Geotechnical Aspects of Underground Construction in Soft Ground, Mair & Taylor (eds)
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Experimental and numerical tests on the excavation of a railway tunnel in grouted soil in Milan

G.Canetta
Centro di Analisi Strutturale srl, Milan, Italy

B.Cavagna
MM Strutture e Infrastrutture del Territorio S.p.A., Milan, Italy

R.Nova
Politecnico di Milano, Italy

ABSTRACT: The paper presents the results of the experimental investigations aimed at the mechanical characterization of the soil in which a subway tunnel had to be excavated. Since extensive use was made of cement grout injections, both the natural and the improved soil behaviour were studied in the laboratory and by means of in situ tests. The pattern of soil movements during the successive excavation phases was monitored and the experimental results were compared with the results of numerical back-analyses. It is shown that by means of the procedures adopted reasonable predictions of soil movements are possible.

1. INTRODUCTION

The soil of Milan is constituted by an alluvial deposit of coarse sand and gravel to a depth of many dozens of meters from the surface. In such a granular non-cohesive medium, railway and subway tunnels are usually excavated by means of a blind hole technique, after the soil around the tunnel arch, and often below the invert, has been systematically improved by cement grout injections. The construction technique is now well established, having led to the realization of several dozens of kilometers of the Milan Subway, but some doubts still exist on the amount of grout needed to reach a certain degree of soil improvement and on the soil movements associated with the various phases of the construction process.

In order to achieve a better understanding, the constructor of Milan's underground railways, MM Strutture e Infrastrutture del Territorio SpA, promoted an extensive research directed to the definition of a procedure capable of arriving at a reasonable prediction of soil movements, which is crucial in an urban environment. The paper reports on the activities conducted in this framework.

2. SITE INVESTIGATION

A trunk of the 3rd line of Milan Subway was selected for the experimental investigation and the calibration of the settlement prediction procedure. A section of the tunnel to be constructed is shown in fig. 1, together with the soil description retrieved from the logs of several boreholes. The soil comprises alternate layers of sandy gravel and gravelly sand, with some finer inclusions of sandy silt. The water table is found at about 20m from soil surface, that is at about the depth of the tunnel invert.

The soil and water table conditions are the typical ones in which the Milan subway tunnel are excavated. Also geometry (typical 2-tracks tunnel)

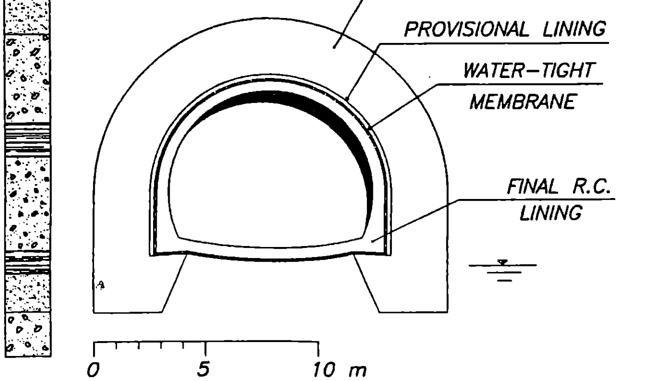

Fig. 1. Typical tunnel section and soil profile

and environmental conditions (flat surface, with buildings regularly aligned at both sides of the tunnel) are ideal for testing.

In order to limit soil movements to a minimum, the usual construction procedure is as follows (fig. 2):

a) excavation of a pilot tunnel, about three meters in diameter, supported by means of steel ribs and shotcrete or wooden planking;

b) improvement of a soil ring around the excavation profile (crown and sidewall and, in many cases, also invert arch) by means of cement grout injections; the injection pipes are inserted into the ground either from the pilot tunnel, or from the surface, or both;

c) excavation of the crown arch and installation of a provisional liner (usually steel ribs and wire mesh reinforced shotcrete)

d) excavation of the bench and the invert, underpinning the liner at the sidewalls;

e) casting of the final concrete lining.

Soil movements can be computed by means of a FEM analysis which, starting from the geostatic conditions, simulates the various excavation phases.

In order to characterize the mechanical properties of the soil in situ, a series of pressuremeter tests were conducted at 12m ad 18m depths. Soil is rather stiff (the apparent elastic shear modulus at 18m depth is of the order of 35MPa) and there is an abundance of boulders within the soil, so that it is impossible to use a self-boring pressuremeter. A dilatometer, usually employed in rocks, was used instead. The soil is therefore disturbed in the first loading phase and the initial horizontal stress is determined with low accuracy. A value for K_o close to 0.45 seems to be appropriate.

The average pressuremeter curve was then analyzed by means of the procedure suggested by Hughes et al. (1977). The friction angle at critical state was first determined in drained triaxial tests on samples of average granulometric characteristics and low density. An average value of 35° was chosen. The friction angle and the dilatancy of the soil in situ were then determined from the slope of the pressuremeter curve in a log-log plot and the assumption that the friction angle ϕ, friction angle at constant volume ϕ_{cv} and dilatancy angle ψ are linked by (Bolton (1986)):

$$\phi = \phi_{cv} + 0.8\ \psi$$

Angles of 37.6° and 3.2° were determined for friction and dilatancy angles, respectively.

From the unloading - reloading curve the elastic shear modulus was determined to be of the order of 140 MPa, 4 times the initial one, a typical value for

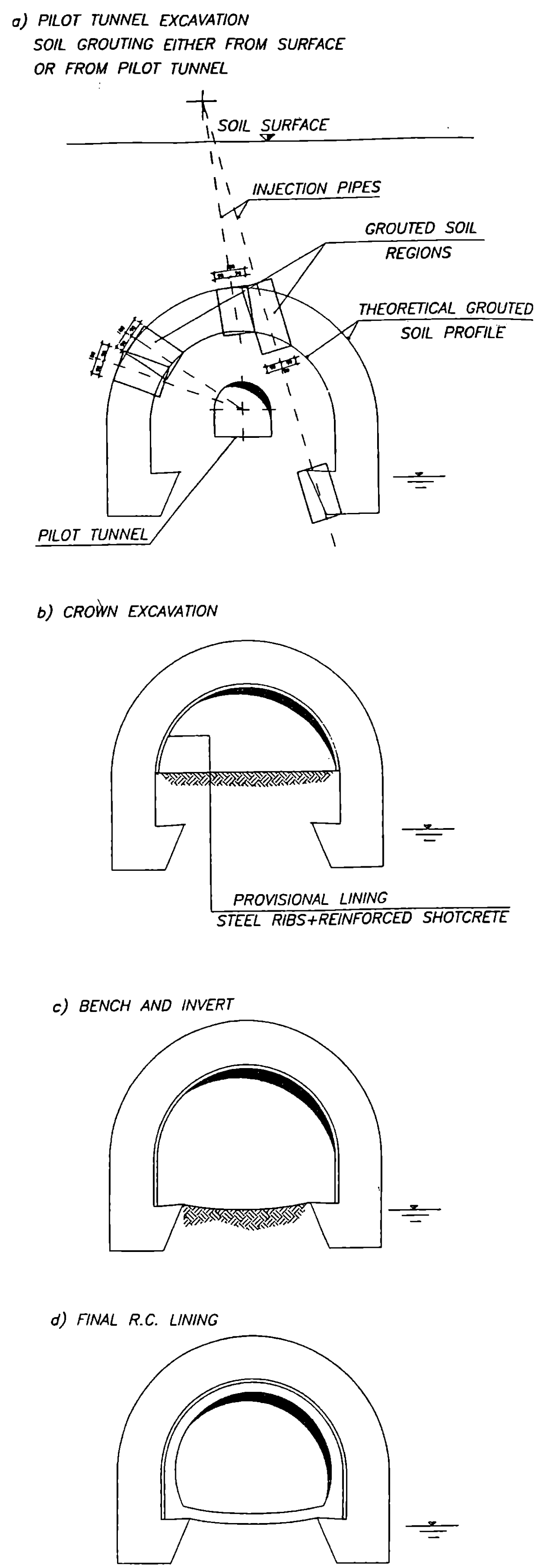

Fig. 2. Typical construction sequence

granular materials (Bellotti et al. (1986)).

A series of pressuremeter tests were also conducted on the grouted soil, with much less success, however. Results are randomly dispersed. Two of the tests showed a very irregular behaviour, two others show results which are very close to those of the natural soil, as if the improvement had no effect, and only two show a relevant increase in stiffness and overall strength. Such tests were not analyzed in detail because of their irregularity. They are an index of the non uniform treatment of the soil. Such irregular improvement was also shown by a series of sonic tomographies, performed in a companion research aimed at determining the extension of soil improvement (de Franco and Cavagna (1995)).

3. LABORATORY TESTS

In addition to the triaxial tests on the natural soil, two extensive series of tests were performed on grouted sand specimens.

In the first series, sand was poured in a mold and then grout was injected from below at low pressure in laboratory. The grout permeated uniformly the specimens, so that high reproducibility of results was obtained. Such specimens are therefore representative of an ideal situation in which the grout fills most of the voids and transforms a granular material in a soft rock. The cemented material is stiffer and stronger than the natural soil. In particular, the soil acquires a cohesion, which varies with the curing time. The friction and dilatancy angles also increase. di Prisco et al. (1992) show that the observed behaviour fits well with that of naturally cemented materials, so that a conceptual model such as that of Gens and Nova (1993) seems to be appropriate for describing its mechanical behaviour.

The second series of tests was performed on specimens obtained by sampling the consolidated material in situ. Considerable problems were encountered in coring directly the soil from boreholes, since the disturbing destroyed almost completely the cementation. On the other hand, specimens obtained from cemented soil blocks, excavated during tunnel construction, were characterized by a stiffness and a cohesion one order of magnitude larger than those of the specimens grouted in laboratory, although the cement grout was the same. The explanation of such a paradox is that in situ the permeation of grout through soil is not at all uniform. By visual inspection of the sidewalls of the tunnel, during bench excavation, it was possible to note the existence of a periodic laminar structure, created by the grout injections, having a period corresponding to the values of the tubes 'á manchettes' used for the injections.The specimens of the second series were actually specimens of the cementing grout itself, and not of the grouted soil.

4. THE CONSTITUTIVE MODEL

The constitutive model adopted is an elastoplastic strain hardening model, called "X-Lamber" (Canetta and Nova, 1989). It is in a sense a simplified version of the Nova and Wood (1979) model, that describes the behaviour of granular soils in practical problems. This has obvious important consequences in that the number of constitutive parameters must be very limited and the flow rule must be associated, in order to avoid numerical instabilities with the ordinary structure of the continuum assumed. The constitutive parameters used in the model are the elastic bulk stiffness, assumed to vary linearly with the isotropic pressure, a constant Poisson's ratio, a parameter χ which gives the plastic logarithmic compressibility in isotropic loading, the stress ratio at critical state and the dilatancy at failure. The only non traditional parameter is χ, which was empirically found to be related to the stress ratio at failure, i.e., to the friction angle. Hardening is assumed to be isotropic as in Cam Clay (Schofield and Wroth (1968)).

By means of this model, it is possible to calculate the pressuremeter curve, assumed as a cylindrical cavity expansion (the aspect ratio of the pressuremeter being as large as 10). The results are relatively satisfactory, as it can be seen from fig. 3.

It is further assumed that grouted soil is described by a similar model. The effect of the grout

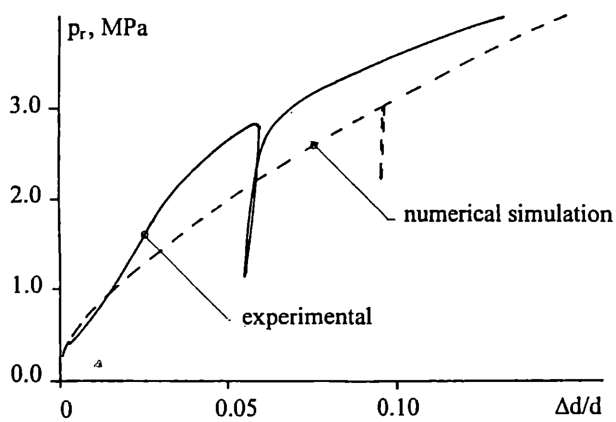

Fig. 3. Pressuremeter test results and numerical simulation.

is that of increasing the elastic stiffness and of widening the elastic domain. This is accomplished by introducing two new parameters, p_m and p_t as in fig. 4, after di Prisco et al. (1992). Actually, it is found that that p_t is directly proportional to p_m, the proportionality coefficient being of the order of 15, so that in addition to the natural soil constants only two other parameters need to be chosen.

It is worth noting that, from a numerical standpoint, it is quite convenient that natural and grouted soil are described by essentially the same model. The effect of the cementation at a constitutive level is in fact modelled simply by a change of parameters. Nevertheless, it should be emphasized that grout injection causes not only a change in material characteristics, but also induces soil displacements and generates a state of self-stress, quite difficult to model.

It was shown by di Prisco et al. (1992) that the essential features of the behaviour of a sand specimen uniformly permeated by cement grout are captured by the model X-Lamber. In this case, however, the permeation of in situ soil was rather non-uniform, and it is questionable whether such a modelling of the improved soil is appropriate. It will be shown in the following that, for engineering purposes, reasonable results can be obtained even in this case.

5. THE NUMERICAL MODEL

The excavation of the tunnel was modelled by the following phases:

a) initial geostatic conditions: the unit weight of the soil of 18 kN/m^3 above groundwater level and the effective unit weight of 11 kN/m^3 below are applied along with surface loads, and internal effective stresses within soil mass are initialized to the geostatic values, with an at rest pressure coefficient K_o equal to 0.45. The system reaches equilibrium in a unique elastic step, with negligible displacements.

b) excavation of the pilot tunnel: the finite elements within the excavation profile are removed, and the pressures exerted by them along the excavation boundary are progressively reduced. This phase and the following are assumed to take place in plane strain conditions, for the sake of simplicity. The end of this step coincides with the initial conditions of the in-situ monitoring of the excavation phases.

c) grouting: the mechanical characteristics of the material are updated first; next an isotropic anelastic strain is imposed in the grouted regions so that a field of self-stresses is generated.

d) excavation of the crown arch: the equivalent pressures at the excavation boundary are first reduced to 70% of their value, then the stiffness of the provisional liner is inserted and boundary pressures are reduced to zero. This procedure takes account of the actual three-dimensionality of the excavation procedure. The initial value of boundary pressure reduction prior to liner insertion, was calibrated in previous back-analyses (Botti et al. (1988), Balossi Restelli et al. (1989)).

e) excavation of the bench and of the invert, with a similar technique to simulate the construction of the provisional lining of the sidewalls;

The construction of the final concrete lining is of no interest, in the numerical simulation, as in the initial phases of its life it carries only its self-weight.

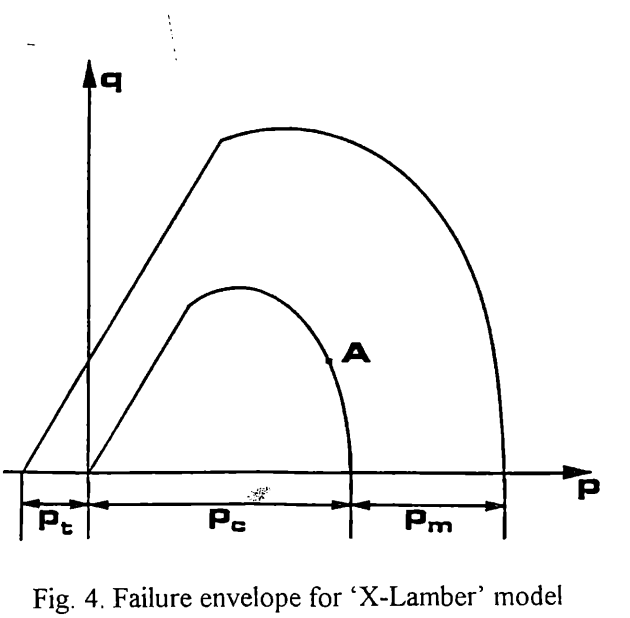

Fig. 4. Failure envelope for 'X-Lamber' model

6. COMPARISON WITH THE IN-SITU MEASUREMENTS

Soil measurements were measured at the surface by means of survey levelling and along six boreholes by means of multibase extensometers-inclinometers (Trivec), positioned along vertical alignments that cross the grouted soil at both sidewalls of three tunnel sections. The former type of measurements gives a result of direct design interest, since possible damages to nearby buildings are strictly correlated to movements of surface soil layers. The latter one gives more information about the way in which the surface settlements are generated, separating in particular the contribution of improved and natural underlying soil, which is overloaded by the field of self-stresses generated by the grouting process and by the crown excavation. Predictions of soil movements were attempted with the set of parameters retrieved from the analysis of the pressuremeter curve. The computed settlements of the free surface were systematically lower that those observed during the excavation of the tunnel. A back-analysis was then performed and the elastic stiffness of the soil was modelled accordingly, keeping other constitutive parameters unchanged. The value which better fitted the experimental data was very close to the value used in previous back-analyses performed on different tunnel sections, excavated in other parts of the town, where pressuremeter tests were not available.

The difference between the stiffness retrieved from the pressuremeter tests and that obtained from the back-analyses may be due to the different duration of the two phenomena. The much longer excavation process (one month) leaves time for soil deformation to occur, while the shorter pressuremeter test (one hour) shows an apparently stiffer soil, because sand grains have no time to rearrange their packing. Similar time-dependent phenomena have been observed also in sands in the laboratory, under carefully stress-controlled tests, by di Prisco and Imposimato (1996).

The elastic stiffness of the grouted soil was back-computed to be only twice as large as the stiffness of the natural soil. The stiffness of the soil fully permeated by grout was found to be larger than that. It is possible that the observed increase in stiffness is only due to a reduction of the volume of compressible soil. The laminae of grouted soil can be considered as rigid, in comparison with the natural soil. The compressibility of a macroelement such as that shown in fig. 5 depends on the compressibility of natural soil and is inversely proportional to the ratio of the volume occupied by the laminae to the global volume of the macroelement. Since such a ratio is of the order of 50%, the overall stiffness turns out ot be twice as much as that of the unreinforced soil. Clearly, this is true only in a direction orthogonal to the laminae, i.e., in vertical direction; this is the relevant one, however, for the problem at hand.

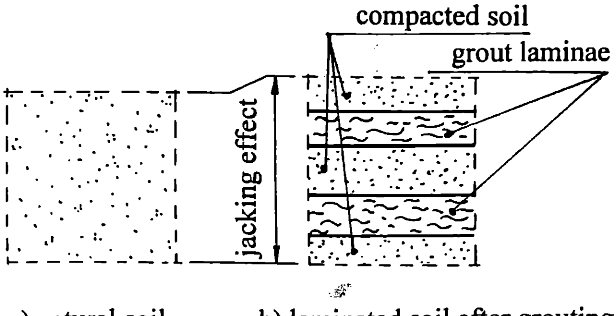

Fig. 5. Soil macroelement before and after grouting

The numerical model captures rather well the trend of the surface movements, as shown in fig. 6 for the crown excavation phase. The qualitative description of the (vertical) strains within the grouted soil is fairly good, as shown in fig. 7.

What is not accurately predictable, even in the back-analysis, is the heave associated with the grouting process. Presumably, this is due to the fact that such operation was modelled by means of an isotropic anelastic straining of the grouted zone. Instead of that, the effect of the grout injections, due

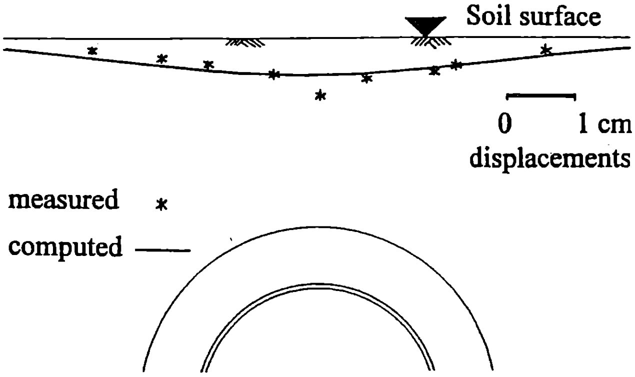

Fig. 6. Measured vs. computed surface displacements

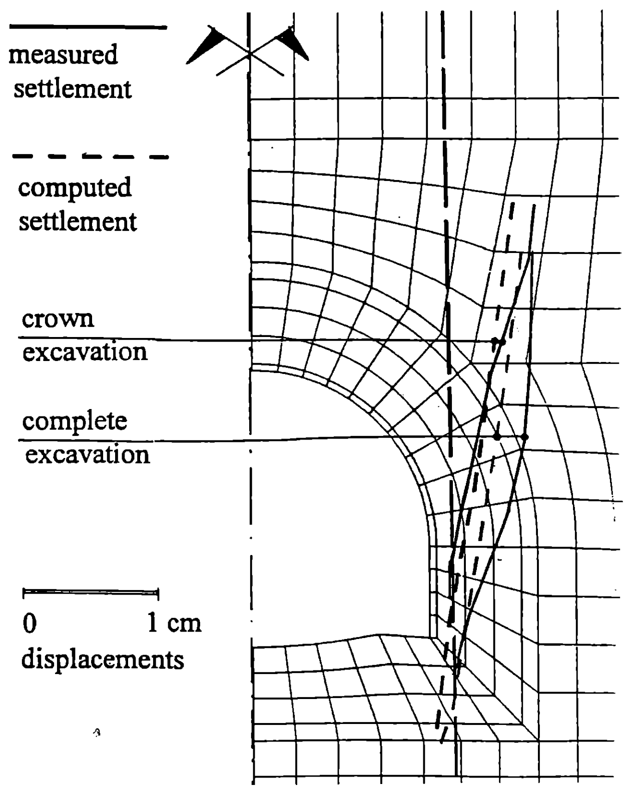

Fig. 7. Vertical displacements within soil mass. Measured vs. computed.

to the systematic formation of horizontal grout laminae within the soil, was more similar to the effect of a series of hydraulic jacks. The possibility of describing grouting in this way will be investigated in the near future.

7. CONCLUSIONS

The paper presents the results of an investigation, both experimental and numerical, on the soil movements during the excavation of a soil in a ground partially improved by grouting. It has been shown that it is possible to reproduce numerically the observed displacements, with an acceptable degree of accuracy at least, by:

a) modelling the numerical behaviour of soil with an elasto-plastic strain-hardening model called "X-Lamber" whose 5 constitutive parameters are simply linkable to traditional soil constants. Of fundamental importance for accurate predictions is the value of the elastic stiffness. On the basis of various back-analyses this was estabilished to be much lower than that retrievable from the pressuremeters tests conducted. A possible explanation for this lies in the different duration of the two loading processes of soil (the test is much faster than the crown excavation process).

b) modelling the mechanical behaviour of the grouted soil with a model which is an extension of the model for the natural soil; the new parameters are an enhanced elastic stiffness (twice as much as that of the natural soil in the observed case), and two mutually linked parameters which are related on the one hand to the tensile strength of the grouted soil and to a widening of the elastic domain on the other.

c) modelling the various phases of the excavation as listed in section 5. Of particular interest is the simulation of the effect of the provisional liner, installed immediately behind the excavation face.

On the whole, the results appear to be satisfactory, with the exception of grouting, as far as vertical movements are concerned. In the case considered, the grouted injections acted mainly as hydraulic jacks, causing mainly vertical displacements. In the numerical simulation, grouting was modelled by imposing an inelastic volumetric strain, as if the soil were uniformly permeated, which is certainly reasonable for more permeable soils, but evidently not correct for the type of sand at hand.

It should be noted, however, that such a procedure of settlement calculation has been successfully employed only in the soil of Milan, a coarse granular material with a relatively high friction angle but small dilatancy. For other types of soils, the constitutive model, characterized by an associated flow rule, could be inappropriate.

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