A BACK-ANALYSIS OF AN UNDERGROUND TUNNEL IN GROUTED SAND

BACK-ANALYSIS DI UNA GALLERIA URBANA IN TERRENI SABBIOSI CONSOLIDATI

G. Canetta, CeAS Milan
R. Conta, MM Strutture e Infrastrutture del Territorio SpA, Milan
R. Nova, Milan University of Technology (Politecnico)

The paper presents a backanalysis of the data measured during the excavation of an underground tunnel in alluvial soil improved by grouting. Such data are compared with the results obtained by means of two finite element analyses.

The analyses (the former used as a design tool in the construction phase, the latter as a backanalysis performed after tunnel completion), differ in the constitutive models which are employed to describe the behaviour of grouted soil.

The former is a conventional elastic-perfectly plastic law, while the latter is based on the strainhardening theory of plasticity. It is shown that the latter gives results which are closer to experimental evidence and is numerically more convenient, since the phenomenon of grout curing can be modelled by a gradual improvement of the numerical characteristics of the material.

L'articolo presenta un'analisi a posteriori (back-analysis) di una serie di dati osservati durante lo scavo di una galleria in terreno alluvionale, le cui caratteristiche meccaniche sono state migliorate per mezzo di iniezioni. Durante lo scavo della Stazione Repubblica del Passante Ferroviario Vittoria - Garibaldi a Milano, e' stata effettuata una vasta campagna di misure di cedimenti superificali e delle fondazioni di un attiguo edificio di oltre 100m di altezza; misure di deformazione nel terreno normale e consolidato sono state inoltre ottenute mediante l'impiego di "sliding-micrometers" (vedi Figure 1 e 2).

Questi dati sono stati confrontati con i risultati ottenuti per mezzo di due studi agli elementi finiti. Le analisi numeriche (la prima impiegata come strumento progettuale durante lo scavo della galleria, la seconda come verifica a posteriori a scavo completato) si differenziano nel tipo di modello matematico impiegato per descrivere il comportamento meccanico del terreno iniettato. Il primo e' un tradizionale modello elastico-perfettamente plastico, mentre il secondo e' basato sulla teoria della plasticita' con incrudimento. Si mostra che quest'ultimo modello fornisce risultati piu' vicini ai dati osservati (vedi Figure 4, 5, 6); esso inoltre e' piu' conveniente dal punto di vista numerico perche' il fenomeno dell'indurimento della miscela puo' essere descritto con un graduale incremento delle caratteristiche meccaniche dei materiali e perche' il modello costitutivo con incrudimento consente una rappresentazione piu' accurata del comportamento meccanico del terreno consolidato.

1. INTRODUCTION

In order to excavate the tunnels of the new underground lines in Milan, without inducing excessive settlements on the nearby structures, it is common practice to improve the mechanical characteristics of the natural soil, a coarsely grained sand or sandy gravel of alluvial origin, by injecting grout in the soil pores. The extent of soil improvement is at present largely dependent on empirical considerations. Only scanty data are in fact available so far of the displacement pattern within the soil mass and at the surface, and of the pressures acting on the tunnel liner.

An extensive research programme was therefore planned by MM, the general contractor for underground transportation in Milan, aiming at a deeper knowledge of the grouting process, its effects on various types of loose soils, and its numerical simulation, with the final goal of a more effective design. A series of in situ measurements was carried out on a tunnel segment; the interpretation of the measurements was attempted by means of finite element analyses in two different phases:

- during the early phases of the tunnel construction, a set of analyses was carried out (see Section 3) as a tool to optimize the design of grouting and excavation procedure and to predict settlements of the nearby buildings;

- after tunnel completion, a set of backanalyses was run (see section 4) to assess the capability of the numerical model to represent the actual behaviour.

In both types of analysis the behaviour of natural soil is modelled by means of an elastic-plastic strainhardening law. In the former, however, the behaviour of the grouted soil was described by an elastic-perfectly plastic law, with a Drucker-Prager yield criterion. The transition from natural to grouted soil was therefore modelled by an abrupt change of behaviour of the elements that represent the improved soil. Such a sharp transition caused unrealistic changes in the state of stress and strain both in the grouted and in the natural soil around it.

In order to overcome this difficulty, a new constitutive model was conceived for grouted sand. Such a model has the same structure as that used for virgin soil. It is assumed, however, that along with the curing of the grout, the initial yield surface grows in size and the tensile strength of the improved soil increases.

In this way the effect of grouting can be modelled by means of a gradual modification of the constitutive parameters of the soil, leaving unchanged the structure of the constitutive law itself. As a result, the numerical analysis is more robust. The actual behaviour of grouted soil is also better modelled by a constitutive law with a strainhardening structure as shown in a companion paper by di Prisco et al. (1991).

The paper presents the measurements recorded during the excavation of a tunnel under a tall building in Milan, and the results of the numerical analyses. It is shown that with a convenient choice of the constitutive parameters, the strainhardening model is able to reproduce the observed behaviour with a resonable accuracy in the whole section for all construction phases.

2. PROBLEM STATEMENT

A station of the new Underground Railway Link, presently under construction in Milan, was to be excavated in the proximity of the foundations of a 29 floor high storey building. The building is founded on a raft resting on a deep deposit of alluvial soil, with alternating layers of sandy gravel and sand. The tunnel to be excavated was 25m wide and only limited displacements of the raft were admissible. It was then decided to improve soil characteristics by the extensive use of grouting. The grouting procedure should be carefully designed, however, in order to avoid excessive heave of the building and a dangerous increase of the state of stress in the tensile reinforcements of the raft. To keep the soil movement under control, a series of datum points, sliding micrometers and inclinometers were positioned prior to the tunnel excavation. A numerical analysis was performed to predict the displacement trend during the subsequent construction phases. The model employed and the way in which constitutive parameters were determined are discussed in section 3. The numerical predictions of surface displacements were reasonably close to the observations. The displacements within the soil mass, however, were less accurately predicted, especially those concerning the grouting phase. It was then decided to backanalyse the problem with a different constitutive model, which is presented in section 4. Section 5 presents a comparison between the results of the two analyses and the experimental data recorded.

3. DESIGN PHASE ANALYSIS

A set of finite element analyses was carried out in the construction phase of the tunnel; the main goal of such analyses was to predict the settlements and the stresses induced during the excavation in the raft of the multi-storey building close to the station tunnel. Three were the major construction phases which could induce modifications in the stress and strain state in the building raft and surrounding soil:

- tensioning and subsequent detensioning of the ties of a diaphragm wall in a nearby open cut excavation
- grouting below the raft (differential heave of the raft), and
- crown excavation (differential settlements).

Constitutive parameters were needed both for natural soil and for grouted soil; in order to obtain such parameters, some backanalyses were run in this phase on already excavated tunnels, of a nearby segment of the underground railway, which were characterized by simpler geometrical conditions. Moreover, a backanalysis was required of the open-cut excavation with diaphragm and ties, which was already at the final stage, in order to assess the modifications induced in the foundation soil.

The parameters determined with the aforementioned backanalysis were then chosen as representative of that type of soil. It was then possible to analyze the critical section of the tunnel, directly below the foundation raft, and to optimize the grouting profile and the excavation phases.

3.1 Results of the former backanalyses

For natural soil, the constitutive parameters to be evaluated were the following (material model: elastic-plastic strainhardening):

- bulk and shear moduli for unloading/reloading
- logarithmic compliance under virgin compression
- friction angle and dilatancy at failure
- maximum past consolidation pressure, as a function of soil history; this parameter could be influenced by building erection and open-cut excavation.

For grouted soil (elastic-perfectly plastic) the constitutive parameters were simply elastic bulk and shear moduli, cohesion and friction angle. However the soil heave induced by grouting pressures had to be reproduced in order to evaluate self-equilibrated stresses within the soil mass and stresses induced in the building raft. This phenomenon was simulated by means of an anelastic isotropic expansion applied to the grouted zone.

Avaliable data from the in situ measurements were:

- surface and building foundation displacements, measured by means of a network of datum points;

- strains within soil mass, along the axis of sliding micrometers; strains were measured for the grouted arch in all construction phases; strains above the crown arch were instead retrieved until the excavation front reached the instrumented section, cutting the instrument.

The results of the backanalyses, thoroughly reported elsewhere (Balossi Restelli et al., (1989)), are summarized below:

- the grouting process, which is simulated by an abrupt change of soil characteristics, can be modelled in a rather approximate way: either grouting heave is underestimated, or crown excavation behaviour will not be correctly matched. In such analyses it was chosen to refrain from reproducing soil heave during grouting phase, in order to obtain, with the same set of parameters, reasonable results in the more important steps of the analysis;

- different constitutive parameters are required for grouted soil depending on the grain size of the initial soil: while in coarse sands and sandy gravels it is possible to attain a homogeneous grouted soil of good mechanical characteristics, in fine sands the overall performance is worse, and the results can be higly inhomogeneous, with local soil rupture and the formation of grout lenses which act as a flat jack (claquage). As a consequence, the phenomenon of soil heave is more pronounced in sandy than in gravelly layers;

- the open cut excavation is not responsible of extensive modification of the soil state, as appreciable changes only occur in soil surrounding tie anchors; indeed, due to the accurate sequence of tie tensioning and lowering of the excavation, very small displacements occur for the diaphragm walls and therefore the state of the foundation soil undergoes little changes.

3.2 Stress-analysis of the main tunnel section

The main characteristic of this tunnel section is the direct interaction between the raft and the grouted soil. The construction phases are summarized in Fig. 1; the main steps of the process are the following:

1. grouting all around the tunnel section and below the foundation raft;
2. excavation of two side-wall drifts and casting of the side-walls of the main tunnel;
3. crown excavation and casting of the crown arch
4. lowering and casting of the invert.

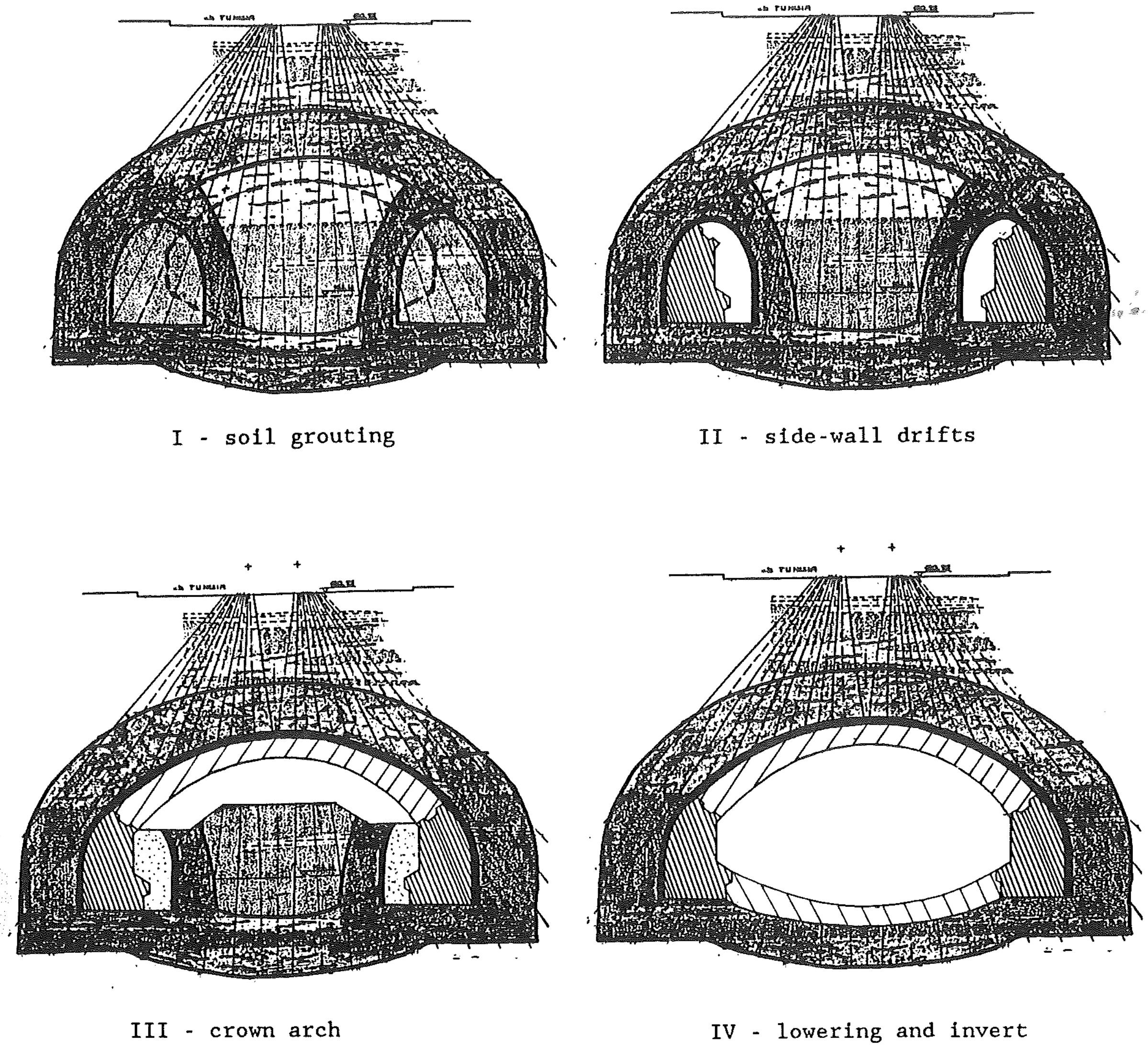
The results of the stress-analyses were used to define those phases of the process which were likely to produce maximum effects on the raft. Different solutions were compared a) for the main grouted arch, and b) for the extension of grouting below the raft. Such results are reported elsewhere (Amagliani et al. (1991)). The numerical analyses allowed the grouting and excavation process to be optimized as follows (see also Fig. #):

1. grouting from soil surface at the sides and above the crown of the main arch and of the side-wall drift arches, and below the invert; this was likely to produce soil heave above the crown arch, but to have little effects on the foundation raft; the usage of jet-grouting columns below the foundation was discarded, since it was found that such columns acted as stress concentrators causing important settlements close to the raft;

2. excavation and lining of the upper half of the side-wall drift below the foundation; in order to minimize settlements of the raft in this phase, the lower part of the drift had been grouted to minimize horizontal convergence of the drift walls; the resulting drift was large enough to perform successive grouting directly from it (see Fig. 2);

3. second-phase grouting below the foundation raft, from the excavated drift; the grouted zone was extended below the core of the raft in order to avoid an excess of deformation of the edge of the raft; a particularly "soft" grouting procedure was adopted in this zone (Balossi Restelli et al., (1989)), with particular cement grout and silicate mixtures, and continuous monitoring through instruments positioned in the raft;

4. crown excavation and subsequent lining at a short distance from the excavation front, to reduce crown convergence.



I - soil grouting

II - side-wall drifts

III - crown arch

IV - lowering and invert

Figure 1. Typical excavation scheme

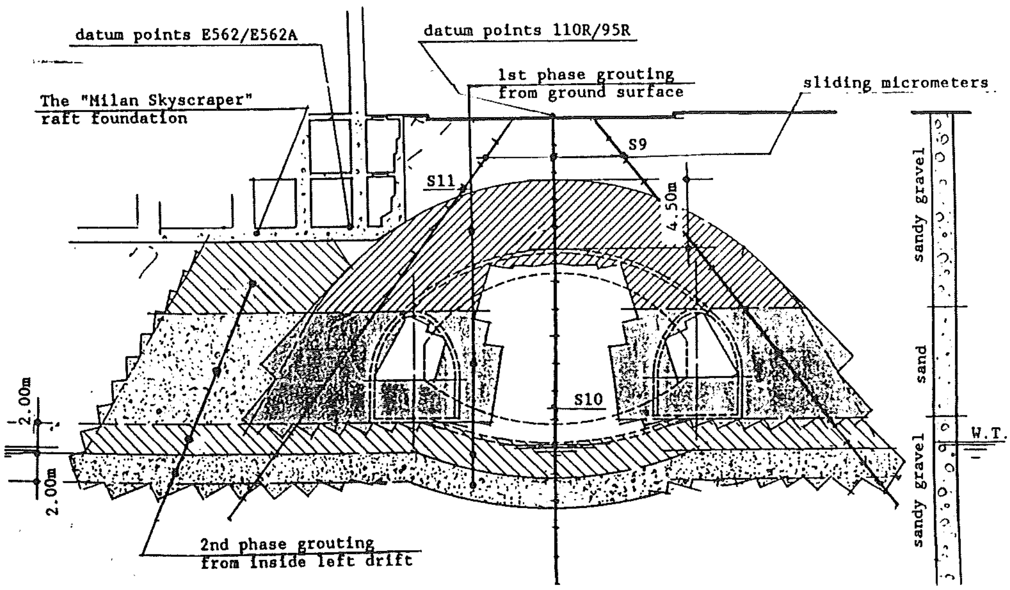


Figure 2. Grouting sequence below the foundation raft

3.3 Conclusions on the modelling of grouted soil

The overall behaviour of the model was quite good; it gave good predictions for the main construction steps, both for soil surface displacements and for raft settlements. This was evidenced by the comparison of the computed settlements with the measured ones (Balossi Restelli et al.,(1989)); some of such results are presented in the last sections of this paper, along with the results of the latest backanalyses.

The model failed completely, however, in representing soil heave in the grouting phase, as already mentioned. More generally, the calculated behaviour of grouted soil was only partially satisfactory: in fact, only the crown excavation phase is well reproduced, while other steps, e.g. side drift excavation, are not; this became evident when the latest backanalyses were carried out on the basis of available in situ measurements, as shown in the next sections.

The main reasons of this result are two:

1. the grouting phase is represented as a single-step soil modification, with an anelastic isotropic expansion which creates prestressed arches within the grouted soil; such process is quite rough, especially for a complex geometry as is the case;

2. in the next steps, the behaviour of the grouted soil is essentially elastic, as plastic zones are quite limited. Therefore, the same stiffness moduli were used both for those regions where unloading/reloading effectively takes place (like at the side walls), and for those where virgin compression actually occur (like in the crown arch). As a consequence, an accurate simulation of the former behaviour will lead to an overestimate of the stiffness for the latter, and vice versa.

4. BACKANALYSIS

In the back-analysis performed after the work completion, the model for the natural soil was improved by varying the hardening law (Canetta and Nova (1989)) which was assumed to depend on both volumetric and deviatoric plastic strains. This allows the actual soil dilatancy to be better reproduced. The most important novelty was, however, the new development of a consititutive law for the grouted material.

It was in fact assumed that the grouting process alters the soil characteristics in two ways: it improves the elastic moduli and expands the region of elastic behaviour (elastic domain). In Fig. 3 a sketch of such modification is plotted. Point A represents the stress state in the natural condition; since the soil is assumed to be in a normally consolidated state, point A lies on the yield surface which is characterized by an isotropic pressure p_c. After grouting, the elastic domain is larger: the maximum isotropic pressure is now:

$$\bar{p}_c = p_c + p_m \qquad (1)$$

and the material acquires a tensile strength, which is related to the parameter p_t, defined in Fig. 3.

Four new parameters should then be identified: the two improved elastic moduli and p_m and p_t, while it is assumed that the other plastic parameters do not change.

This model has several advantages on the former one. First, it reproduces well, at least qualitatively, the observed behaviour of grouted sand in laboratory tests. Secondly the transition from the natural state to the fully grouted state can be simulated by gradually increasing the elastic moduli and the plastic parameters p_m and p_t, as occurs in nature. With the former model it was necessary to change abruptly the constitutive law of the material, what induced numerical

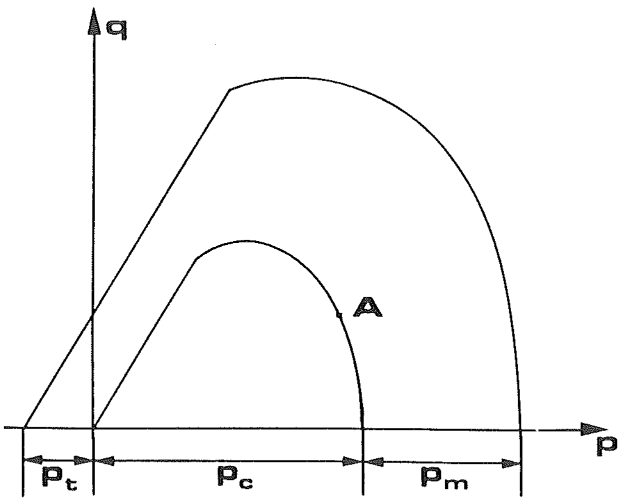


Fig. 3. Evolution of the plastic surface during grouting.

problems and uncertainties. Note that the number of parameters one has to estimate is the same in the two cases.

The modelling of the grouting process was performed by assuming that improvement of soil parameters is larger in a gravelly than in a sandy layer. On the other hand, since grout seepage is difficult through fine grained soils, it was assumed that compaction effects prevail in sand. Such an effect was modelled by imposing an uniform anelastic volumetric strain in the grouted zones. The choice of the value of such a strain is however very difficult. Indeed, one of the reasons to perform such a back-analysis was that of determining the most appropriate value of anelastic strain to simulate the effects of grout injection.

The numerical values of the constitutive parameters to be introduced in the backanalysis were evaluated on the basis of experimental results in the grouting and drift excavation phases; such measurements include surface settlement data as well as soil deformations along sliding micrometers.

For the crown excavation phase a further parameter had to be assessed, that is the percentage of the soil pressures acting on the excavation profile at the time when provisional liner begins acting. This value is usually in the range 55 to 70%; in our case, a value of 60% gave optimum agreement with the measured surface settlement at the time when the excavation front crossed the instrumented section. It should be noted that, despite the unusual section profile and the complexity of the excavation process, the value of such percentage is practically the same as that used to model the excavation of current 9m diameter underground tunnels in Milan soil. It can be concluded, therefore, that this parameter is low-sensitive to tunnel profile and size.

5. COMPARISON OF RESULTS

Experimental results were modelled by the backanalysis in a fairly satisfactory way, while the design analyses failed to reproduce soil heave. In Fig. 4, it can be seen that actual displacements, measured along sliding micrometer S11, (whose position is shown in Fig. 2) are correctly matched by backanalysis results in the grouted region; in the deep segment (from -34 to -24m) a 2mm settlement is calculated which does not really occur (in that region soil is probably stiffer than assumed in the model); in the surface segment(from -10m to surface) a sharp increase in soil displacements is not reproduced by the model.

It should be born in mind, however, that the sliding micrometer passed there close to a wedge of rigid material. Before the erection of the tall building, under the base of the raft, extensive use was made of high water content cement mixture, which was allowed to seep through the foundation soil. It is possible that the grouting pressures that cannot dissipate in that region are channelled

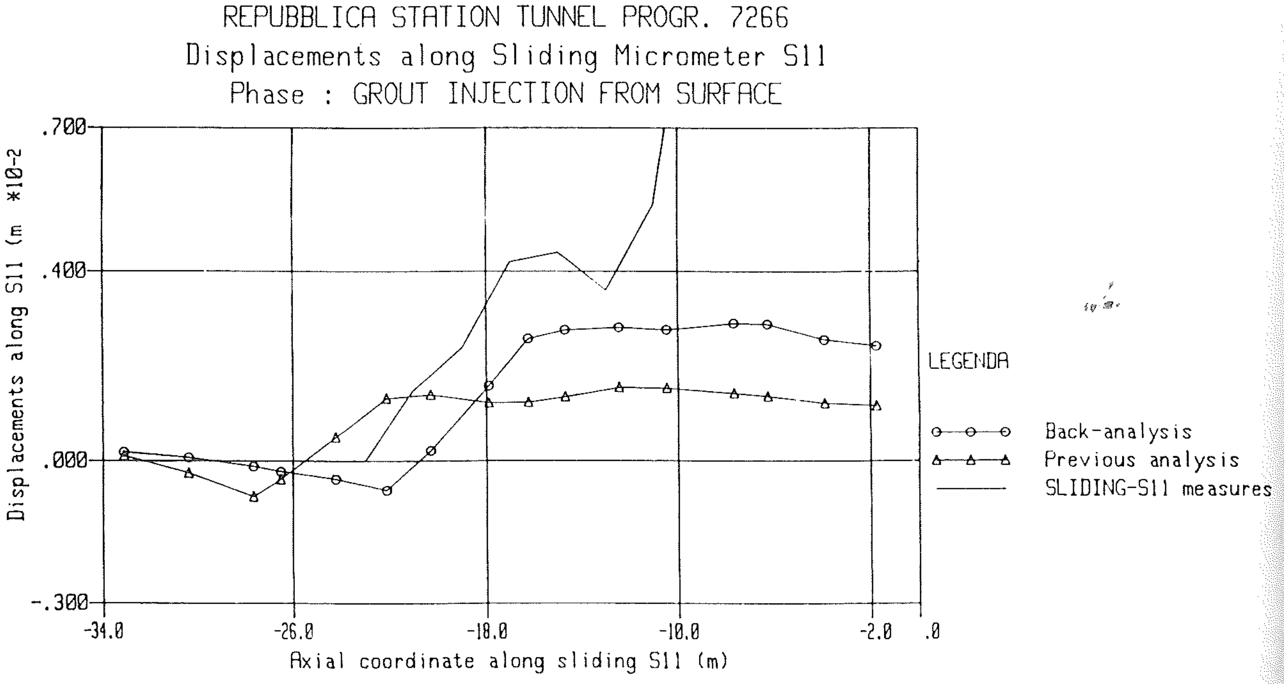


Fig. 4. Comparison between observed and computed displacements along sliding micrometer S11 during grouting from surface

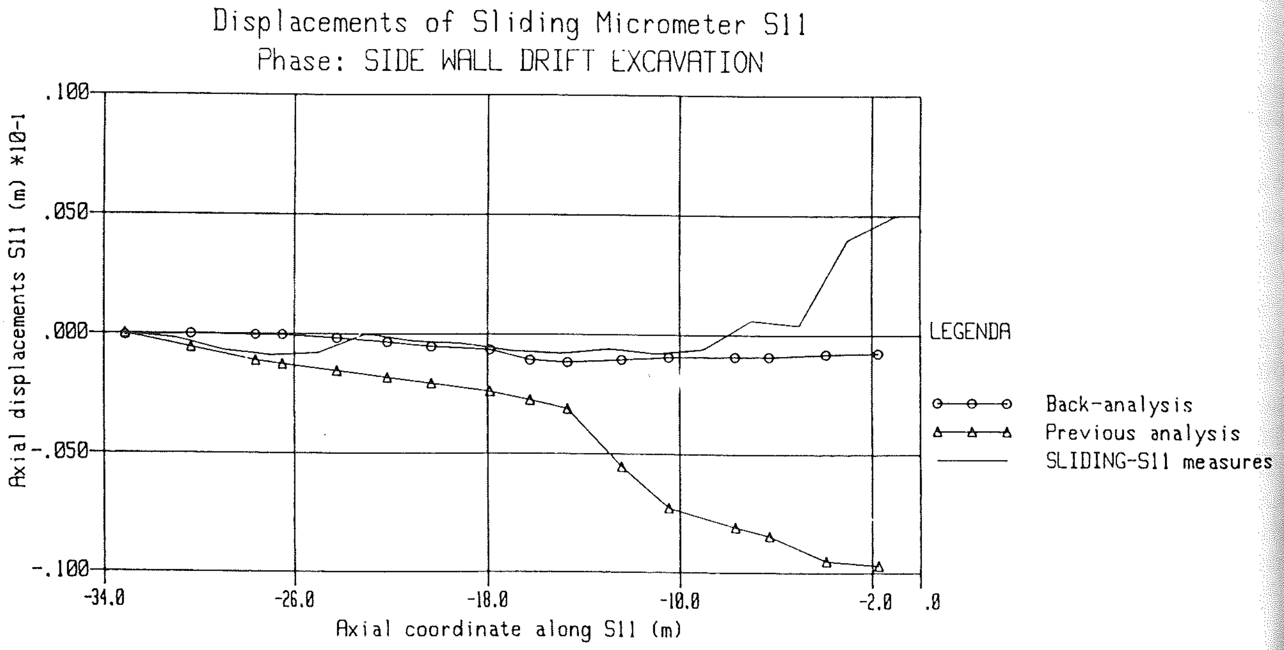


Fig. 5. Comparison between observed and computed displacements along sliding micrometer S11 during side-wall drift excavation

along the direction of the sliding micrometer to give rise to the recorded displacement. Since neither the mechanical properties nor the extent of such a zone are known, it was decided to disregard it in the numerical analysis. The calculated results are consequently much more regular than the observed ones.

The model of the excavation of the side wall drift was quite successful. Observed displacements were indeed well reproduced as shown in Fig. 5. In the design analysis, on the contrary, computed displacements were larger than those observed. The reason for the improvement is linked to the elastoplastic strain-hardening structure of the constitutive model employed to describe the behaviour of the grouted material: in the previous analyses, grouted zones remained essentially elastic, and the elastic modulus had to be underestimated in order to reproduce the behaviour during crown excavation, as described in section 3.3.

For the crown excavation phase, computed results for sliding micrometer S11 are reported in Fig. 6, while Table 1 summarizes the calculated surface settlements in the main steps of the crown excavation process, compared with measured displacements in two instrumented sections. The settlement after the transit of crown excavation front as computed by the numerical stress-analyses, has to be weighted to take into account the short length of the tunnel. As known from literature, the length of the tunnel has to be at least 5 diameters to be considered "infinite", as in a plane strain model, while the length of the excavated section was only about 2.5 diameters. The weighting factor is therefore of the order of 0.75.

This phase is well reproduced in the backanalysis, as it was in the design model, as shown by the comparison of the sliding micrometers axial displacements, Fig. 6; it should be noted, however, that in the design model this was the main step on which parameters were estimated; therefore this is the only phase well reproduced in those analyses. On the contrary, in the backanalysis parameter estimation was performed on the grouting and drift excavation phases, with no further adjustment. Consequently, the correct behaviour obtained for the crown excavation phase suggests that the model is able to correctly simulate all the construction

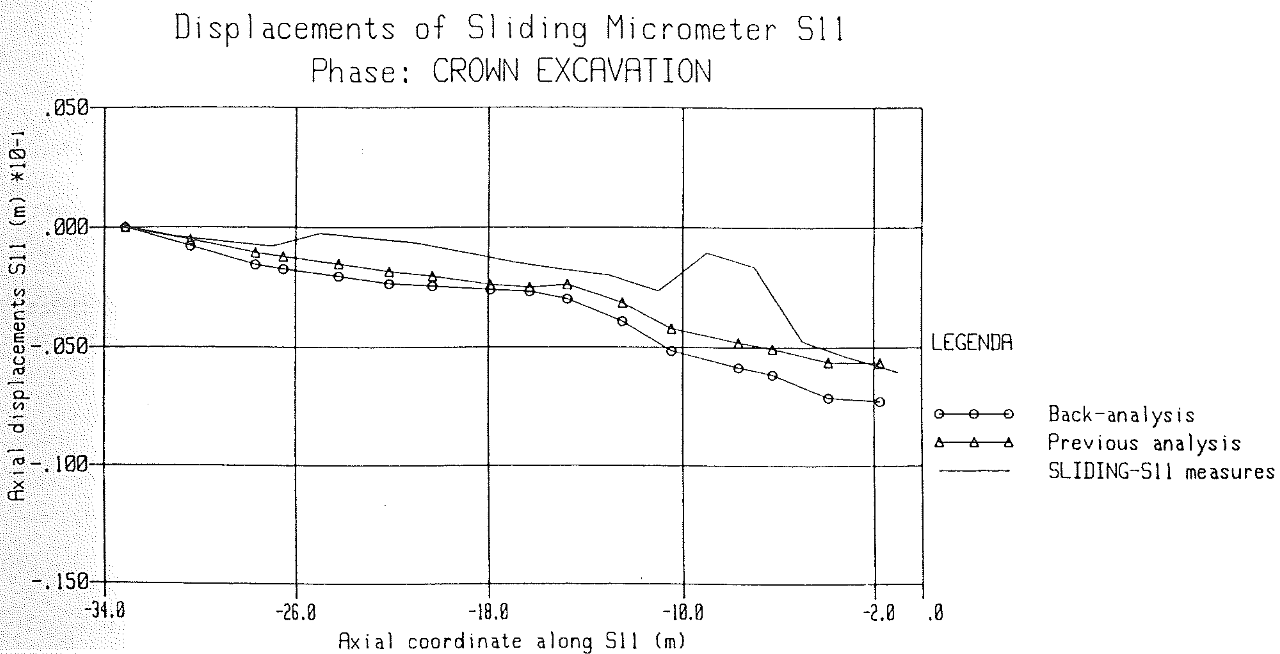


Fig. 6. Comparison between observed and computed displacements along sliding micrometer S11 during crown excavation phase

TABLE 1 - Surface settlements (mm) during crown excavation

STEP	TUNNEL AXIS DATUM POINTS 110R	95R	DESIGN ANALYSIS	BACK-ANALYSIS	RAFT CORNER DATUM POINTS 562	562A	DESIGN ANALYSIS	BACK-ANALYSIS
front transit	2.8	3.	3.4	2.15	2.6	1.8	1.6	0.21
crow arch casting	10.3	12.5	(not av.)	10.33	4.5	3.7	(not av.)	4.79
end crown excav.	11.9	14.3	10.	13.16	6.2	6.6	4.5	7.25
final settl. (*)	16.1	18.	(not available)		(not available)			

(*) NOTE: soil heave in previous steps should be deduced to obtain final absoulte settlements.

phases.

Other problems are still to be solved, however:

- the model is quite sensitive to the rate of load application, especially in application of the anelastic volumetric deformation in the grouting proce this is due to numerical problems that arise at the boundary between natu and grouted soil, which is presently modelled as a sharp discontinuity smooth variation of parameters through this boundary could yield more sta and accurate results;

- soil dilatancy is not yet modelled in a satisfactory way; this is a conseque of the associated flow rule employed in this constitutive model. An impro ment could be obtained with a non associated plasticity model.

6. CONCLUSIONS

Two different approaches are compared to the design problem of a shallow ur tunnel in grouted alluvial soil; the approaches differ mainly in the constitut model used in the description of grouted soil.

It is shown that while a traditional elastic-perfectly plastic model, w Drucker-Prager yield condition, gives only partially satisfactory results, elastic-plastic strainhardening model can lead to more accurate results; this due to the following two features that are included in the latter constitut model: a) a smooth transition from virgin to grouted soil, with a gradual cons tutive parameter increase and the preservation of the constitutive model str ture; and b) a more realistic modelling of the behaviour of the grouted s through the strain-hardening law.

In order to further improve the accuracy of the model, a more refined scheme the modelling of the grouting pressures should be developed, and a more sophist ated constitutive model with non associated flow rule should be implemented.

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