Tunnel face reinforcement in soft ground design and controls during excavation

P.Lunardi
Rocksoil S.p.A., Milano, Italy

A.Focaracci
Rocksoil S.p.A., Milano, Italy

P.Giorgi
Fondedile S.p.A., Napoli, Italy

A.Papacella
Fondedile S.p.A., Napoli, Italy

ABSTRACT: During the construction of the tunnels located along the Firenze-Roma Direttissima, stretch Arezzo Sud-Figline Valdarno, V lot, in order to allow the construction of the 12 m. in diameter tunnels in soft saturated sandy-silty and clayey soils it was used, the first time in Italy, a new full face excavation method with core reinforcement by means of fiberglass tubes.
The technological and static aspects related to the effectiveness of this method have been analysed carefully in this work and two control compaigns have been carried out to measure the desplacements of the ground close to the heading.

1 INTRODUCTION

The paper outlines the operational and control features developed during the construction of a tunnel, 12 m. in diameter, located along an important high speed railway line in Central Italy: the Firenze-Roma Direttissima (Fig.1).

In the design stage the Rocksoil Engineers had studied, the first time in Italy, the full section excavation with core stabilization by means of fiberglass tubes.

The idea to use this preconsolidation method arised from the need to allow safe full face excavation in soft saturated silty-clayey soils with frequent interposed layers of water bearing silty sands, and overburden up to 90 m.

These soil conditions had caused serious difficulties during divided section excavation in the adjacent lot tunnels, involving uncontrollable convergences on the excavation profile.

In order to avoid prosecution in already collapsed soil it was feeled that it was necessary to increase as much as possible the excavation rate, with suitable measures to preserve the core integrity on the working face.

Since it was the first application in soft ground, the technological and static aspects related to the effectiveness of the face and cavity stabilization, have been analysed carefully in the course of the work.

A thorough program of site controls has been carried out and will be herein after reported.

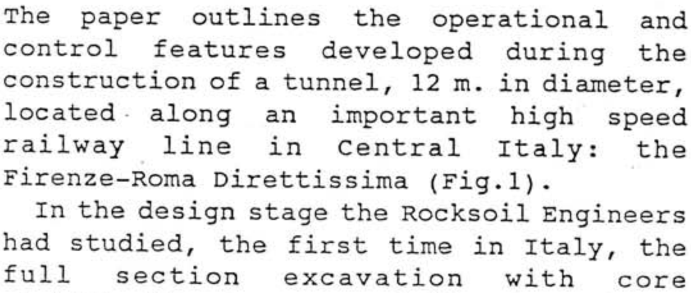


Fig. 1: Lay out Rome-Florence high speed railway line.

2 DESIGN STAGE

The Poggio Orlandi tunnel, 850 m. long, is located within the last stretch (between Arezzo Sud and Figline V) of the Romá-Firenze high speed railway line.

This stretch runs through the Valdarno lacustrine basin, characterized by the deposition of soft silty-sandy and clayey soils.

The soil investigations along the tunnel layout had shown the prevailing presence of saturated sandy-silty soils exhibiting a low consistency.

The extrusion tests had proved the instability of the heading even under a small overburden (Fig.2).

Consequently the desing has been based on a full face excavation with a previous

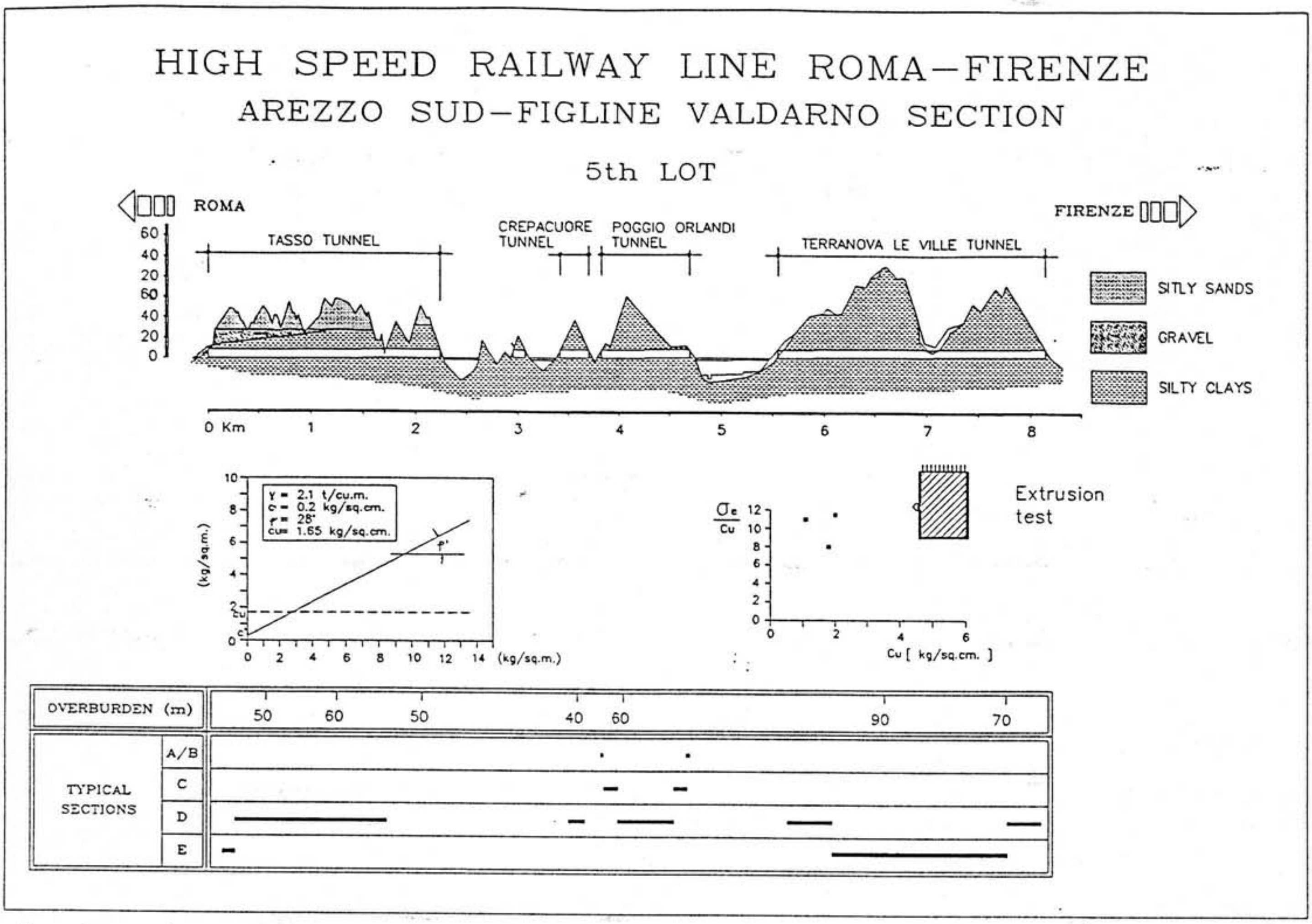


Fig. 2: Typical soil profile, geotechnical data and design features.

consolidation by an adequate pattern of fiberglass tubes thoroughly cemented into the ground ahead of the face, immediate reinforced shotcreting, and invert casting at a max. distance of about 1.5 times the tunnel diameter (Fig.4).

This procedure enables to stop promptly or to prevent the deformations involved by the excavation, since soil release around the cavity (unavoidable when divided section tunnelling is carried out) is prevented and exacavtion rates may be increased and kept constant.

The stabilization is aimed to preserve the natural soil structure, acting ahead of the face when this is a still in a triaxial constraint condition.

Besides a soil strength improvement, the function is to prevent an excessive decrease of confining pressures when the bore outline

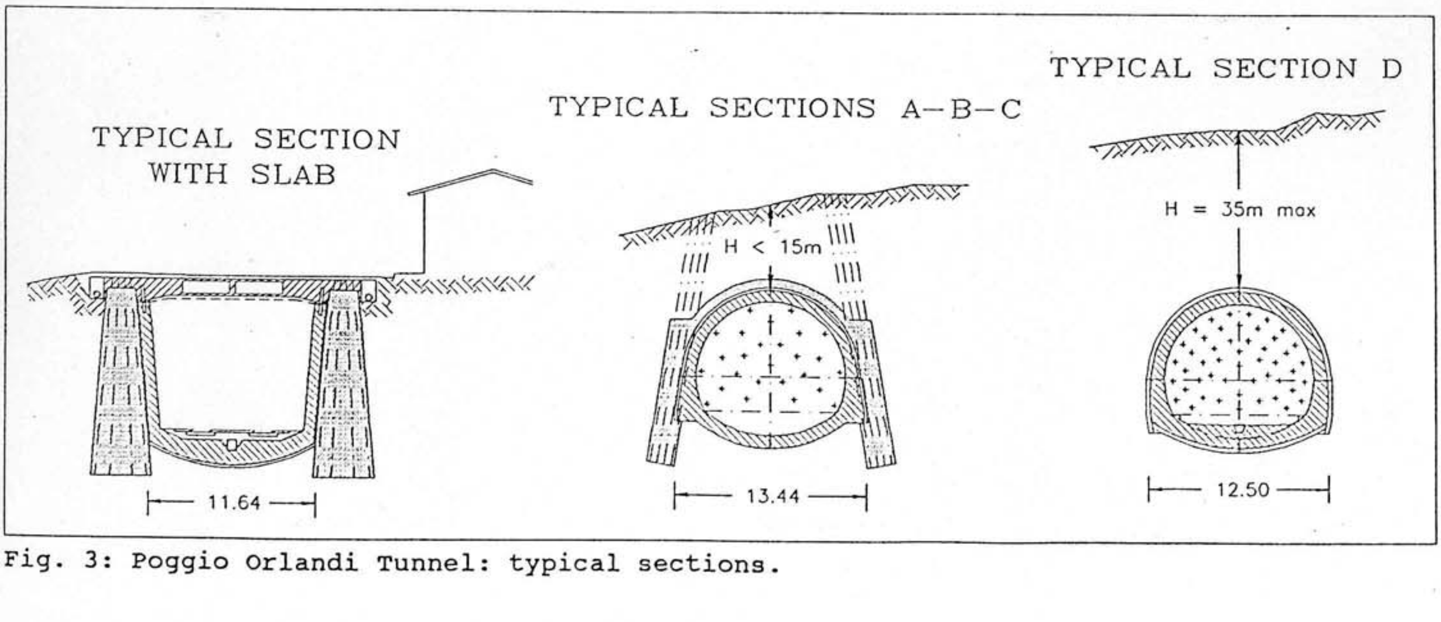


Fig. 3: Poggio Orlandi Tunnel: typical sections.

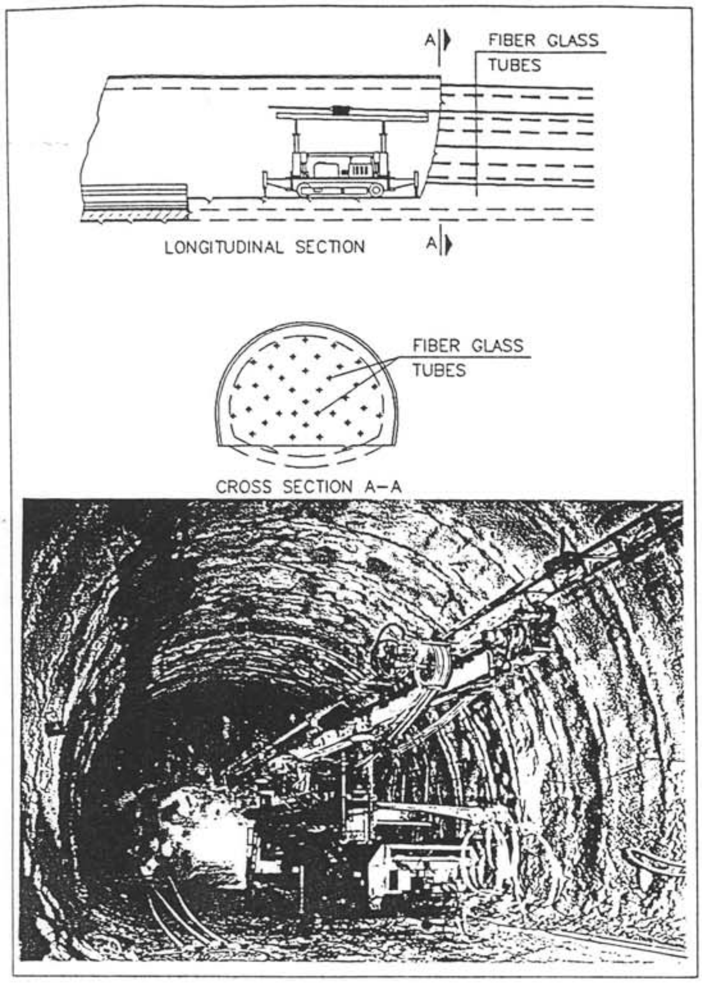


Fig. 4: Tunnel face reinforcement.

deformations may be still under control.

Since in the low overburden sectionsthe cave-in risk was greater, the design had provided for the realization of a bearing arch formed by consolidated soil around the bore perifery after each excavation stage, besides the core stabilization (Fig.3).

The tunnel construction has been started on june 1st, 1989 and completed on july 7th, 1990.

The mean rate of excavation and pre-lining (with reference to each heading) was 10 m per week, that is about the double in comparison with the rate recorded by divided section tunnelling.

3 CONSTRUCTION STAGE

3.1 Thechnological aspects

The design guidelines about the face stabilization procedure regarded the drilling technology and the type of cementation of fiberglass tubes into the ground.

In prevailingly clayey soils dry auger drilling was carried out; the fiberglass tubes were then introduced into tho holes sealed by an adeguate grout (Fig.5).

During the excavation of Poggio Orlandi tunnel, the remarkable amount of sandy fraction in randomly distributed lenses or levels advised against augering and suggested the use of jet grouting technique to create the cemented bulb of each tube, 15 m. long.

The jet-grouting parameters are summarized in the following Table.

Table 1. Jet-grouting parameters

jet-grouting pressure:	5 MPa (up to 2 m) 40 MPa (2 to 15 m)
cement/water ratio of the grout:	0.8
drilling tool diameter:	90 mm
nozzles number and diameter:	3 x 1.8 mm. dia.
monitor rotational speed:	13 to 18 r.p.m.
number of jet-grouting passes in each hole (lenght = 15 m):	3
monitor penetration speed:	1.5 to 3.0 m/min (1st pass) 2.5 to 3.5 m/min (2nd pass) 2.5 to 3.5 m/min (3rd pass)
monitor withdrawal speed:	5.0 to 7.5 m/min (1st withdr.) 5.0 to 7.5 m/min (2nd withdr.)
cohesive soils:	1.0 m/min (3rd withdr.)
cohesionless soils:	0.5 m/min (3rd withdr.)

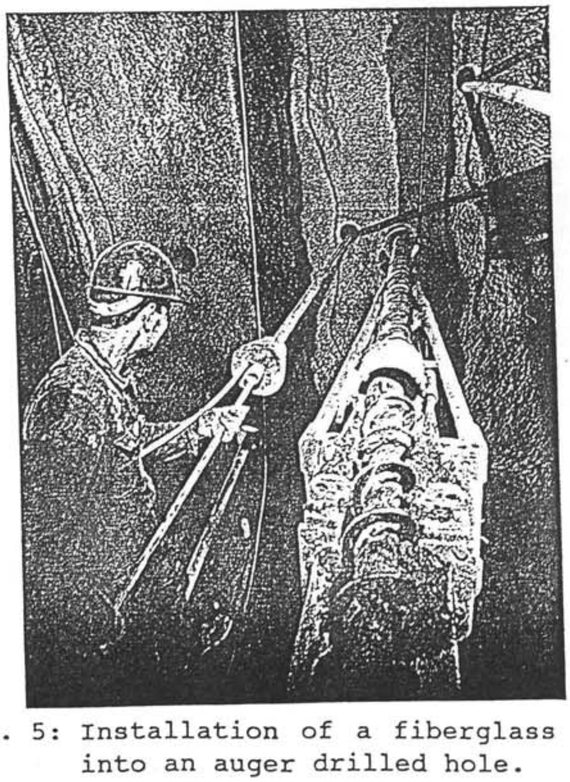


Fig. 5: Installation of a fiberglass tube into an auger drilled hole.

When drilling and jet grouting operations are ended, the fiberglass tubes are introduced into the holes.

In the case of Poggio Orlandi Tunnel, this technique has enabled to obtain the following advantages:

a. In presence of cohesionless soils, besides the creation of a stabilized soil bulb, with a size even more than 4-5 times the drilling tool diameter, there are no difficulties for the insertion of the fiberglass reinforcement.

b. Both in cohesive and cohesionless soils the fiberglass tubes are centred into the bulb of consolidated soil, with the assurance of an effective sealing on the full length of the reinforcement tubes.

c. The execution of subhorizontal upward treatments does not involve outflow of the stabilizing grout owing either:

- to the sealing effect abtained in the top two metres of drilling and grouting at 5 MPa, (Table 1), with subsequent introduction of the fiberglass tube;
- and to the higher viscosity of the grout mixed in place with the soil.

d. In cohesive soils the diameter of the jet grouting columns, executed as shown in Table 1, ranges between 140 and 170 mm against the 100 mm that are obtained by augering and subsequent low-pressure injection.

The disadvantages of this technique regard highly altered cohesive soils, since release of jetting pressures are likely to cause the detachment of big blocks of soil during the excavation works.

The stabilizing mix is a Portland 425 cement grout with a cement/water ratio equal to 0.8 by weight.

The mechanical strength of recovied grout-soil admixtures, evaluated by unconfined compression tests on cubic 15 cm. specimens, is about 6.5 MPa at 24 hours, with a trend to 15 MPa after 120 hours curing.

The site controls during the work have shown that the last 15 columns reach a range of strength between 0.9 and 6.0 MPa, that is 6% to 40% of the experimental reference strength after 120 hours (Fig.6).

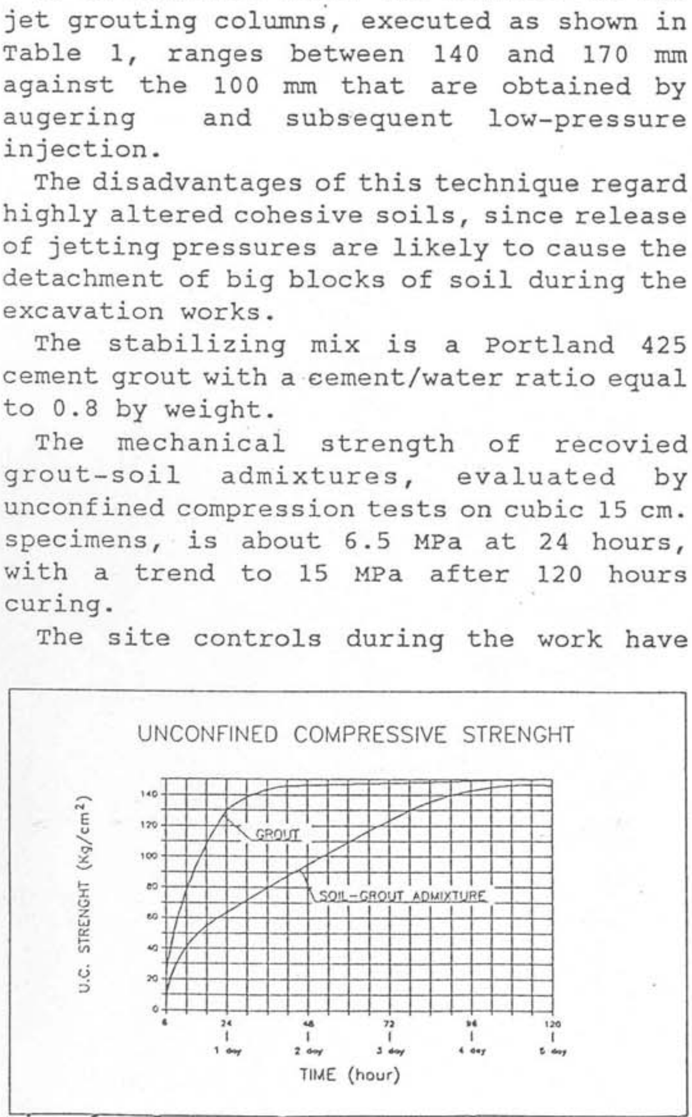


Fig. 6: Plots of compressive strength versus curing time.

Consequently the treatment sequence was examined everytime according to the actual soil conditions at the working face, in order to give priority to the consolidation of the most unstable zones (sand seams or highly altered cohesive soils), so as to obtain higher strength values in these critical soil conditions from the beginning of the excavation stage.

3.2 Static aspects

3.2.1 Face stabilization

During the excavation, the working face showed the tendency to assume a concave shape, likely due to the natural mibilization of an arching effect in both vertical and horizontal planes (Fig.7).

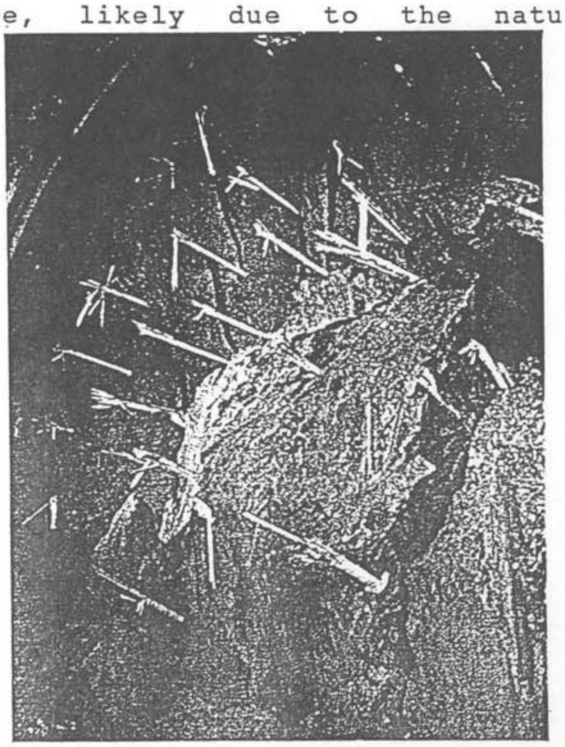


Fig. 7: Detachment of soil blocks due to arching effect.

Consequently the face shaping at the end of each excavation stage has been conceived so as to comply with this natural trend, in order to prevent the detachment of soil blocks.

Moreover, to the purpose to make safer the work near the face, a shotcrete plug (10 cm thick) reinforced with metal wire net, has been considered as a right measure.

In-fact the reinforced plug is anchored to the top of the fiberglass tubes, that are cemented into the core over a min. lenght of 5 metres.

This solution has prevented sudden

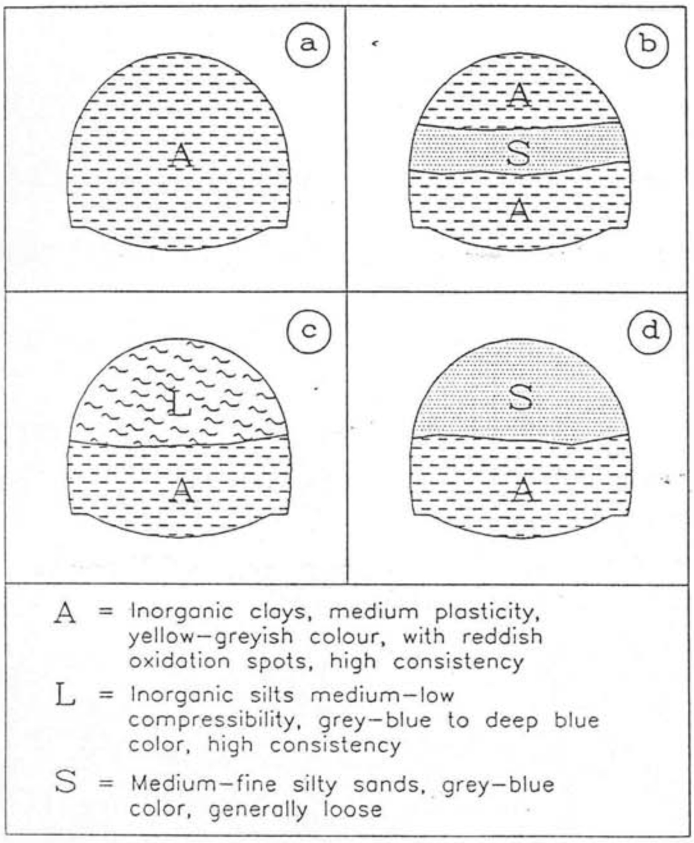
detachements of material.

3.2.2 Core stabilization

The factors taken into account to define the nailing pattern and the sequence of preconsolidation works have been the following:

a. Soil profile at the limit of penetration and expected conditions within the soil mass

Fig. 8: Typical soil profiles.



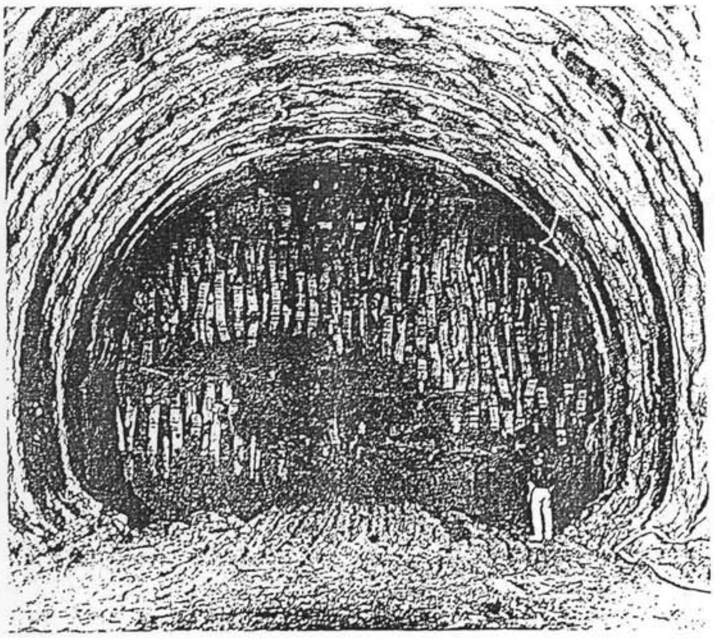


Fig. 9: Example of soil profile in geotechnical condition b.

to be consolidated (Fig 8 e 9).

b. Constraint state within the same.

c. Setting time of the stabilizing grout.

Furthemore it has been observed that a good preconsolidation has been achieved when, during sebsequent penetration under the action of the excavator ripper, the face material tends to rupture into bloks of fairly small size (less than one cu.m.).

On the contrary, when the size of bloks reached the order of some cubic metres, it has been observed:

a. Uncontrollable shaping of the cavity with consequent overbreaks.

b. Greater deformations (recorded by convergence measurements) and therefore heavier loading conditions on the tunnel lining.

The concepts here before expressed have suggested to increase the nailing density on the arch and in unstable zones according to the schemes hereinafter reported.

A particularly critical situation had to be faced when the face presented seams or fairly thick layers of sandy soil in a loose saturated state, confined by compressible silty-clayey layers.

This geotechnical situation has been found on the whole tunnel layout and particularly in the stretches with a low overburden, where a bearingarch was provided for (typical section A-B-C, fig. 4).

In order to optimize the consolidation effect, the working sequence has been studied everytime for the face around the bore perifery (in low overburden cases) in such a way as to adapt the procedure to the soil profile and geotechnical characteristics.

In general the consolidation works were started in the most unstable zones, then carried uot along the periphery and finally in the central part of the face.

This sequence has been followed up, to the purpose of obtaining the highest strength of the stabilized soil just in the most critical zones, as regards the overall face stability.

The recurrent geotechnical situations are represented in fig. 8.

The working sequence in shown in fig. 10.

The selected procedures in any case tended to concentrate mostly the treatment on the zones where soil cohesion was lower.

The consolidation works have been carried out from the bottom upwards in order to enable high pressure grouting aver an already consolidated portion of soil.

The initial treatments in each face zone have been useful also to check the assumed soil condition ahead of the working face.

This has been possible due to the specific experience acquired by the site staff.

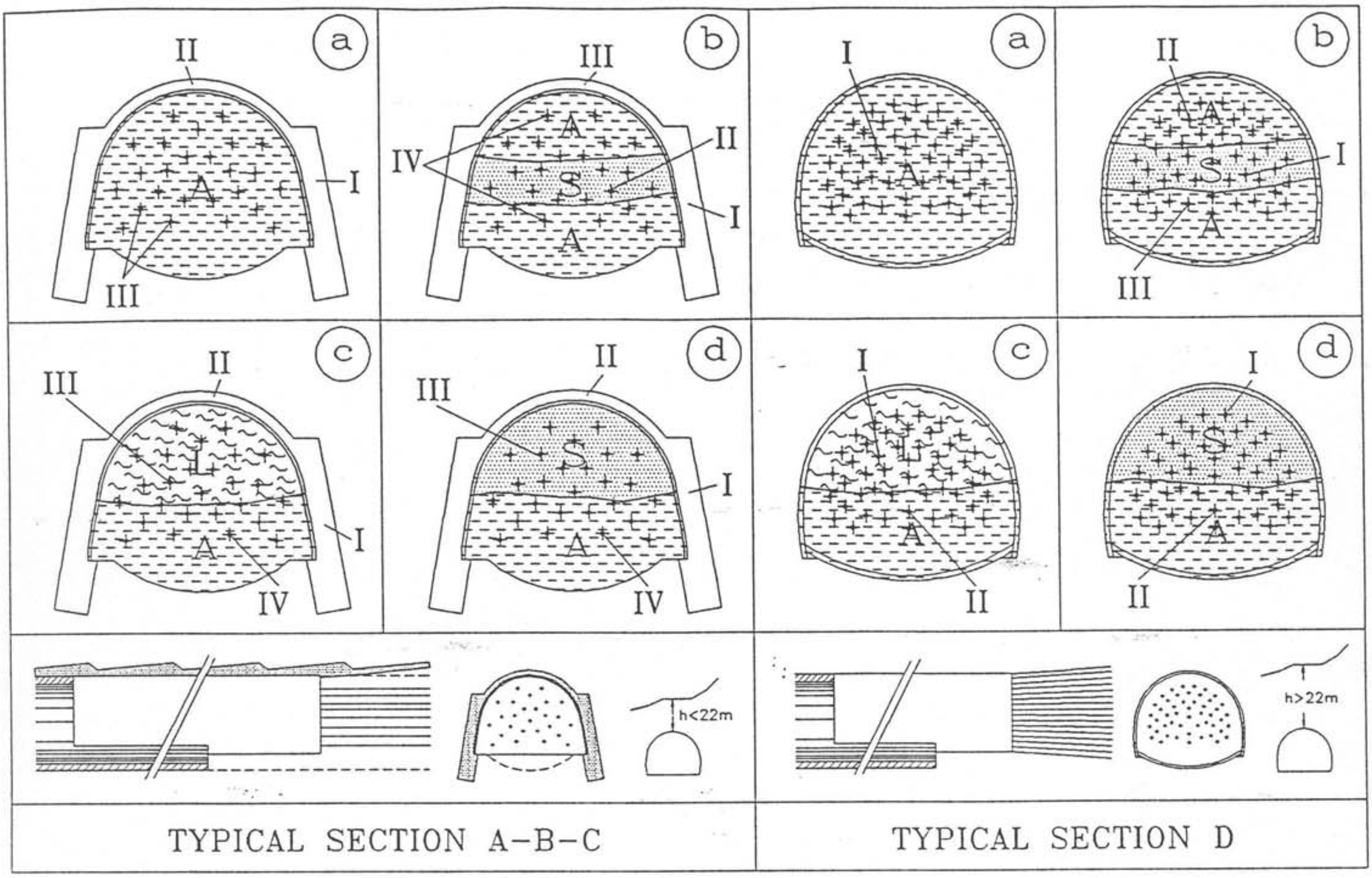


Fig. 10: Sequence of consolidation stages in different typical soil conditions;

4 GROUND STRAIN MEASUREMENTS CLOSE TO THE WORKING FACE

4.1 Introduction

During the construction of Poggio Orlandi Tunnel, two control campaigns have been carried out, to the purpose of measuring the absolute and relative desplacements of the ground, close to the heading.

By these controls more information could be obtained on the behaviour of the preconsolidated soil core.

The measurements have been made both in "complete" consolidation conditions on a section type D (50 fiberglass tubes 15 m long and 50 tubes 5 m long, remaining from the preceding nailing stage) and in "residual" consolidation conditions on a subsequent type D stretch (50 tubes metres long).

The first campaign has been carried out at chain. 29+455.80 under an overburden of about 25 m, between april 28 th, 1990 and may 2nd, 1990; the second control campagn was located at chain. 29+505.80, where the overburden was about 17 m, from june 22nd to 26th, 1990 (fig. 11).

The first campaign involved 13 measurement points for recording the absolute displacements of the heading surface and one multiple extensometer (15 m length, strain measuring marks at 1 m spacings) installed sub-horizontally on the face, in order to record the relative displacement of the points within the soil core.

The second control campaign comprised 5 points for the measurement of absolute displacements of the heading surface and one multiple extensometer to record relative movements (10 m length, 1 m spacing of measurement bases), installed as the previous one.

The results of measurements carried out

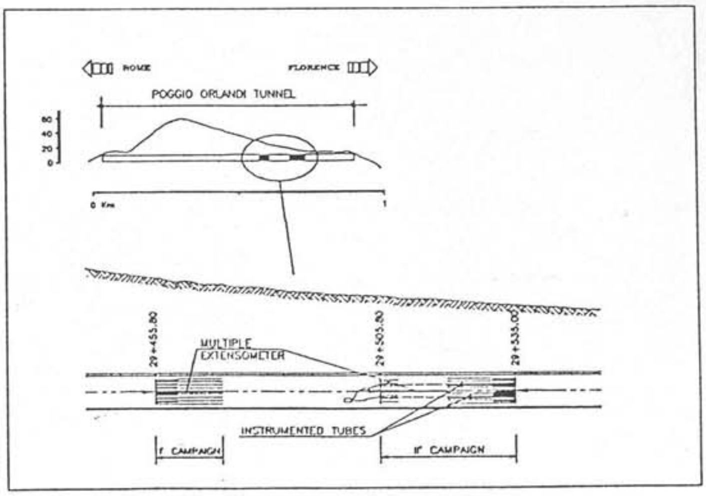


Fig. 11: Location of instrumented sections.

during the two campaigns shall be compared and discussed hereinafter.

1st Campaign

- topographic displacement measurements of the heading surface, carried out during a working break;
- extensometer readings, still during a break, with 15 m ahead preconsolidation and surface protection by shotcrete;
- extensometer readings after 5 m penetration, that his half-way during the 10 m excavation stage; the heading surface had no shotcrete protection.

2nd Campaign

- heading surface displacements, topographically measured as above;
- extensometer readings during excavation on the opposite face, initially at a distance of 30 m (fig. 12).

The geotechnical characteristics of the soil involved by the two control campaigns, resulting from samples recovered during excavation were fairly similar to those assumed in the design stage.

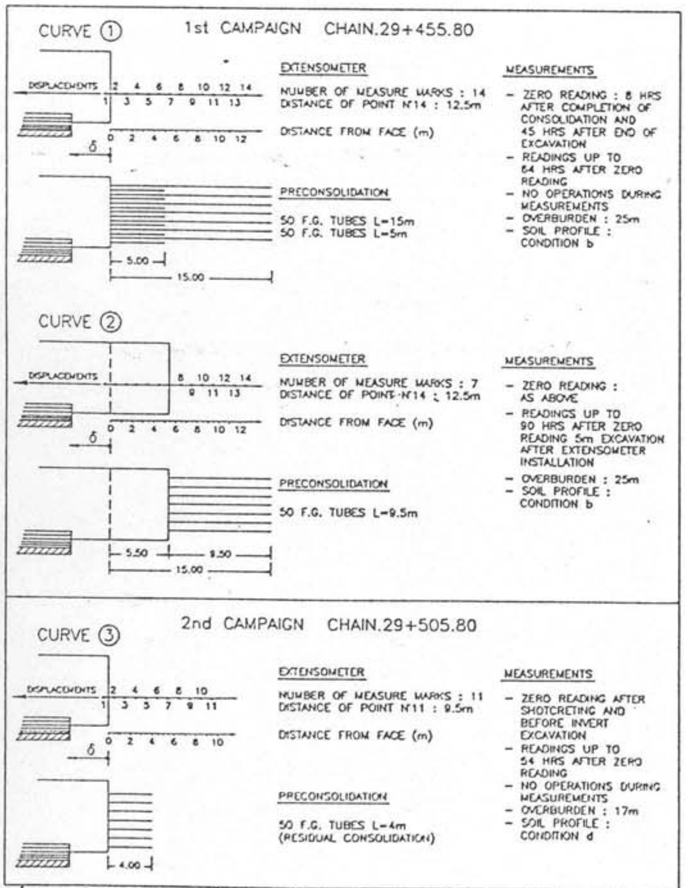


Fig. 12: Working face conditions in the different measurement stages.

4.2 Considerations on control data and interpretation

The sub-horizontal multiple extensometers enable to measure the extrusive movements of the core during a break of 64 hours in the 1st Campaign and 54 hours in the second one.

In both cases the heading surfaces had been protected by reinforced shotcrete, whereas the core stabilizing treatments were different.

In fact in one case (1st campaign) there were 50 sub-horizontal columns reinforced by fiberglass tubes 15 m long, besides the 50 similar reinforced columns corresponding to the residual consolidation carried out in the preceding stage.

Differently in the other case (2nd campaign) there was no new core consolidation; hence there were only the residual columns, still reinforced by fiberglass tubes but with a length of 4 m only and therefore reproducing the most unfavourable conditions in regard to extrusion effects.

The readings sequence in the two campaigns and the relative displacement values recorded by the multiple extensometers are summarized in fig. 13.

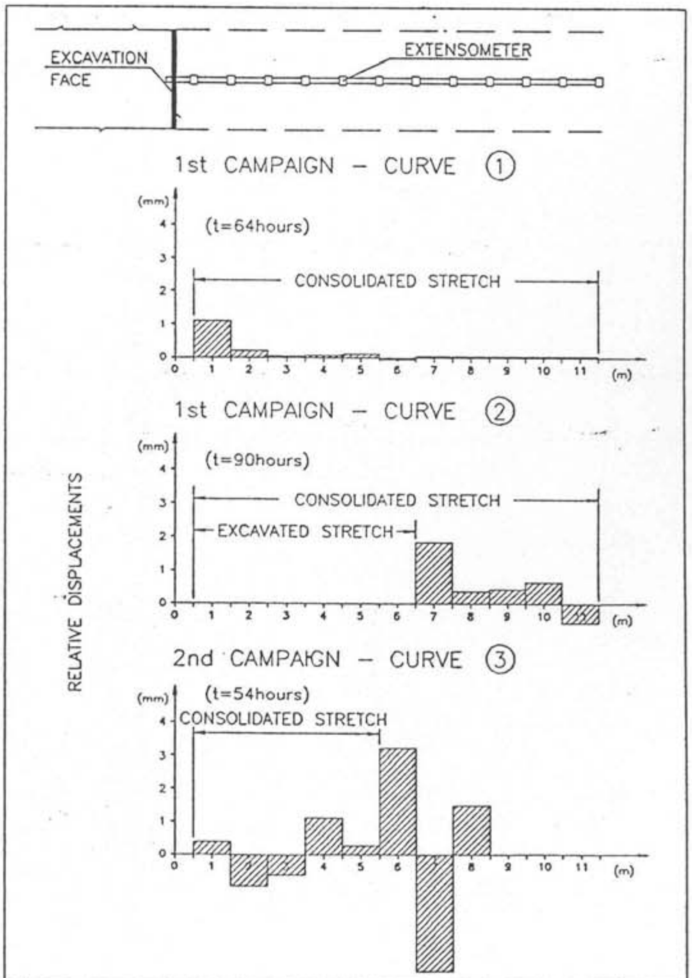


Fig. 13: Plots of relative displacement versus distance from excavation face.

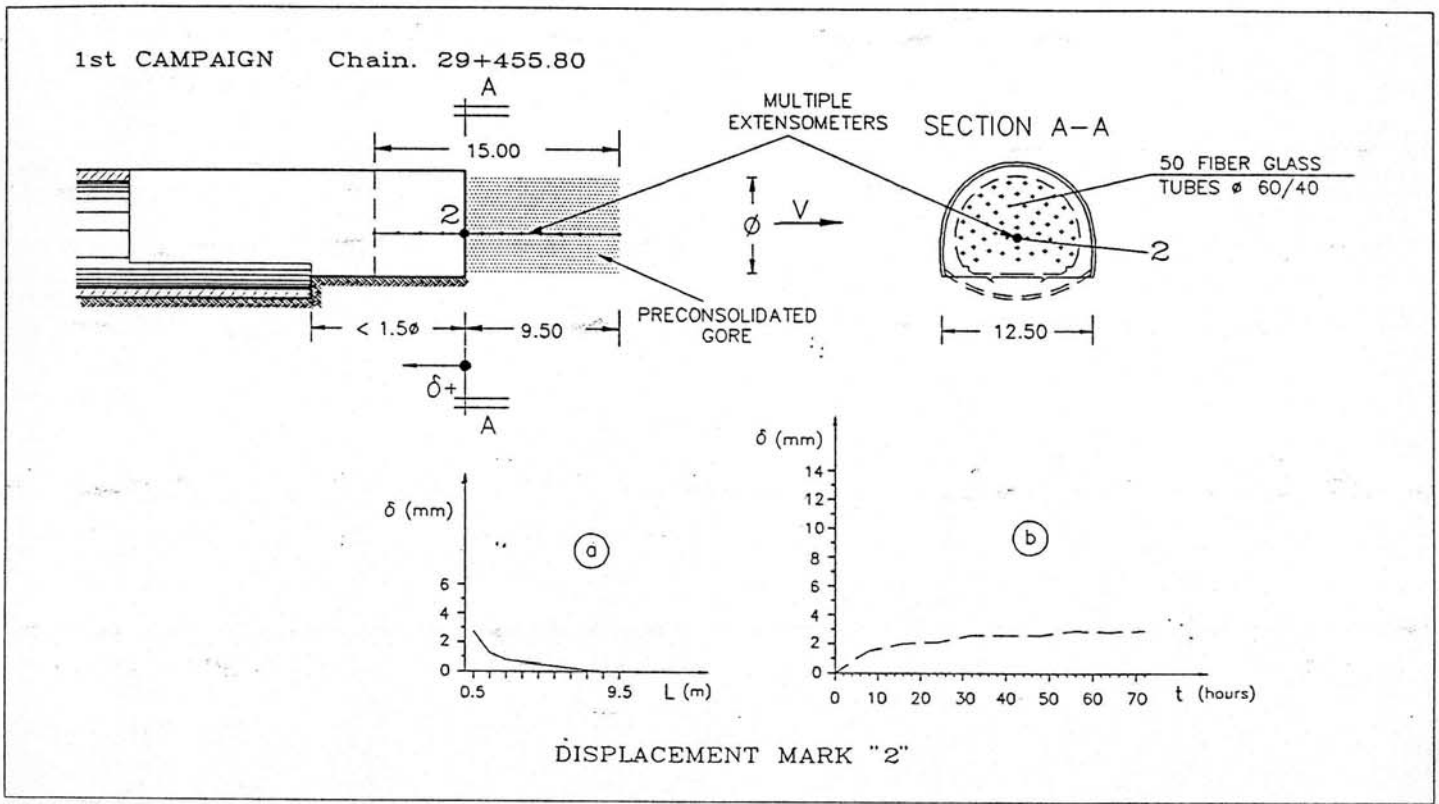
Fig. 13 shows the relative strains between each couple of marks, measured in the 3 different conditions curves 1, 2 and 3.

These displacements are referred to the middle of distance between the marks of each couple; the distance of these middle points from the face are plotted versus relative displacements, expressed in mm.

From the analysis of this graph we may draw the following comments:

- consolidated core - temporary excavation break (curve 1).
  The relative displacements within the core are fairly slight and decrease gradually with increasing distance from the face; the max. value recorded at the

Fig. 14: Plots of absolute displacement versus distance from face and elapsed time. I compaign.



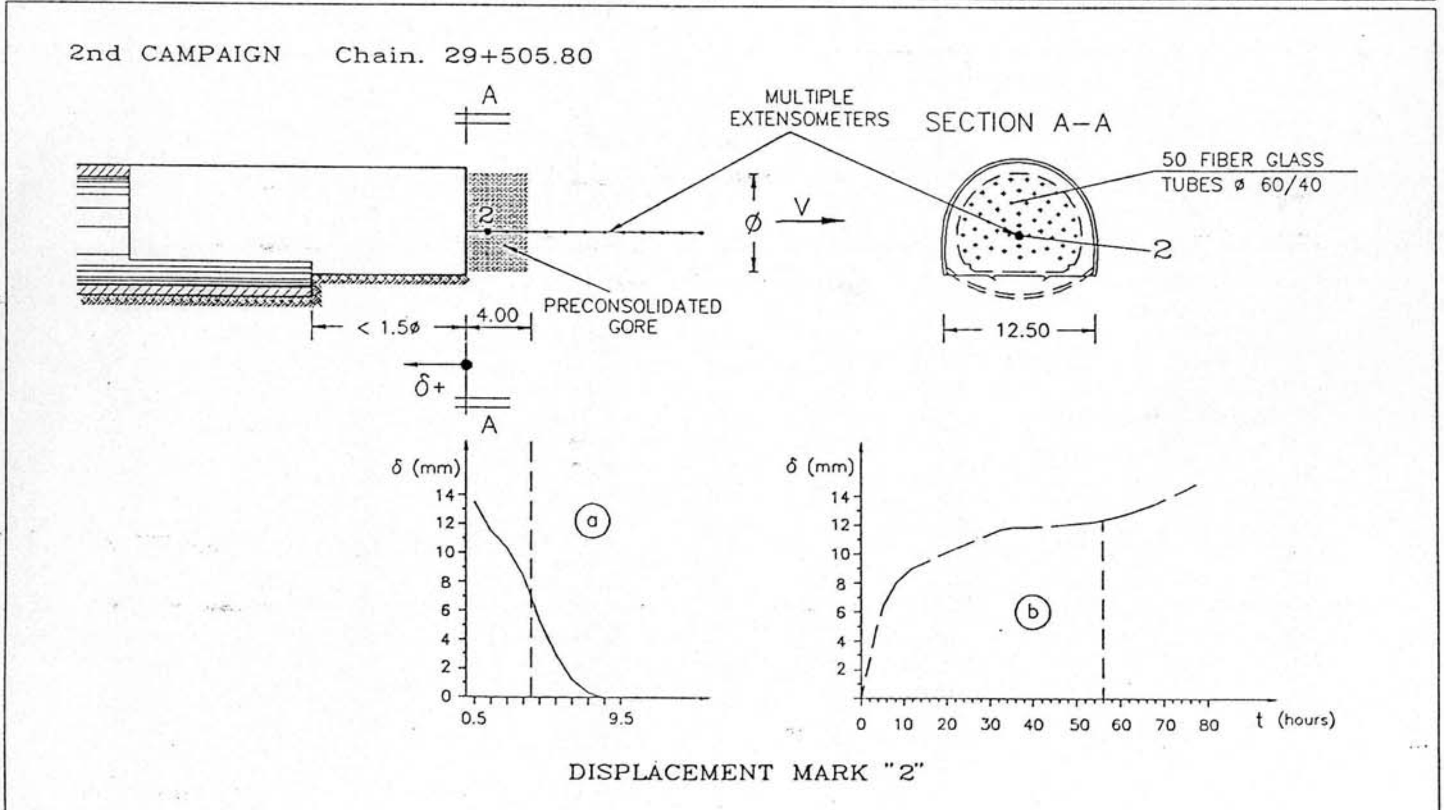


Fig. 15: Plots of absolute displacement versus distance from face and elapsed time. II compaign.

least distance from the face (1 m) is 1.5 mm only.

- consolidated core - excavation in progress (curve 2 - face 5 m ahead).
  The relative displacements measured during excavation are much higher than those recorded in the above break conditions; the point 2 m distant from the new face position (7 m distant from the former face position; see curve 2) underwent a 4.2 mm di splacements.
  The time interval between the measurements in break and excavation stages was 26 hours, while the times elapsed from 0 readings were 64 and 90 hours respectively.
- partially consolidated core - excavation break (curve 3).
  The relative displacements measured within the partially consolidated core are higher than those recorded in the same points of the fully consolidated core.

The max relative displacement has been found at 4 m distance from the heading, that is the consolidation limit ahead of the face.

The graphs of fig 14 ad 15 too show the extrusive movements of the two cores differently consolidated, though expressed in terms of absolute max displacements, on the assumption of zero displacement at 9.5 m distance from the face.

Fig. 15/b shows the absolute displacements recorded in the time interval between zero reading and the reading after 56 hours; during this period the excavation had been interrupted on both faces, located 30 m apart.

The same graph shows also the absolute displacements measured between the 56th and the 78th hour, that is the time interval during which the excavation had been resumed on the opposite face at an avarage rate of one metre every 4 to 5 hours.

The following conclusions may be drawn from the analysis of the data plotted in fig. 15/b:

- the max absolute displacement, measured in the closest point (0.5 m ahead of the face) is about 13 mm;
- the core extrusion rate is influenced to a fairly moderate extent by the initial excavation stage at the opposite heading, started from a distance of 30 m from the instrumented face;
- the 60% of the final absolute displacements has occurred in the first 12 hours after the zero reading, therefore within a period that is about 20 % of the observation time, during which the invert has been excavated and concreted.

The reliability of the assumption of zero dsplacement at the max distance from the face along the extensometers is confirmed by the results of the absolute displacement controls of the extensometer tops.

These controls have been carried out at the same time as the measurement of relative displacement of the marks within the ground core.

Finally we shall consider the horizontal absolute displacements of the heading surface, that is to say of the soil behind

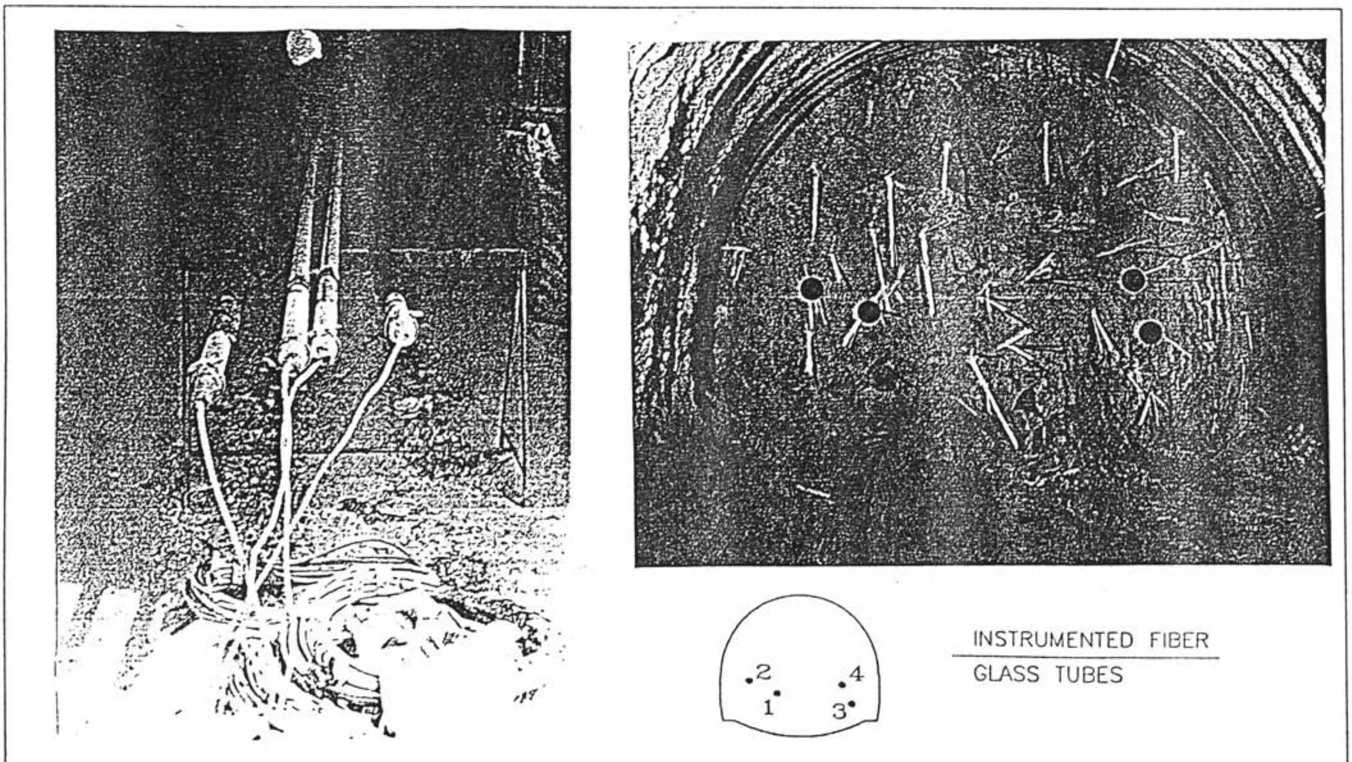


Fig. 16: Instrumented tubes ready to be installed and location of instrumented fiberglass tubes.

the shotcrete wall, measured during both control campaigns.

Also the measurement of such horizontal displacement has been made at the same time as the extensometer readings; in the first campaign the displacements of 11 marks into the soil mass have been measured; in the second campaign the controlled points were only three within the soil and one on the shotcrete wall.

The results recorded by the two campaigns are qualitatively similar: negative horizontal displacements in the upper half on the excavation face and positive values (extrusion effect) in the lower part.

In the observation period (48 hours) a max negative value of 0.5 mm and a max positive value of 2.5 mm has been recorded.

5 MEASUREMENT OF AXIAL STRAINS IN THE FIBERGLASS TUBES

5.1 Introduction

In order to relate the measured core deformations to the strain behaviour of the working face, four instrumented tubes have been installed at chain. 29 + 535.00 in the course of the 2nd control compaign.

The four tubes were part of complete consolidation stage on a section type D.

The instrumentation consisted of strain-gauges fitted to measure axial strains and consequently the tensile and compressive stresses in the instrumented sections.

In each tube, 6 sections 2.5 m. apart have been instrumented.

The mechanical properties of the fiberglass tubes are outlined by the following experimental data:

- tensile strength = 300 to 500 MPa;
- shear strength = 9 to 11 MPa;
- elasticity modulus = 12000 to 20000 MPa.

The testing operational scheme is shown in fig. 11:

two working faces 30 m. apart, length of consolidated sections equal to 5 m. on the Rome side and 15 m. on the Florence side.

The four tubes have been fitted into the holes so as to enable straingauge readings on the Rome face, during excavation from the Florence side (Fig. 16).

This operation assured the integrity of connections between strain gauges and control station, even during excavation.

5.2 Considerations on control data and interpretation

The strain measurement have been carried out at an average time interval of about one hour.

However, only the readings without excavation in progress have been analyzed (ever 5 hours on an average), thus neglecting the data possibily influenced by the excavator action.

These results are represented in the graphs of fig. 17, each of them related to a single instrumented tube.

The axial strains and corresponding stress are plotted versus the distance of measuring marks from the working face (taken as the origin).

We remark that the external surface of tubes No 1-2 had been treated in order to improve the tube-cement grout adhesion, while the surface of tubes No 3-4 was normally smooth.

The calibration of single instrumented tubes gave the following results: a deformation of 300 $\mu\varepsilon$ corresponds to a tensile force equal to 1000 kgf, acting on a tube section 15.7 sq.cm; hence the resulting tensile stress is about 64 Kg/sq.cm.

The following deduction can be drawn from the analysis of the graphs shown in fig. 17

Reading 1: working face position: 0 m.
elapsed time: = 11 hrs

In tubes No 1 and No 3, respectively with improved adhesion and smooth, the strain-gauge No 2 placed at 4 m distance from the faces has recorded a deformation of about 100 $\mu\varepsilon(\sigma=21$ Kg/sq.cm);in tubes No 2 and No 4 at the same distance from the working face the strains are lower than 50 $\mu\varepsilon$.

Reading 2: working face position: 1st m.
elapsed time: DT1 = 17 hrs

In both improved adhesion tubes (No 1-2) the max. strain is recorded by mark No 3, placed at 6.5 m. from the initial face position, corresponding to 5.5 m from the actual working face.

The measured values are 545 $\mu\varepsilon(\sigma=115.7$ Kg/sq.cm) and 420 $\mu\varepsilon(\sigma=88$ Kg/sq.cm) respectively.

In strain-gauges No 2 (the closest ones to the actual face, l = 3 m) the measured values are lower; the same occurs in strain-gauges No 4-5-6.

The behaviour of smooth tubes is different; in fact the max. deformation is recorded by strain-gauge No 2 that is the closest to the face (l = 3 m) and the trend of deformation is to decrease with increasing distance from the face.

The max. strain is 1.141 $\mu\varepsilon(\sigma=240$ Kg/sq.cm) in tube No and 459 $\mu\varepsilon(\sigma=96$ Kg/sq.cm) in tube

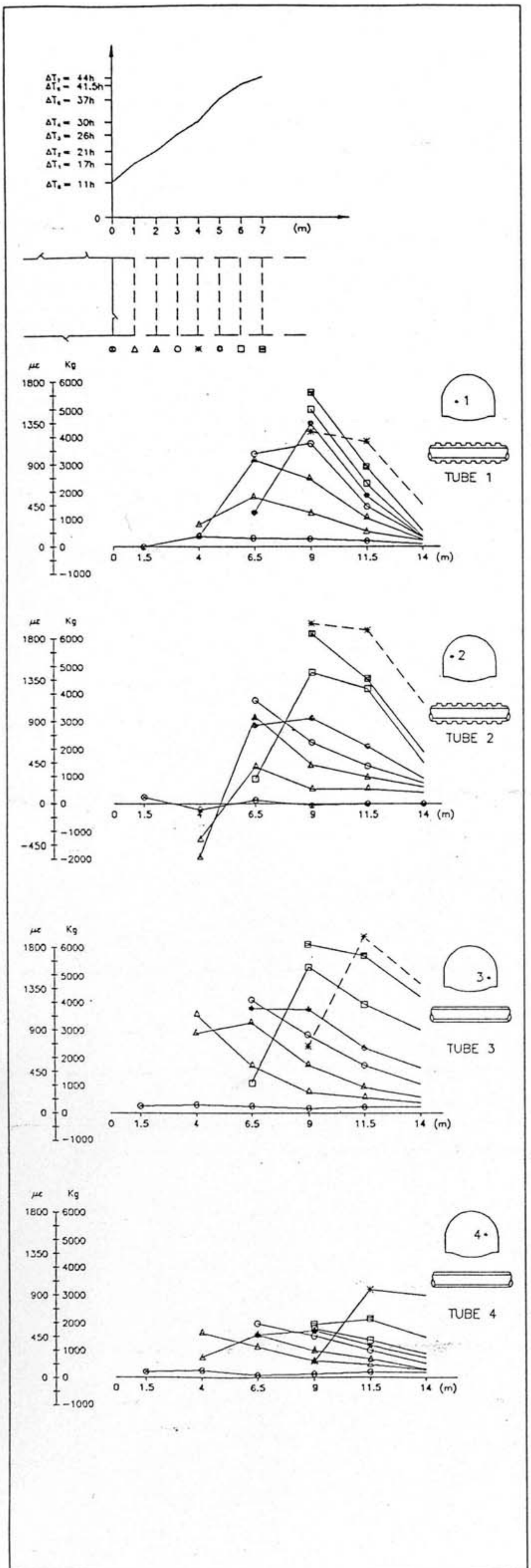


Fig. 17: Results of strain measurement in instrumented fiberglass tubes.

No 4.

Reading 3: working face position: 2nd m.
elapsed time: DT2 = 21 hrs

In both improved adhesion tubes (No 1-2) the max. strain is still recorded by mark No 3, 6.5 m. distant from the initial face position, and 4.5 m. fron the actual working face.

The measured values are 961 $\mu\varepsilon(\sigma=202$ Kg/sq.cm) and 933 $\mu\varepsilon(\sigma=196$ Kg/sq.cm) respectuvely.

Also in smooth tubes the highest strain has been measured by strain-gauge No 3; however the two values, differently from the case of improved adhesion tubes, are one the double of the other one (1072 $\mu\varepsilon$ in tube No 3, against 440 $\mu\varepsilon$ in tube No 4).

From reading 4 to 7:
working face position 3rd to 6th m
elapsed time: DT6 = 41 hrs

The strain behaviour is characterised by a max. value recorded by strain-gauge No 4, placed 3 m ahead of the face, and by progressively decreasing values with increasing distance from the working face.

The max. strain values measured by strain-gauge No 4 are as follows:

tube No 1	$\varepsilon=1672\ \mu\varepsilon$	$\sigma=357$ Kg/cm^2
tube No 2	$\varepsilon=1895\ \mu\varepsilon$	$\sigma=404$ Kg/cm^2
tube No 3	$\varepsilon=1874\ \mu\varepsilon$	$\sigma=400$ Kg/cm^2
tube No 4	$\varepsilon=600\ \mu\varepsilon$	$\sigma=128$ Kg/cm^2

We point out that the highest strain is recorded by tube No 2, that is of the improved adhesion type, while the lowest refers to the smooth tube No 4; this value is still 1/3 of the strain measured in the same position of tube No 3.

Reading 8: working face position:6th-7th m
elapsed time: DT7 = 44 hrs

This measurement has been made during exacavation of the seventh metre, differently from the previous readings that were at rest.

The measured strains had the same order of magnitude as those recorded at the end of the 6th metre excavation.

On the contrary, after the 7th metre and also in the subsequent 3 m of excavation, up to the end of the planned experimental stretch, the deformations recorded by the remaining strain-gauge No 4-5-6 underwent a remarkable decrease.

In particular, the strain-gauge No 4, after the 7th m excavation, shows a 50% strain reduction with reference to the value recorded after completion of the 6th metre.

The reason is likely related to the reduced tube-soil adhesion along the residual tube lenght (2 m) between the strain-gauge and the working face.

The above detailed analysis of experimental data can be summarized as follows:

- the tubes of "improved adesion" type have shown a similar behaviour both in qualitative and quantitative terms, while the smooth tubes had undergone quite different strains (50% scattering);
- the max. axial strain (corresponding to the max. tensile stress) has been measured at 3 m distance from the working face, when this is at the 6th m of excavation, that is beyond the stretch of core consolidated by 100 tubes (50 tubes 5 m long and 50 tubes 15 m long).

Considering the highest strains measured in the first three tubes and their average that is 1813 $\mu\varepsilon$(σ=385 Kg/sq.cm against 3000-5000 at failure) the applied force on the tube section is 6043 Kg; this force, due to extrusive core action, is transferred to the tube through adhesion stresses t developing along the tube lenght between the working face and the strain gauge No 4; this adhesion comes out to be on an average: t=3.19 Kg/sq.cm.

6 CONCLUSIVE CONSIDERATIONS

The measurements, studies and observations during the excavation of Poggio Orlandi Tunnel (850 m long) about the performance of full face excavation and core preconsolidation by fiberglass tubes, have enabled to improve the theoretical knowledge of this particularly new static subject.

The developement of this knowledge and the application to the design and construction of more than 11 km of tunnels to date, confirm the full reliability of the working face preconsolidation procedure and as well of the full face excavation in soft ground.

Even in the most difficult geotechnical conditions, construction times and costs could be maintained as expected in the design stage, obtaining the industrialization of underground excavations, to the best advantage of both owners and contractors.

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OWNER:	Ferrovie dello Stato
GENERAL CONTRACTOR:	Fespi-Fondedile S.p.A.
SUB-CONTRACTOR:	Italsonda S.p.A.
DESIGN:	Rocksoil S.p.A.

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