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# GROUND TECHNOLOGY & GEO-INFORMATION

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# IMPROVEMENT IN SOFT GROUND TUNNELLING USING AN INNOVATIVE TECHNIQUE

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The primary stabilization of tunnel face in soft ground tunnelling by means of soil nailing has been found to be an effective and economical method. This paper focuses on some aspects of the performance of two different techniques for the stabilization of the tunnel face and surrounding ground, with reference to a real tunnel project in Southern Italy. The difficult conditions met during tunnelling, due to poor quality of the rock mass and presence of high pore water pressures, required a design solution featuring a preliminary ground improvement and a heavy tunnel support. An innovative technique for ground improvement was applied using special soil nail consisting of a fibreglass bar element and an external sheath devised to contain the injected grout, which can also be integrated with a coaxial drain. The high strength of the nails, evaluated by field pull out tests, and the ability to reduce the pore water pressures ahead of the tunnel face resulted in an effective increase of the stability during excavation.

## 1. INTRODUCTION

The need for underground spaces has required the development of innovative technologies and equipments for the excavation and stabilization in the various type of ground from good quality rock to soft ground medium.

Ensuring the stability of an advancing tunnel face during excavation in soft ground condition is an important engineering design problem. Failure at the face can progress quickly as it causes weakening of the ground and can thereby induce a complete tunnel collapse. Eventually, this can lead to surface collapse or large subsidence when work is done at a shallow depth.[1]

One of the techniques used for the improvement of soft ground material for temporary and long term support is the soil nailing. Its application, firstly addressed to reinforce natural and artificial slopes, has extremely grown over the past twenty years, being accepted as one of the most efficient and economical methods of ground improvement.

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This method has also been extended to other geotechnical applications, such as the temporary stabilization of underground excavations, particularly in soft ground condition where, if no provisions are applied, tunnel face and roof are likely to collapse.[2,3] The use of soil nails to reduce and to control the pre-convergence and the face extrusion is at the base of tunnel design methods, for instance the ADECO method.[4]

In this paper, a case study is discussed, concerning soft ground tunnelling in Southern Italy which required the application of an innovative soil nailing technique and the installation of heavy support to ensure the stability of the excavation. The performance of the new technique, called P.E.R.Ground® (Pressure Earth Reinforcement Ground), was primarily investigated and compared with the one offered by a conventional nail on the basis of pull out tests in a natural coarse soil deposit.[5] Additional tests were carried out during tunnel construction.

Based on results of evaluation of the reinforcing action, the P.E.R.Ground® reinforcing system was found to be satisfactory in terms of its effectiveness in ensuring safety for the tunnel excavation.

## 2. DESCRIPTION OF THE CASE STUDY

The innovative technique for improvement in soft ground tunnelling was applied in *Timpa delle Vigne* tunnel, a new infrastructure located in Italy, along the Salerno-Reggio Calabria motorway. The twin-tube tunnel has the following features:

- Total length of tunnel is 780 m, of which 650 m and 130 m belong to the conventional and artificial excavation parts, respectively;
- Maximum overburden is about 65 m;
- Maximum longitudinal inclination is equal to 3.8% for both tubes;
- Planimetric curve of alignment have a minimum radius of 1205 m for the North tube while of 995 m for the South tube;
- The total useful span for each tube is 11.2 m to meet the traffic requirements;
- Seven niches, provided for the safety equipment installation and required accessories, are placed every 150 m;
- Two pedestrian cross-passages are placed every 300 m, connecting the two tubes for the purposes of emergency and escape.

The excavation area of the tunnel varies between 150 m$^2$ and 170 m$^2$. The so-called conventional method by means of mechanical hammer was adopted for a full face excavation. A total of four advancing faces were planned to meet the project scheduling.

A heavy primary support section type was considered to sustain the tunnel and to control the global stability of the surrounding rock mass. Such a primary support consists of soil improvement measure by P.E.R.Ground, forepoling, steel arch umbrella, steel ribs, reinforced shotcrete, lateral bolts. The in-situ cast reinforced concrete forms the final lining for the long term stability of the tunnel.

From geological point of view, the hill part consists of metamorphic phylladic schist of the Castagna Unit, covered by some debris and eluvial-colluvial deposits that form the portal area on the Salerno side. The slope of hill is characterized by several fault systems, having normal and parallel orientation with respect to the tunnel alignment, which were directly observed on the field and detected during tunnel excavation. The rock mass is quite heterogeneous, composed of clayey phylladic schist partly weathered and affected by some stretching tectonic features that create metric thickness of gouge, mylonite and cataclasite.

The hydrogeological condition, observed at the portal areas in the igneous-metamorphic unit, was characterized by low permeability due to high degree of jointing and fracturing. Conversely, along the tunnel, the tectonic features determine a heterogeneous permeability of the rock mass. In addition the formation of isolated aquitards plays a key role in terms of rock mass stability. The changes in water content induce physical degradation, weathering and uneven interstitial pressures in jointed rock mass. These effects eventually cause a decay of the geomechanical properties.

Based on the geotechnical characterization of the rock mass, two distinct geomechanical units were identified, referred to as G2a and G3a. The geomechanical parameters were also calibrated by means of back-analysis, to account for the real response observed during tunnel excavation, and they were eventually adopted in the design.

## 3. DESIGN REVIEW

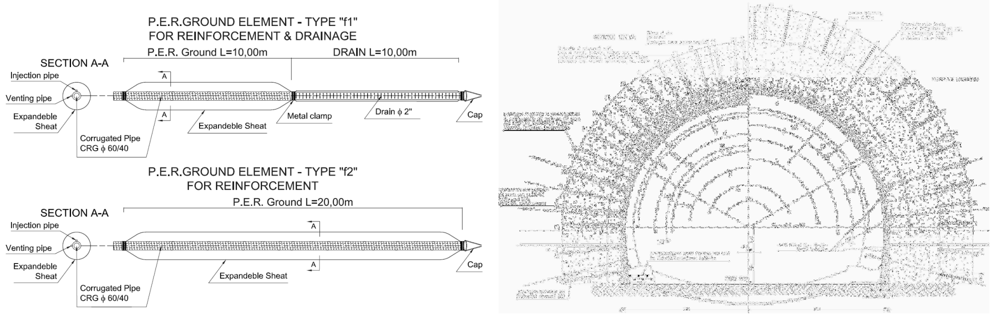
### 3.1. *Design concept*

During the excavation of the first 100 m of north tunnel *Timpa delle Vigne* from South, a remarkable tunnel deformation caused a big unforeseeable face collapse, due to the unexpected problematic condition related to poor quality rock mass, as a convergence rate of 15 mm/day was being still recorded. Generally, phyllitic schists affected by a relevant presence of clay result in a greatly weathered rock mass. The low permeability of clayey ground bearing a high water content leads to the formation of local aquitards; giving rise to pore pressures that might induce instability.

In order to reduce and to control the ongoing tunnel convergence after collapse, the need for additional counter measures was inevitable. An increase in amount of fibreglasses, umbrella pipes and longitudinal drainages was realized, but the intervention was not sufficient because not enough bond strength between ground and fibreglass was provided. In addition, the pore water pressure was not adequately reduced due to insufficient number of installed drainages.

A more detailed study of the unusual and unpredictable stress-strain response to tunnelling was conducted through a new stability analysis, additional geophysical, geological surveys and back-analysis of the induced deformation. The intervention was then focused on the introduction of an innovative solution, with a twofold goal: to apply an effective reinforcement, by means of fibreglass bars with adequate bond strength between element and ground, and a substantial reduction in pore water pressure.

These objectives were achieved through the application of two different P.E.R.Ground® systems (Fig. 1(a)). The basic system consists of a fibreglass reinforcing bar with a length of 20 m surrounded by an expandable sheath, which is injected with a low shrinkage cement mortar (type f2). The other system, with the twofold function of reinforcement and drainage, is made up of one P.E.R.Ground® element, 10 m long, and a coaxial drain of equal length, consisting of a micro slotted PVC pipe protected by geotextile (type f1).




(a) (b)

Figure 1. (a) Two different types of P.E.R.Ground®: type f1 for both reinforcement and drainage and type f2 for reinforcement only; (b) Heavy support section prescribed for tunnelling in soil like material using P.E.R.Ground®.

The basic system is characterized by an excellent anchorage between element and ground, due to the mortar being confined within the sheath, which provides an area of adherence wider than the one usually given in ordinary fibreglass elements. Such a wider area leads to a relevant skin friction that can be effectively transmitted to the ground. Furthermore, the expansion of the pressurized sheath generates a compaction of the ground surrounding the borehole and the associated increase of radial pressure can be accounted for as an increase of apparent cohesion (cf. section 3.2).

The system with additional drain has the double benefit to reinforce the soil and to reduce the pore water pressures before excavation. The drains evenly distributed at tunnel boundary and at the face, in fact, facilitate the water seepage to the boreholes and not to the excavated surfaces and reduce the water pressures at depth, consequently increasing the overall stability of the tunnel. The draining action can also be improved through the use of pumps. The combination of reinforcement and drainage allows for a reduction in installation time and cost.

The distribution of reinforcing elements and supports is shown in Fig. 1(b), with reference to a typical tunnel section.

### 3.2. *Analysis of the tunnel face stability*

To evaluate the stability of the tunnel face, the generalized statically admissible solution developed by Caquot & Kerisel[6] was used. This method was originally based on lower and upper bound theorems of plasticity and recently modified by Carranza-Torres.[7] Caquot & Kerisel developed a solution for the determination of support pressure for 2D circular tunnel sections, considering the equilibrium condition for a mass undergoing failure above the crown of a shallow circular (cylindrical or spherical) cavity in soil or rock. For the case in Fig. 2(a), this solution proposes a value of internal pressure as the minimum or critical pressure before the tunnel collapses.

In this solution a safety factor is defined which, according to the Strength Reduction Method,[8] corresponds to the ratio of the actual values of Mohr-Coulomb parameters to the critical values that would lead to failure. This approach adopts a proportional reduction

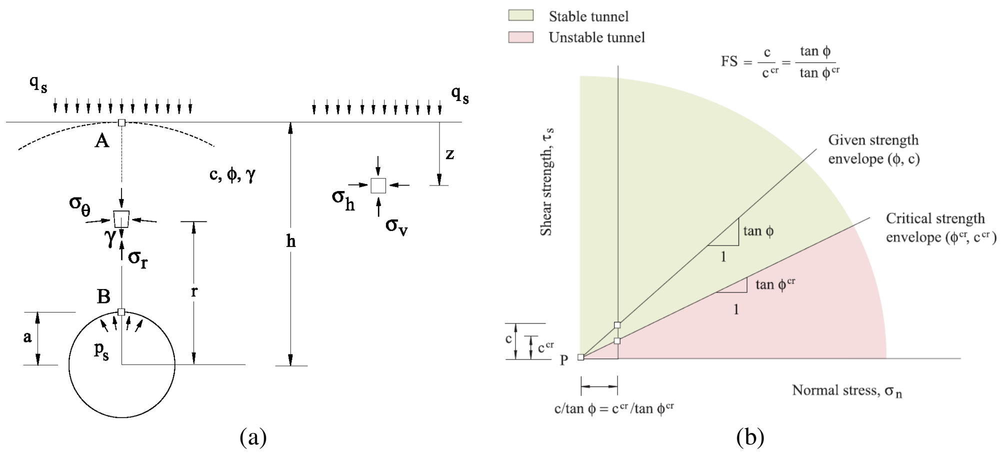



Figure 2. (a) Basic scheme for the Caquot-Kerisel solution; (b) Sketch of the concept of Strength Reduction Method in the Mohr plane.[7]

for the values of Mohr-Coulomb parameters, as expressed in Fig. 2(b). For the unreinforced tunnel face a safety factor lower than 1 was obtained, indicating that the tunnel face was likely to collapse and a reinforcement was to be required.

### 3.3. *Model of rock mass improved by P.E.R.Ground®*

The so-called "equivalent improved rock mass method" was used to account for the effects of P.E.R.Ground® on the tunnel face stability. The concept of effective cohesion proposed by Grasso *et al.*[9] was considered to evaluate to what extent the P.E.R.Ground® is effective in increasing the safety factor of the aforementioned Caquot-Kerisel method.

This concept consists of increasing the value of the effective cohesion c* in the reinforced zone around the tunnel, by evaluating it as:

$$c^* = c + \frac{\Delta\sigma_3}{2} \cdot \tan\left(45 + \frac{\phi}{2}\right) \tag{1}$$

where: $c$ = cohesion of un-reinforced ground; $\phi$ = friction angle of un-reinforced ground; $\Delta\sigma_3$ = increase in confinement pressure provided by the reinforcing elements at the excavated surface.

The confinement pressure is calculated from the confinement force $F_{\text{VTR}}$ given by each element, which in turn is obtained as the minimum value between force $F_1$, associated with the bar pull out, and force $F_2$, associated with the bar yielding:[10]

$$F_{\text{VTR}} = \min\left[F_1 = \frac{\tau_a \pi D k l}{F_{s,1}}; F_2 = \frac{A f_{yk}}{F_{s,2}}\right] \tag{2}$$

where: $\tau_a$: limit skin friction between the injected grout and the surrounding ground, $D$: borehole diameter, $l$: minimum length of bar inside the reinforced ground, $k = 1.5$ increased diameter factor, $A$: bar section area, $f_{yk}$: bar yield strength, $F_{S,1}$, $F_{S,2} = 2.0$ safety factors. In the case at hand, a value of the skin friction of 250 kPa was considered as the minimum value obtained from 6 pull-out tests. These tests and the interpretation of the results were based on the criteria given in Eurocode 7.

Having obtained the confinement force $F_{\mathrm{VTR}}$, it is possible to determine the confinement pressure applied at the face as:

$$\Delta\sigma_3 = \frac{F_{\mathrm{VTR}}}{A_F}\sum_{i=1}^{f}(n_f \cos i) \tag{3}$$

where: $A_{\mathrm{F}}$: section area of the tunnel face, $f$: number of rows of elements at tunnel face, $n_f$: number of elements in the single row; $i$ inclination of the element with respect to the horizontal direction.

Following this approach, the increment of confining pressure at the excavated surfaces provided by P.E.R.Ground® elements and the associated increment of apparent cohesion have been calculated (Table 1). The improvement at the tunnel face eventually led to an increase in the safety factor calculated by means of Caquot & Kerisel method. The safety factor remained always greater than 1.5, which is acceptable in such a difficult ground condition.

## 4. FIELD PULL OUT TESTS

A series of field tests were carried out to estimante the pull out strength of traditional and improved soil nails. Two different types of soil nail have been considered: the first (VTR) is the one customarily adopted for the reinforcement of tunnel face and consists of a fibreglass tube, 60 mm in diameter, inserted in a previously drilled borehole later filled by cement grout. The grout is injected at low pressure using a small pipe at the tube side. Using this system a maximum injection pressure up to 4 bar can be reached. In the P.E.R.Ground® system the fibreglass tube is wrapped in a flexible sheath sealed at the head of the tube. The grout is injected in the space between the tube and the sheath by injection pipe, it fills the gap between the tube and the membrane. Using this system an injection pressure up to 15 bar can be achieved.

### 4.1. *Set up and procedure for pull out tests*

In order to execute a pull out test, the fibreglass tube is installed so that its head extends from the excavation face (Fig. 3a). The head is prepared using a threaded steel pipe, which acts as a protective case and permits a widening of the lateral surface that carries the applied load by friction, thus reducing the risk of damage of the fibreglass tube. The test follows a load controlled procedure, similar to what is prescribed for pull out tests of rock bolting.[11] Given increments of axial tensile load are applied to the nail by an electrically operated hydraulic jack and the reaction force is transferred to a stiff steel plate placed against the ground surface.

The values of pressure are controlled by an analog manometer, while the displacements of the nail head are measured by optic differential levelling. For each load increment two measurements are made, the first at the application of the load and the second two minutes after, with constant applied load. Eventually, the tube is unloaded and, one minute after unloading, the residual displacement is measured, to control the occurrence of permanent

Table 1. Calculation of confinement pressure and apparent cohesion at tunnel face and surrounding ground provided by P.E.R.Ground®.

| Parameters | Value | Value |
|---|---|---|
| Application improvement of PERground | Tunnel Face | Surrounding Ground |
| $L_{useful}$ (m) = Length of overlapped reinforced zone around the tunnel | 10 | 10 |
| $R_0$ (m) = Equivalent radius of the tunnel | 6.88−7.33 | 6.88−7.33 |
| $L_{unsupported}$ (m) = Unsupported span behind the face | 1.0 | 1.0 |
| Number of improvement elements | 55 | 40 |
| $\Delta\sigma_3$(MPa) for the geomechanical group G3a | 392.3 | 32.3 |
| Geotechical parameters in presence of improvement – for the G3a,min (to use the minimal parameter) | $\Phi = 22(°)$ $c^* = 311$ (kPa) | $\Phi = 22(°)$ $c^* = 44$ (kPa) |
| Geotechical parameters in presence of improvement – for the G3a,max (to use the maximal parameter) | $\Phi = 26(°)$ $c^* = 354$ (kPa) | $\Phi = 26(°)$ $c^* = 66$ (kPa) |

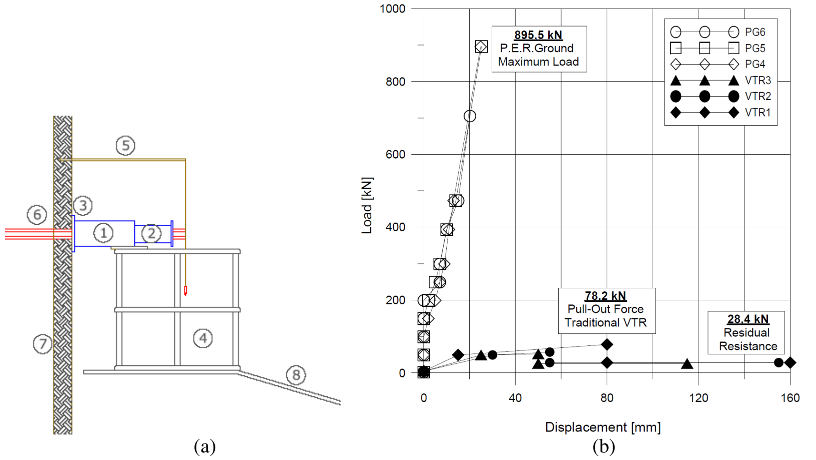



Figure 3. (a) Pull out test set up and devices: 1) jack, 2) gripper, 3) stiff steel plate, 4) platform for operator, 5) plumbline, 6) reinforcement system, 7) excavation face, 8) mechanical device for platform positioning, (b) Results from pull out tests on conventional VTR bars and PERGround®.

sliding of the whole nail along the borehole. The tests, three for each kind of reinforcing system, were carried out 24 hours after injection.

### 4.2. *Results of pull out tests*

The results of some representative tests in terms of displacement measured two minutes after the load application are shown in Fig. 3(b). Table 2 presents the test reference and the results of pull out tests for VTR and P.E.R.Ground® systems, respectively.

Table 2. Fibreglass pipe (VTR) and P.E.R.Ground (PG) test references and the results.

| | Borehole diameter [mm] | VTR length [m] | VTR diameter [mm] | Injection length [m] | Theoretical grouting volume [$m^3$] | [liter] | Grouted Volume [liter] | Lost volume [liter] | Pullout force [kN] |
|---|---|---|---|---|---|---|---|---|---|
| VTR1 | | | | | | | 60 | 33,4 | 78,2 |
| VTR2 | 131 | 2,00 | 60 | 2,50 | 0,0266 | 26,6 | 55 | 28,4 | 57,2 |
| VTR3 | | | | | | | 50 | 23,4 | 52,0 |
| | Borehole diameter [mm] | PG length [m] | PG diameter [mm] | Injection length [m] | Theoretical grouting volume [$m^3$] | [liter] | Grouted Volume [liter] | Seepage under pressure [liter] | Pullout force [kN] |
| PG4 | | | | | | | 24,8 | 3,50 | 889,0 |
| PG5 | 131 | 2,00 | 60 | 2,00 | 0.0213 | 21.3 | 27,2 | 5,90 | 895,5 |
| PG6 | | | | | | | 27,2 | 5,90 | 705,0 |

a) b)

Figure 4. (a) P.E.R.Ground® bar removed after injection and (b) tunnel face improved by means of P.E.R.Ground®.

The confinement provided by the sheath of P.E.R.Ground® for the grout dispersion makes it possible to control the injected volumes of grout. Moreover, the presence of the sheath allows for injection pressures up to 15 bar without the risk of soil claquage. Finally the homogeneity of the P.E.R.Ground® and the continuous adherence at the interface between the reinforced bar and the borehole surface (Fig. 4), leads also to high values of pull out maximum load, that are approximately 10 times higher than the values measured with the conventional VTR system.

## 5. CONCLUSIONS

A difficult tunnelling condition in Southern Italy, due to poor quality rock mass and to high pore water pressures, required the recourse to an innovative ground improvement technique, P.E.R.Ground® system, a new soil nailing technique which provides high pull out strength and the possibility to couple the reinforcement with a drainage action.

Field pull out tests have been carried out to investigate its performance. From a mechanical point of view, P.E.R.Ground® provides a pull out strength higher than that provided by conventional soil nails. This is likely due to the possibility to apply high injection pressure without loss of grout and the occurrence of claquage; accordingly to obtain a homogeneous grout column and a continuous adherence at the interface between the reinforcement element and the borehole surface.

The presence of a coaxial drain reduces effectively the pore water pressure to some extent ahead of the excavation face, whereby increasing the stability.

Finally, the performance of P.E.R.Ground® system is less dependent on the conditions of the site and on the quality of grout and injection, consequently leading to a more reliable soil treatment.

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