encompass both tunnels in soil and in rock. There is, however, an important difference between tunnels in these materials: to stabilize the soil around a tunnel opening in soil, a shell-like structure is necessary, normally consisting of shotcrete and/or concrete. This shell stabilizes ground pressures as a structure – resisting to axial forces, bending moments and shear forces.

For tunnels in rock other ways to stabilize the rock mass are normally used, mainly rock-bolts, that make a part of the rock mass work as a supporting rock arch. This mechanism is only possible if the rock has a) sufficient strength to support the acting stresses and b) the rock bolts are adequately anchored to resist the tensile stresses.

Figure 3 below presents the conceptual differences of both linings / supports.

To differentiate linings / support of tunnels in soil and rock, the following definitions are used in this paper:

- "lining" is associated to the shell-like structure that stabilizes ground pressure resisting mainly to axial forces, typical for a tunnel in soil. This type of lining is sometimes called structural lining;
- "support" is associated to the combination of rock-bolts and shotcrete, typical for tunnels in rock.

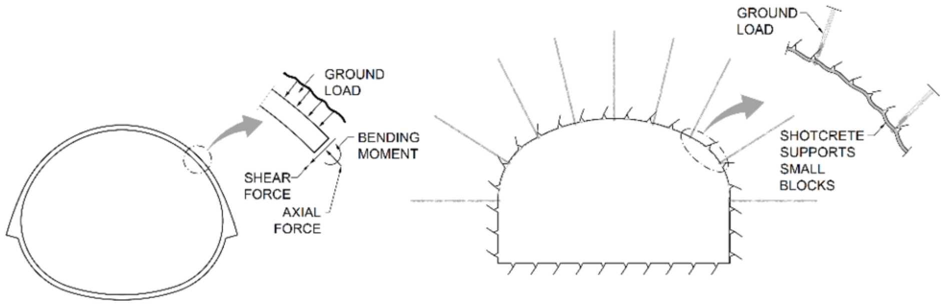


Figure 3. Concept of tunnel support in rock and in soil.

It is important to mention that the differences between linings and the concepts discussed above are far more complex, but for the purpose of this paper, the simplifications above are sufficient.

Considering the mechanisms above and the discussion in item 2, it becomes clear that in the region of soil-rock interface it is not possible to consider that a rock arch can be formed. Therefore, a structural lining is necessary to safely equilibrate ground loads.

The definition of the loads, to be considered acting on the lining and which will be used to define it (shape, thickness, strength, reinforcement, etc.) is a topic which extrapolates the scope of this paper. Comprehensive literature, like [24], may be used, as well as design standards.

Some particularities regarding lining loads are important to emphasize, specifically when designing close or at the soil-rock interface:

- Normally, the ground is not homogenous, but has anisotropic behavior, and includes discontinuities, planes of weakness, etc.
- If the lining is founded on rock, with relatively low deformability, and the surrounding ground settles due to, for example, groundwater lowering, lining loads may be higher than the total soil overburden, i.e., no soil arching will occur;
- Along the tunnel axis, foundation conditions may vary significantly, and usual 2D analyses and their simplifications may not be representative.

5. Typical Failure Mechanisms

The HSE – Health and Safety Executive [5] presented a summary of typical failure mechanisms associated to tunnels excavated using the SEM (sequential excavation method), dividing them into three main categories:

- Ground collapse in heading;
- Failure of lining before ring closure;
- Failure of lining before or after ring closure.

The location of the failures is divided into regions A and B, as presented in Figure 4. It is important to mention that the HSE, when developing his studies, focused on failures in soil, like London Clay.

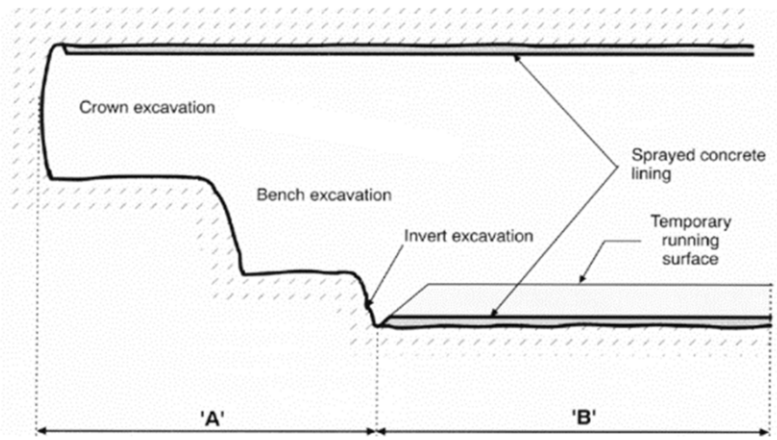


Figure 4. Possible locations of failures – regions A and B, reproduced from [5].

Typical failure mechanisms described in [5] for region A are:

- Bench, crown or full-face failures;
- Weakness in crown due to vertical fissures, pipes and manmade features;
- Insufficient cover to overlaying permeable water bearing strata;
- Insufficient cover to surface;
- Lining bearing failures and failures due to horizontal movement of arch footing.

Typical failure mechanism for region B (and in region A) are:

- Shear and compression failure;
- Combined bending and thrust;
- Punching.

The most common failures, whether published in the literature or not, occur in region A, where no or limited stabilizing effect of the tunnel lining is available. In the experience of the author in several cases, smaller failures are not even made public, if they do not progress to the surface.

Failures in region B, where the lining is already installed, are fewer, but often have a much more severe impact, because a longer tunnel stretch may be affected. If one analyzes, for example, the failure of the Pinheiros Station ([6], [25], [26]), the tunnel length affected by the accident was significant because the failure mechanism occurred not in region A, close to the tunnel face, but region B, affecting a relatively long already lined tunnel stretch. Two other recent tunnel failures in Brazil affected also a significant stretch of the tunnel and can be associated to failures in region B. This type of failure is associated to a condition where the tunnel lining, or its foundation, is not capable of supporting the load of the soil / rock mass and the failure is only interrupted where a change of either the lining or the lining loads occurs.

In addition to the failure mechanisms described in [5], at the soil rock interface other mechanisms may develop due to the presence of non-homogeneous materials, with discontinuities of the original rock mass. An interesting representation of the soil-rock interface can be visualized in Figure 5, reproduced from [27]. It can be seen, that the soil rock interface is not a clearly defined line – interface, as often idealized, but a region where the transition from soil to rock occurs. Discontinuities inherited from the original rock mass, blocks, immersed in the soil mass, soil filling rock discontinuities are some of the potential problems that are faced when tunneling through the soil-rock interface. In the majority of the cases, this region lies below the groundwater level and local high permeability stretches can lead to localized high flow rates. Water inflow with soil erosion [28] is described as an important destabilizing mechanism that occurs at the soil-rock interface.

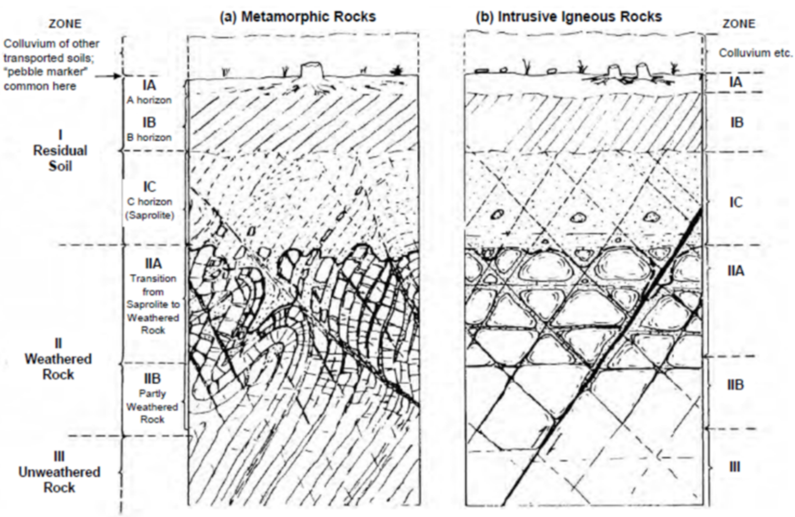


Figure 5. Typical weathering profile for metamorphic and igneous rocks, reproduced from [28].

6. Ground Treatments

The need for ground treatment arises in material that cannot be excavated due to stability problems. Possible solutions are normally:

- Reduction of excavated cross section, generating a more stable condition;
- Ground Treatments (soil and/or rock), including reduction of pore pressures.

Typically, ground treatments can be divided into following types [23]:

- Ground improvement;
- Ground reinforcement;
- Dewatering.

These types of treatments may be installed from inside the tunnel or from the surface, depending on local conditions.

An important issue that often is overseen is the fact that in the region of the soil-rock interface it will be necessary to drill through different materials: soft ground and rock. Drilling through both materials in a same hole may generate the necessity to use special tools or to "telescope", using casings with different diameters. Drilling through rock blocks may be particularly complicated, depending on their size, because of possible movements of the blocks with relation to the surrounding ground, "trapping" drill rods. Therefore, when designing and prior to starting actual ground treatment, a careful evaluation of possible scenarios should be done.

6.1. Ground Improvement

This type of treatment improves mechanical and/or hydraulic properties of the ground. Typical ground improvement techniques are grouting, jet grouting and ground freezing.

6.1.1. Grouting

Grouting is a traditional soil and rock treatment used not only in tunneling, but in several other civil engineering applications, like foundation treatments of dams. Grouting *per se* is a very extensive theme and extrapolates the scope of this paper. [29], [30], [31] and [32] are examples of interesting literature about grouting.

Grouting can be divided according to the way the grout interacts with the soil or rock mass:

- Permeation grouting – filling soil voids, by permeation, i.e., substituting normally water by the grout and not "disturbing" the soil structure;
- Fracture grouting – the grout fills discontinuities or creates ("hydro fracture grouting") and fills discontinuities;
- Compaction grouting – the grout generates compaction of the soil mass by displacement.

Grouting can also be divided according to the material that is injected:

- Conventional Portland cement;
- Other types of cements, like micro-cements, with much smaller granulometry, allowing penetration in significantly smaller voids;
- Different types of chemical grouts – polymers – like acrylic, polyurethane, silicates or epoxy.

The first step of assessing a grouting solution is evaluating its main purpose: reinforcing the soil / rock, reducing its permeability or both. The "groutability" of the massif should also be evaluated: depending of the massifs condition, grouting can even be deleterious due to a possible destruction of an existing structure. Another important issue is existence of water flow: depending on flow velocity, cementitious grouts may be inefficient, because of the cement being carried away by the water. Therefore, a grouting solution should be evaluated considering its purpose, type, material, injection pressure, injected volumes and existence of water flow.

6.1.2. Jet Grouting

The jet grouting technique "transforms" the local ground, using a high pressure grout jet (sometimes with additional air and water jets), into a soil-cement mix. Comprehensive information about it can be found, for example, in [33].

The jet grouting technique forms soil-cement cylindrically shaped columns and can be built from the surface, for shallow tunnels, or horizontally from inside the tunnel. Jet grouting is a very versatile technique, in which the ground can be improved in several ways and shapes:

- installation of a sequence of secant columns can form a "pre-tunnel," including or not a "plug," to configure an impervious pre-lining [34],[35];
- Foundation for tunnel lining in weak ground;
- Tunnel face reinforcement.

One key issue when using jet-grouting is the definition of different operational parameters to achieve the desired column diameter. In relatively homogeneous material, theoretical approaches are possible ([33], [36]). However, in very heterogeneous material, this type of prediction is almost impossible. If rock blocks are part of the ground mass, due to a "shadow" effect, no column will be formed behind this block and the jet grouting solution may be inefficient. Therefore, in the region of the soil-rock interface, jet grouting solutions have to be carefully evaluated, especially if waterproofing action is expected (i.e. one single defect can compromise the whole solution).

6.1.3. Ground Freezing

The ground freezing technique is based on the principle of transforming temporarily the water in the ground into ice, which has the advantage of a relatively high shear strength, as well as being impermeable. The ground is frozen circulating a coolant through previously installed tubes.

Although being a relatively costly and slow process, in some cases this is the only technique of stabilizing the ground, to allow excavation and lining installation.

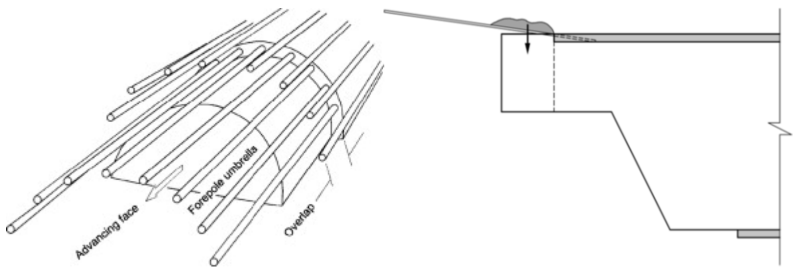
In addition of being costly and slow, a disadvantage of the process is the expansion of the water when turning into ice, which can lead to ground heave, and, during thawing, volume reduction and settlements.

6.2. Ground Reinforcement

Ground reinforcement are methods where elements are inserted into the ground, to improve its properties by mechanical action. The most common types are pipe umbrellas, spiles and face nails / bolts. Pipe umbrellas and spiles are also known as forepolings.

6.2.1. Pipe Umbrellas

The so called "pipe umbrellas" are usually 10 to 15 m long and the pipes have a 75 to 100 mm diameter, with a 30 to 50 cm spacing between tubes. They introduce a stabilizing effect, acting like a beam, with one end fixed in the soil mass ahead of tunnel excavation face and the other end, on the existing lining (Figure. 6 below).



(a) (b)

Figure 6. (a) Schematic view of pipe umbrella, reproduced from [35], (b) "beam" effect of individual pipe.

Pipe umbrellas are often associated to grouting: using rubber sleeves and injection packers, the tube can be used to inject grout. This injection can be used solely to make sure that the tube is adequately fixed to the surrounding ground, or to tentatively improve it.

Topics that should be considered with this type of solution are:

- Installation of tubes can be done:
    - o In unlined borings, if the soil is sufficiently stable;
    - o In unstable ground, cased borings or selfboring tubes have to be used;
- The installation of the individual tubes may generate, by itself, settlements;
- The space between tubes will remain “open” and the local soil exposed, which can be a problem in the case of cohesionless soils;
- Pipe umbrellas, in principle, do not reduce settlements. Settlement reducing effects can only be expected when large diameter tubes are used [37].
- Considering the necessity of installation from inside the tunnel, pipe umbrellas (as all other similar treatments) have to be installed following a conical shape. Therefore, the initial stretch of each tube will be located inside the tunnel (and will have to be destroyed during excavation), and the longer the tubes, the greater is the distance from the tunnel to them, reducing their efficiency.

6.2.2. Spiles

Spiles are relatively short bars (usually made of steel, 20 to 25 mm diameter) installed locally to protect excavation roof or sidewalls, allowing safe excavation and installation of the lining or support. Spiles are 2 to 3 m long, being manually driven or installed in pre-drilled holes.

6.2.3. Face nailing

Face nailing are installed to stabilize the excavation face and also improve global stability. Usually face nails are made of glass fiber, to facilitate is removal, as they are located in a region that will be excavated in the future. Face nails are installed in pre-drilled, grouted holes. Different length and spacings are possible; usual spacing between nails vary from 1 to 2 m and nail length between 8 to 12 m.

Some nailing systems have not only the reinforcement purpose, but also work as drains.

6.3. Groundwater lowering

The control of groundwater is one of the most important issues in tunneling. When the groundwater level is located above the tunnel, the difference in porepressures will generate water flow in the direction of the tunnel face and unsupported areas. This flow, even in low permeability ground, is highly destabilizing and, in the case of ground with low cohesion, can lead to piping and generalized destabilization. Controlling porepressures is often one of the keys to successfully building a tunnel.

The groundwater can be lowered from the surface, by deep wells, or from inside the tunnel, with horizontal drains, with or without the help of vacuum.

Important topics that have to be considered with relation to the groundwater:

- Existence of more than one groundwater level (perched groundwater levels);
- Ground stratification, with different materials and permeabilities;
- Concentrated flow in discontinuities, in rock, weathered rock or saprolitic soils;
- Constructive difficulties, considering that for the installation of deep wells it may become necessary to perforate rock, or rock bocks. Under these conditions,

the use of special tools or equipment may become necessary, increasing costs and reducing productivity.

7. Other particularities

7.1. Over-excavation

In the region of the interface, during a certain tunnel length, part or even the full cross section, has to be excavated in rock, but a structural lining needs to be installed. Depending on the rock type, drill and blast has to be used to excavate the rock. When using drill and blast, the so-called perimeter holes have to be drilled slightly outwards, which automatically leads to an over-excavation. It is very complicated to obtain a regular shape when installing a structural lining; therefore, the outer face of the excavation usually has a “sawtooth” shape, while the internal face has the designed regular shape. Figure 7 shows an example of the excavation and lining shape.

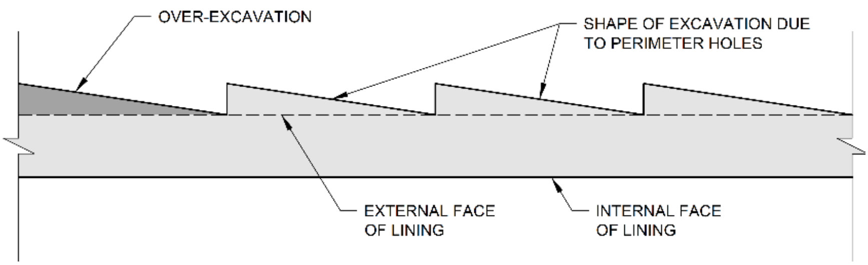


Figure 7. Excavation in rock using drill and blast and structural lining – plan view.

The condition above is theoretical and in reality, considering that the round length has to be small due to potential stability problems, over-excavations are significantly higher: to drill the perimeter holes for the next stretch to be excavated, the drill rod can only drill at an angle compatible with the installed lining, Figure 8.

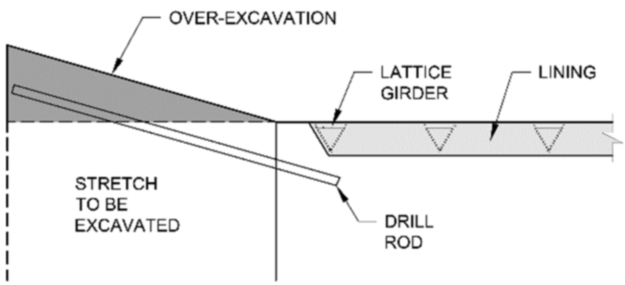


Figure 8. Excavation in rock using drill and blast and structural lining – plan view – including geometry conditioned by installed lining and drill rod.

These over-excavations, depending on the shape of the tunnel, lining thickness, available equipment and round length can easily generate average lining thickness increases of more than 100 %.

7.2. Variable cross sections

When tunneling through the soil-rock interface, its position varies along the tunnel alignment from the invert to tunnel crown, generating cross sections with very different geomechanical conditions, as presented in Figure 9:

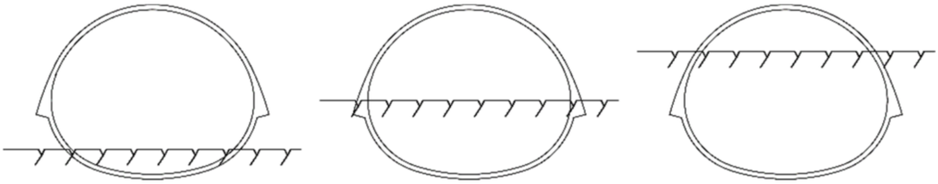


Figure 9. Variable cross sections along a soil-rock interface.

The horizontalized interface is hypothetical; normally it is sloping, generating asymmetric conditions. Its variable position generates different particular conditions:

- Interface close to the invert – possible elimination of the invert, if bearing capacity of the rock below the interface is adequate. This decision has to be investigated and evaluated carefully, due to possible high variabilities, as presented in Figure 2.
- Interface close to the center of the tunnel – possible excavation in two phases;
- Interface close to tunnel crown – difficulties to excavate in two phases.

A relatively common practice in tunneling is to install a structural lining only in the part of the cross section excavated in soil and use the rock support in the part of the section excavated in rock. This type of hybrid solution has to be evaluated carefully. The structural lining may generate high localized loads on the interface, which may not be supported by the rock. Additionally, the part of the section with rock support did not configure a continuous rock arch – Figure 10.

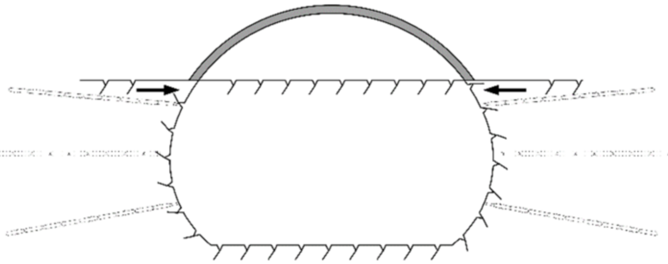


Figure 10. Cross section with crown in soil and side walls in rock.

7.3. Different Constructive Method at the same Cross Section

Figure 10 of item 7.2 above shows a common practice with relation to the lining / support. However, the geomechanical conditions at the same cross section lead to the necessity to excavate part of it by conventional means and the rest of the cross section, using drill and blast. This condition generates the following potential problems:

- Blasting will be used very close to the relatively fresh shotcrete of the conventionally excavated part of the cross section;

- The foundation of the lining will be removed, generating an unfavorable condition;
- Often more time is necessary to install the lining of the stretch excavated by drill and blast, because of the construction phases of drill and blast: drilling, loading, blasting, ventilation, muck removal, scaling, installation of shotcrete and rockbolts.

8. Case Histories

In this item, 4 unsuccessful cases are briefly presented and discussed, including some lessons learned to avoid similar problems in the future.

8.1. Case 1 – Face and Crown Stability Problems – Region A

Figure 11 presents a geological longitudinal section of a 15 m wide tunnel built recently in Brazil. The tunnel lining consisted of a 25 cm thick shotcrete shell in soil and, in rock, 4 m long rock bolts with variable spacing and shotcrete support, defined according to the rock mass classification.

Approximately 20 m from the tunnel portal, a depression of the soil-rock interface was foreseen (using boreholes and geophysical site investigations) and, due to its proximity to the tunnel, a pipe umbrella was designed to protect the excavation.

The real interface was encountered closer to the tunnel portal and extended a few meters deeper, inside the tunnel cross section. During excavation severe stability problems occurred, with soil and water ingress into the tunnel. Excavation was paralyzed and the tunnel heading protected with backfill and shotcrete.

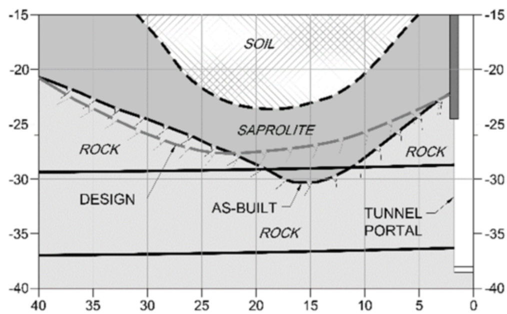


Figure 11. Longitudinal geological cross section with the idealized design soil-rock interface and the "as built" interface.

Different attempts were done to restart excavation, using:

- Conventional pipe umbrella, with pipes being installed in uncased holes. However, the holes proved not to be stable and water (pressure of almost 300 KPa) and soil was washed into to tunnel.
- Conventional pipe umbrella, and face nailing, installed using cased holes. However, water and soil were washed through the casings into the tunnel.
- Jet grouting pre-tunnel and face stabilization. However, no continuous columns were formed in the heterogeneous saprolite, and additionally, part of the soil-

grout mix was washed into the tunnel, even with the use of a so called "preventer".

Finally, a solution that included groundwater lowering with deep wells, horizontal drains and a grouted pipe umbrella, using self-boring pipes, was successfully used, allowing the excavation to proceed.

Tunnel excavation was paralyzed for several months and significant settlements occurred.

8.2. Full Collapse in Region B

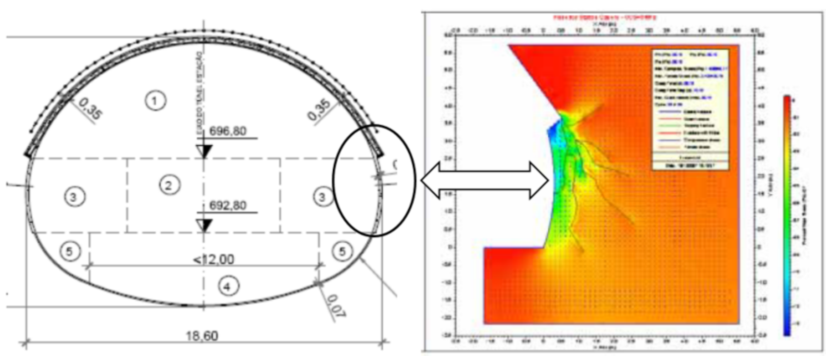
Three recent tunnel failures that occurred in Brazil involved significant tunnel stretches and could be typically classified as collapses in Region B, i.e., failures associated to a regular tunnel stretch and not limited to the region close to the tunnel face.

Constructive method to build the three tunnels was the so called NATM (SEM), with excavation done partially through conventional means and partially with the use of drill and blast.

Causes and responsibilities about the failures are still being discussed and it is not the aim of this paper to evaluate or interpret them. However, some important lessons should be learned from them.

8.2.1. Case 2 - Pinheiros Station

The failure of the Pinheiros Station, in 2007, has been presented by different authors ([6], [25], [26]), with different views about its causes and failure mechanisms. A convergent view, however, is that the overstressing of the tunnel walls by the loads of the structural tunnel lining and the immediately adjacent rock mass are an important factor (figure 12).



(a) (b)

Figure 12. (a) Cross section of the Pinheiros Station, reproduced from [6]. (b) Failure mechanism presented for the Pinheiros Tunnel Station failure by [24]

8.2.2. Case 3

To present date, no technical information was published about Case 3, a failure of a four-lane road tunnel. Failure occurred when excavation reached the transition zone from soil to rock, with relatively high cover. The upper half of the tunnel was being excavated without side-drifts or other subdivision. Figure 13 presents a schematic cross section of the tunnel and simplified geological model.

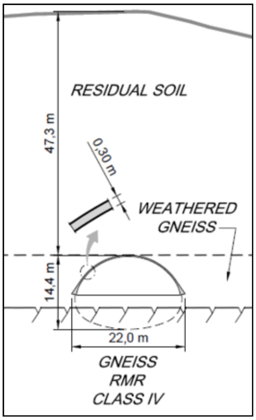


Figure 13. Cross section and simplified geological profile of Case 3.

Failure of the tunnel initiated with signs of overstress in the lining, close to tunnel face and progressed for several meters, probably due to the combination of high lining loads, overstressing of the shotcrete, and difficulties of the ground mass to properly arch and transfer loads. Fortunately, the failure did not cause casualties.

8.2.3. Case 4

To present date, no technical data was published about Case 4. This case is a failure of one of two parallel four-lane road tunnels. Failure occurred during the excavation of the central part of the cross section, connecting the two previously excavated side-drifts. Figure 14 presents a typical cross section of the tunnels. The tunnel that failed was the tunnel on the right side of figure 14 below.

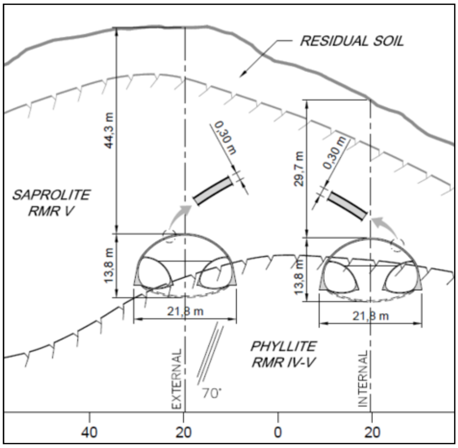


Figure 14. Cross section and simplified geological profile of Case 4.The tunnel on the right side failed.

Failure initiated, also, with signs of overstress in the lining, close to the excavation face and progressed until the tunnel portal for around 130 m. After the failure of the right tunnel, the left tunnel lining showed significant distress.

8.3. Lessons Learned

Problems in tunneling associated to the rock-soil interface generate often as consequence delays and cost increases. Therefore, a generalized conservative approach during design and construction are advisable.

The soil-rock interface is very difficult to map and is not the often idealized “line” drawn between boreholes. In Case 1 the interface was locally encountered approximately 3 m deeper than foreseen and this difference led to significant problems, including a delay in construction and settlements on the surface. If tunnel stability depends on a precise location of the interface, a conservative design approach i.e., evaluation of different scenarios, is advisable. Local vertical variations of the interface for at least 3 m can be considered normal.

In Cases 2 to 4, discussions associated to the failure mechanisms always included the relevance of geological features, that make the ground behave in a non-homogeneous way. In all cases, the foliation / main discontinuities were steeply dipping and oriented approximately parallel to the tunnel axis, and in all cases, tunnel lining failed due to overstresses. Therefore, a conservative approach regarding lining design (shape, thickness, strength, adequate foundation) is strongly recommended, based on a comprehensive evaluation of geological and geomechanical design models.

With relation to ground treatments, an equally conservative approach is recommended, including the evaluation of possible constructive problems:

- Stability of holes bored to install soil treatments (pipe umbrellas, face nails), influencing the decision of using unlined, cased or selfboring elements;
- Water pressures and associated flow rates, leading to soil-piping and “washing” grout out of the ground, reducing / eliminating is effect;
- Difficulties in obtaining jet-grouting column diameters in variable strength ground.

Planning and installing ground treatment as preventive and mitigating action is always more efficient than using ground treatment as remedial measures.

9. Concluding Remarks

This paper presented a brief summary of the main issues associated to the soil-rock interface:

- The knowledge of a representative geological-geomechanical model is crucial. This model should be continuously revised and improved with information obtained from face mappings, probe drillings and other means, updating and adjusting design if necessary;
- Site Investigations:
  - with the current state of practice, a precise location and definition of the soil-rock interface is almost impossible. Variations of a few meters should be considered as being normal. This reality should be considered when designing and building tunnels;
  - A comprehensive site investigation campaign, including quality boreholes (with high recovery rates) and geophysical testing should be foreseen for every important tunnel. The rule of thumb of one m of boring for one m of

tunnel is generally valid, but should be complemented by at least another type of investigation.

- Lining concepts: the use of a structural lining is important, considering that a typical rock tunnel lining will only be efficient if the rock mass, together with rock bolts and shotcrete, form a “rock arch” that supports the overlaying ground. Extreme care should be taken considering the fact that the idealized geometry that supports design studies never occurs during excavation. For this reason, the shotcrete / concrete used to fill the irregular shape of the so called “overbreaks” should not be considered part of the structural lining thickness;
- Ground treatments: different types are available, applicable for different conditions. There is no ground treatment, applicable and efficient for all conditions. A robust solution, in the opinion of the author, includes more than one type of ground treatment, from which, in most cases, groundwater lowering is a very effective part. It is also important to bear in mind that mitigation is normally much better than remediation, i.e., preventive ground treatments are much more effective than tentative remedial ground treatments, often under difficult conditions.
- The presented case histories showed that a conservative approach regarding lining design (shape, thickness, strength, adequate foundation) is important, based on a comprehensive evaluation of geological and geomechanical models. This conservative approach should include a continuous update of geological and monitoring information, verifying if the idealized design conditions are met and, if necessary, adjusting the design to real on-site conditions.
- The concepts of robust and resilient design are important tools to mitigate construction and operational risks during the design phase. It is however fundamental that a risk mitigation philosophy continues during construction and the entire operational life of the tunnel.

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