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Settlement of a tall building induced by a tunnel excavation

Déplacements d'un bâtiment induits par l'excavation d'un tunnel

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ABSTRACT: the paper presents a detailed case study, concerning the excavation of a tunnel of the new Underground Railway Link in Milan and the induced settlements of a tall building nearby. The different strategies of soil improvement, necessary to avoid excessive settlements, are first presented together with the results of a numerical analysis aimed at determining the safest procedure. Details of the instrumentation and of the observed displacements during the subsequent construction phases are next given. The results of numerical backanalyses performed with two different constitutive models of soil behaviour are finally discussed.

RESUME: L'article présents un étude detaillé de l'excavation d'un tunnel du nouveau chemin de fer souterrain de Milan ed des déplacements d'un haut bâtiment, proche a l'excavation. Les differentes strategiés d'amelioration du sol, qui sont nécessaires pour eviter des déplacements excessives, sont presentées avec les resultats des analyses numeriques pour la determination de la procedure la plus satisfaisante. Ils sont en suite donnés les detailles de l'instrumentation mise en place et dex déplacements observés pendant les phases successives de construction. Ils sont finalement presentés les résultats des analyses numeriques effctuées a posteriori pour interpreter les données experimentales.

1 INTRODUCTION

The station "Repubblica" of the new Underground Railway Link, presently under construction in Milan, was 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 material, with alternating layers of sandy gravel and sand. Fig. 1 shows a typical soil section as deduced through borehole sampling.

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 in the state of stress in the tensile reinforcement of the raft. To this purpose, accurate measurement of raft displacements and of soil deformation were carried out together with numerical simulation of the excavation process.

The paper discusses the criteria followed in designing the grouting procedures, reports the observed displacements of the building during the subsequent construction phases and presents the results of numerical back-analyses aimed at interpreting the overall behaviour of the structure.

It is shown that numerical analyses, supported by an extensive site investigation, can be used as a profitable design tool during construction; their results were in fact used to choose between different possible strategies of soil improvement. The displacements of surface points observed at the end of the construction were in good agreement with the predicted ones. Predicted displacements within soil mass were less accurate, especially those concerning grouting phase.

A further back-analysis was then performed after the completion of the work. A new model which was used to describe grouted soil behaviour is presented. It is shown that with such a model it is possible to improve the modelling of soil excavation and of the grouting process. The latter is however the most difficult step, since grouting alters in a complex way the mechanical behaviour of natural soil. It will be shown that even with this refined analysis experimental data are not yet adequately reproduced and that further fundamental research on soil grouting is needed in order to obtain fully satisfactory results.

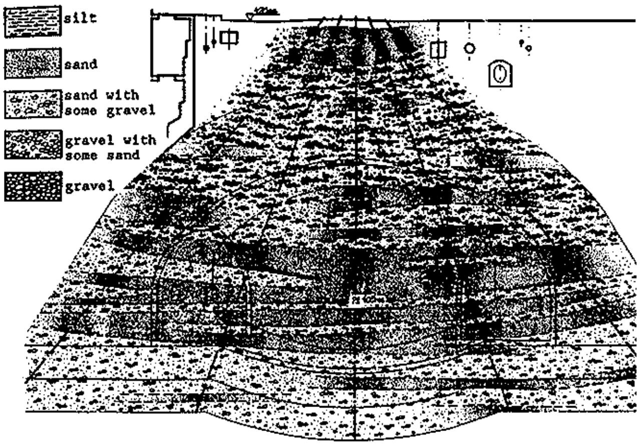

Fig. 1. Typical alluvial soil stratigraphy.

2 ANALYSIS OF THE PREEXISTING STATE OF STRESS

In order to choose the type of soil improvement necessary to limit the disturb that the excavation could produce in the nearby structures, it was first necessary to estimate the state of stress and strain existing in the soil under the foundation and in the raft prior to tunnel excavation. This was a difficult task, since the load history of the foundation soil was quite complex. Fig. 2 shows two side sections of the building and of the tunnel to be excavated. It is possible to note that on the left of the building, in section A-A, there exist a rigid diaphragm wall that was anchored to the soil under the raft by means of seven orders of ties. This was done in order to excavate the Third Line of the Underground (which runs orthogonally to the axis of tunnel under construction), without inducing excessive settlements of the tall building. The natural state of stress of the soil was then altered by the construction of the building, the construction and anchoring of the diaphragm wall and the grouting process. It was likely to be highly non-uniform and difficult to measure.

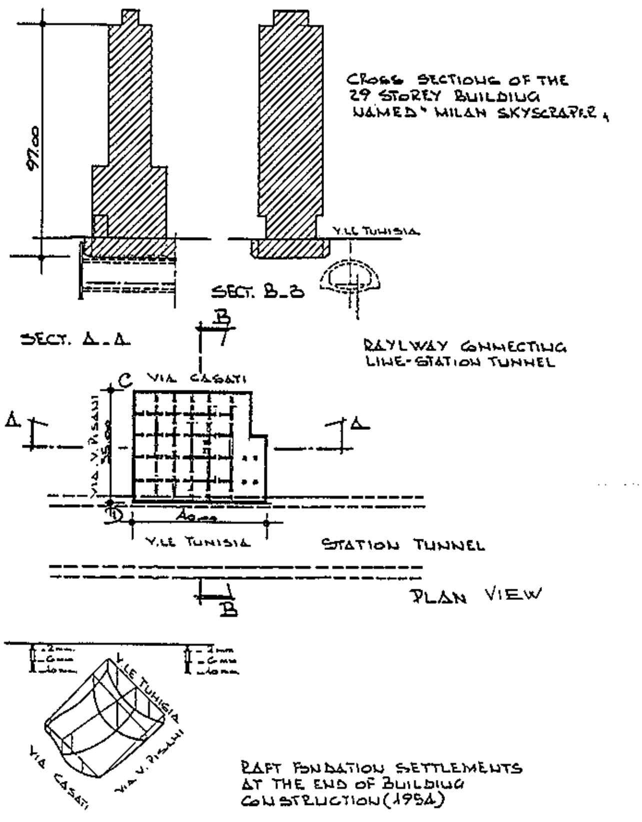

Fig. 2. Cross sections of the building and underground works.

The deformation of the raft was instead well known, since a number of datum points were positioned in the raft at the moment of the building erection, about 35 years ago. Before the construction of the diaphragm wall, the settlement of the center of the raft was measured to be about 15mm, whilst the corresponding settlements of the raft edges were between 6 and 9mm, see Fig. 2. The pretension of the ties of the diaphragm wall caused the edge of the raft to heave; as can be seen in Fig. 3, the largest heave occurred in corner C (about 10mm), and then decreased both along the edge of the foundation and towards its center. The rotation was then far from being rigid and likely to produce a high stress state in the raft.

It was then decided to perform a series of numerical analyses to investigate the state of stress in the soil and in the foundation. Natural soil behaviour was modelled by means of an elastic-plastic strain-hardening model, called 'Lamber' (Botti et al. (1988)), while grouted soil was assumed to be an elastic-perfectly plastic material; the constitutive parameters of such constitutive laws were determined on the basis of various backanalyses performed with the same models.

In order to do that, two tunnel sections, at a distance of a few dozens meters from the tall building, were studied. The surface settlements and soil deformation during excavation were recorded by various sliding micrometers. Several numerical analyses were necessary to reach the desired agreement between observed and calculated displacements. Fig. 4 shows a comparison of the displacements measured along an inclined sliding micrometer with those calculated. The numerical analyses were mainly focused on crown excavation, which was the phase producing the most important deformation and stress redistribution within soil.

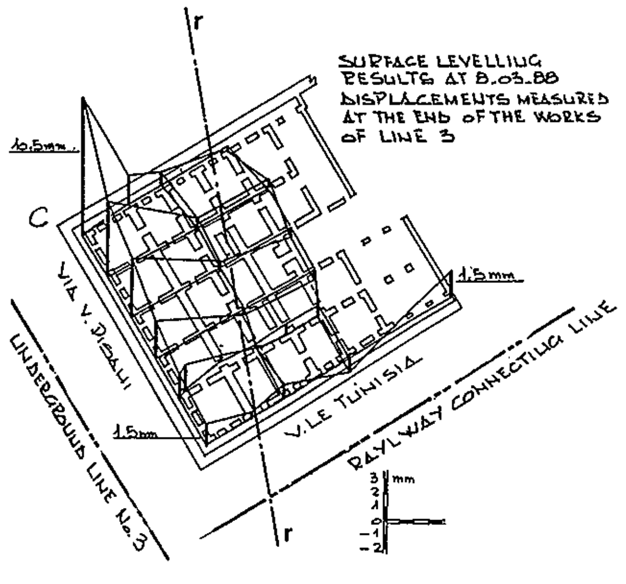

Fig. 3. Raft displacements after tie tensioning.

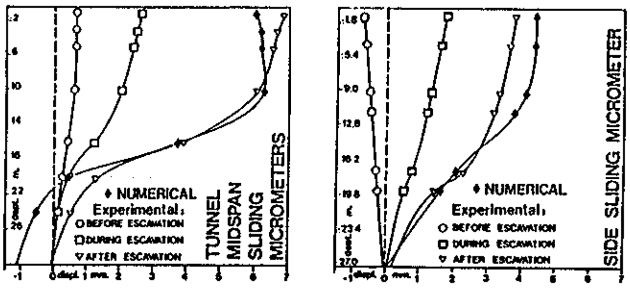

Fig. 4. Station tunnel back-analysis: comparison between monitored sliding micrometer displacements and numerical analysis results.

To evaluate the effects of the prestressing of the ties on the soil behind the diaphragm wall, another numerical analysis was run. The wall, the soil below the raft and the open cut, ties and building foundation were discretized by means of finite elements and all the excavation and tie tensioning phases were simulated; displacements of the diaphragm wall and of the raft were compared with in situ-measurements; as shown in Fig. 5, quite reasonable agreement was found; this was interpreted as a further confirmation of the validity of the assumed constitutive parameters for natural soil.

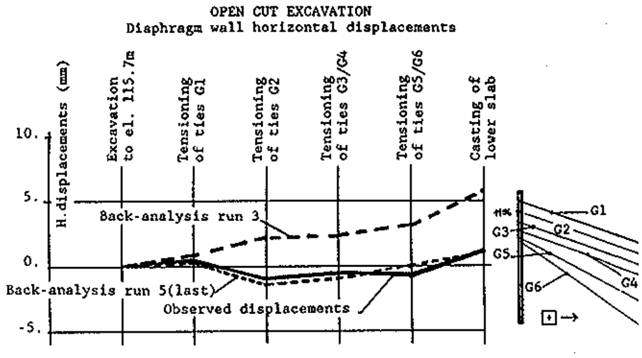

Fig. 5. Comparison of the displacements of the diaphragm wall for open-cut back-analysis

It was found that the effect of grouting and anchoring was negligible outside the prestressed zone. The state of stress in the ground was then calculated by simulating the construction of the tall building.

The state of stress in the raft was estimated by means of a 3D finite element elastic stress analysis. The deformed shape of the raft after diaphragm wall construction was accurately reproduced, by applying pressure wedges below the foundation, which represented the effects of soil heave due to tie tensioning. It was found that somewhere the reinforcement was at the serviceability limit, so that further straining of the raft due to differential settlements caused by tunnel excavation should be limited within a strict range. On the basis of such a result, it was decided to enlarge the grouted zone towards the center of the foundation, to give the raft a support as uniform as possible.

3 THE DESIGN OF SOIL IMPROVEMENT

Three different solutions were analyzed for soil improvement. In order to limit the heave effects on the neighbouring building, in the grouting phase for crown arch and side walls, two symmetrical jet-grouting cut-offs 1.2m wide were initially designed, Fig. 6. Traditional grouting from surface and subsequently from side wall drifts was also foreseen. This solution was however discarded, since a numerical analysis showed that settlements as large as 17mm were likely to occur under the tall building.

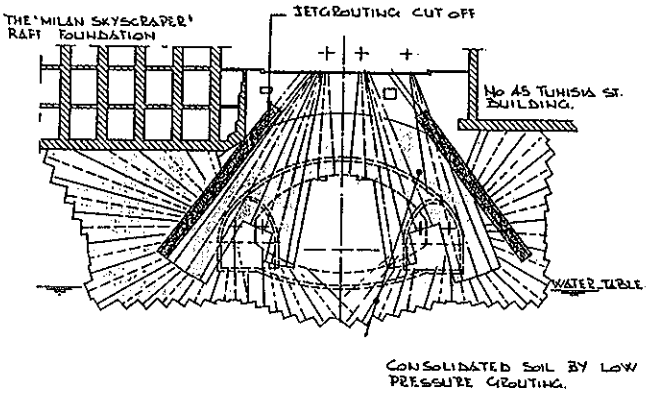

Fig. 6. First design scheme with jet-grouting cut-offs.

In a second solution the width of the cut-off under the tall building was increased to 1.8m and soil was reinforced from surface by grouting even below side drifts. Even in that case, however, settlements were too large, of the order of 12mm.

Such a result was essentially due to the stiffness of jet-grouting columns, which were able to carry a considerable amount of load; pressures at the base of the columns were consequently high enough to cause yielding in the soil underneath, even if grouted, with consequent remarkable settlements.

It was then finally decided to avoid the use of jet-grouting. Two cement mixtures were instead injected at low pressure. The former, known as Solena, was used in the arch around the tunnel and to protect the side drifts. The latter, known as Mistra'-L, was used under the raft. To permeate even the finer voids a low viscosity chemical mixture known as Silacsol-S was used everywhere.

Injection 'manchette' pipes were arranged in two radial patterns, having centers the one at soil surface, on the tunnel mid plane, the other inside the drift below the tall building. The amount of grout was depending on the type of soil to inject, the desired degree of improvement and the admissible heave. It varied from 10 to 14% of the theoretical soil volume for cement mixtures, and reached a value as high as 18% for the chemical mixture.

The subsequent numerical analyses showed that the settlements that were likely to occur were smaller than in the previous cases; even the deformation of the raft was more favourable, as a stress concentration was avoided near the corner of the raft. Therefore soil improvement was carried out according to the last scheme.

4 OBSERVED DISPLACEMENTS

To keep under constant control the movements of soil and of the surrounding buildings during tunneling, it was decided to extensively instrument the foundation of the tall building and two sections of the tunnel. As shown in Fig. 7, the raft was instrumented with fixed bidirectional inclinometers, liquid level gauges, long base extensometers. The displacements of several datum points were surveyed together with that of a pendulum positioned in the building staircase.

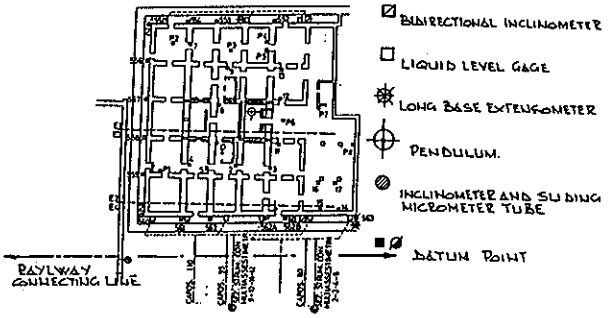

Fig. 7. Layout of site instrumentation.

Sliding micrometers coupled with inclinometers were put into the soil to measure displacements in three orthogonal directions. Finally the liner was instrumented with extensometers and pressure cells to measure the normal stress on the liner, the tension of the reinforcement and the state of stress in the ribs near the advancing front.

4.1 First phase: grouting from surface

The grout was initially injected into the outer pipes, and then into the inner ones. The soil heave under the building was limited to 1.5mm. Only outside the raft a datum point and extensometer E1 recorded a heave of 6mm. Correspondingly, the sliding micrometers positioned under the raft showed limited elongations during this phase, while pipes more distant from the tall building recorded extensive tensile strains. This is possibly due to the fact that this phase was performed initially under the tall building, and only some months later on the opposite side. Grouting in the right side of the section caused appreciable displacements only in that region. This was to to the fact that in the left side soil was already compacted by the high state of stress caused by the building weight, whilst on the right soil was still in the virgin state.

4.2 Excavation of the side drifts

The excavation of the side drift on the side of the tall building started when grouting on the opposite side was not yet completed. The walls were shotcreted and ribs positioned every 0.8m. The side

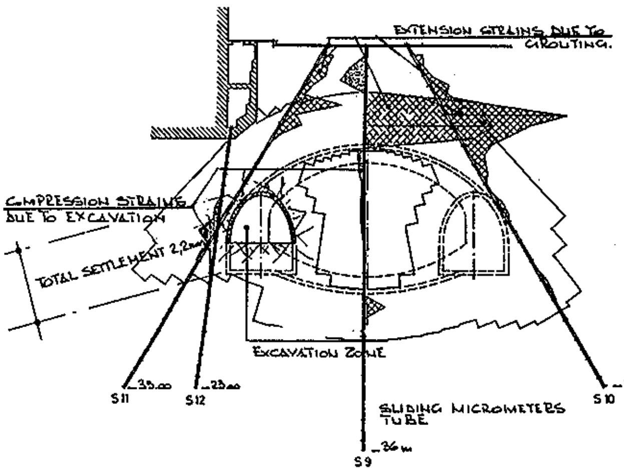

Fig. 8. Strains recorded by she sliding micrometers during side-wall drift excavation.

wall of the final reinforced concrete liner was cast only after the completion of grouting under the raft and the lowering of the drift. The largest compressive strain (see Fig. 8) recorded by the sliding micrometers was of the order of 0.075%. While the effects of the excavation of the first drift (below the tall buiding) were recorded also at the opposite side of the tunnel section, the excavation of the second drift caused appreciable strains only in the neighbouring soil.

The inclinometers denoted horizontal displacements in a direction opposite to that of the excavation. This was possibly due to the beginning of the arching effect of the grouted crown.

4.3 Second grouting phase

The most delicate step was the grouting from the drift under the tall building, just under the raft. Excessive grout pressures could cause heave of the raft and untolerable stresses in the reinforcement.

The injections were performed in various steps, controlling the amount of induced displacements after each step. In order to keep the heave below the prescribed limits, it was necessary to employ less cement grout than foreseen in the design. The chemical grout was instead adsorbed according to design specifications.

The overall vertical displacements recorded form extensometer E2 and datum point 562B are shown in Fig. 9. It can be seen that the largest heave occurred during this phase.

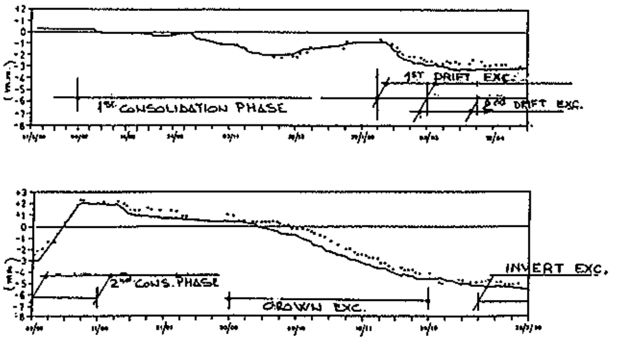

Fig. 9. Displacement records for extensometer E2 (solid) and datum point 562B (dotted).

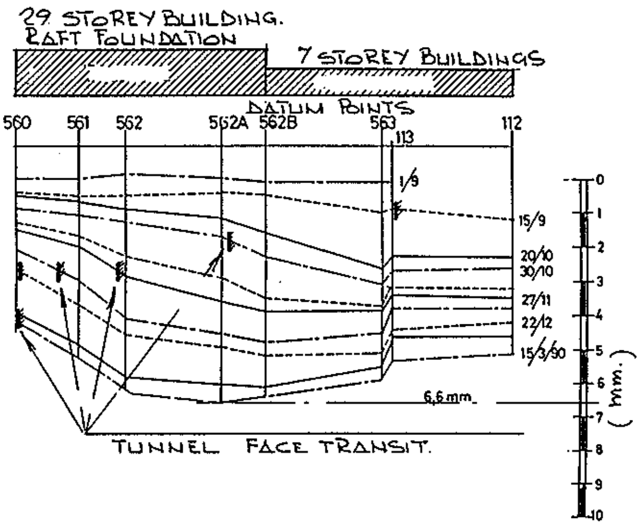

Fig. 10. Building settlements at the edge of the raft, during crown excavation.

4.4 Drift lowering, crown excavation and subsequent casting

The final phase started after the completion of phase 3 and of the improvement of soil below the invert arch. Settlements measured by the datum points at the edge of the buildings are shown in Fig. 10. The largest settlement of 6.6mm was recorded in the zone of the tall building.

Along the center line of the street, settlements were higher: about 12mm at a section near the tall building, and more than 20mm far from the building, where soil improvement was less enhanced, and the distance between the excavation front and the crown lining was larger (23m vs. 15m). The measured stresses in the reinforcement of the final lining were far from the allowable stresses. A small rotation (about 10^{-4}rad) of the axis of the building was recorded by the pendulum.

4.5 Comparison between observed and predicted displacements

Observed displacements were generally higher than those predicted by means of the numerical analyses. Near the tall building, however, the calculated displacements of surface datum points were quite close to the measured ones. In fact, the predicted displacement at the edge of the building during crown arch excavation was of 4.5mm (as opposed to the measured value of 6.6mm). Correspondingly, the calculated settlement of the street axis was approximately 10mm (as opposed to the measured value of 11.9mm).

The displacement field within the soil mass, as recorded by the sliding micrometers and the inclinometers, was instead poorly modelled, especially in the grouting phase. It was then decided to perform a further back-analysis with a more refined constitutive model to describe the behaviour of grouted material.

5 BACKANALYSIS OF TUNNEL EXCAVATION

The displacements measured by the sliding micrometer S11 were analyzed during two construction phases: grouting from free surface and excavation of a side-wall drift, a few meters below the edge of the tall building foundation raft. In the design analysis the grouted soil was modelled by an elastic - perfectly plastic constitutive law with a Drucker-Prager yield condition. The results of such an analysis, which was also used as a verification tool during the construction phase, were thoroughly discussed elsewhere (Balossi Restelli et al. (1989)).

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. 11 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. 11.

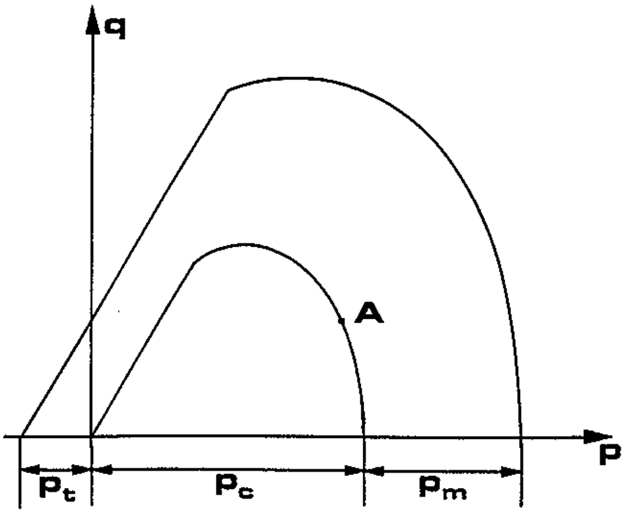

Fig. 11. Evolution of the plastic surface during grouting.

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 (Di Prisco et al. (1991)). 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 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.

Experimental results were modelled in a fairly satisfactory way, partially at least, while the design analyses failed to reproduce soil heave. In Fig. 12, it can be seen that actual displacements, measured along sliding micrometer S11, are correctly matched by back-analysis 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.

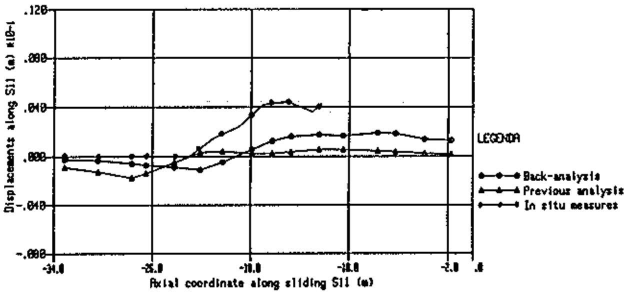

Fig. 12. Comparison between observed and computed displacements along sliding micrometer S11 during grouting from surface.

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 along the direction of the sliding micrometer so 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. 13. 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

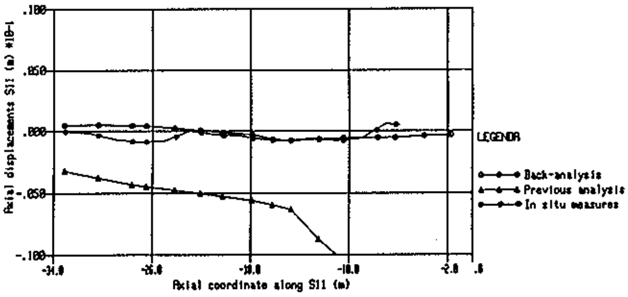

Fig. 13. Comparison between observed and computed displacements along sliding micrometer S11 during side-wall drift excavation.

material: in the previous analyses, grouted zones remained essentially elastic, and strains were governed by a single elastic modulus, whose value was chosen to well reproduce settlements during the crown excavation phase; the new constitutive model for grouted soil, on the contrary, accounts for a stiffer unloading/reloading behaviour (this is the case for pilot drift excavation), and a softer virgin compression (which plays the leading role during crown excavation).

Other problems are still to be solved, however:

- the model is quite sensitive to the rate of load application, especially in the application of the anelastic volumetric deformation in the grouting process; this is due to numerical problems that arise at the boundary between natural and grouted soil, which is presently modelled as a sharp discontinuity. A smooth variation of parameters through this boundary could yield more stable and accurate results;
- soil dilatancy is not yet modelled in a satisfactory way; this is a consequence of the associated flow rule employed in this constitutive model. An improvement could be obtained with a non associated plasticity model.

6 CONCLUSIONS

The paper presents a detailed case study, concerning the excavation of a large shallow tunnel in a granular soil improved by grouting and the induced settlements of a tall building nearby. The different strategies of soil improvement, necessary to avoid excessive settlements, are first presented. It is shown how combining the experimental data retrieved by an extensive campaign of in situ measurements and the results of numerical analyses it is possible to optimize the design of soil improvement and to predict the settlement trend in good agreement with actual results. The instrumentation employed and the observed results are presented and discussed. The hypotheses on which the numerical analyses are founded and the comparisons between observed and calculated results are described. It is shown that using an elasto-plastic strain-hardening constitutive law to model natural and improved soil behaviour it is possible to reproduce reasonably well not only surface settlements but also the field of displacements within the soil mass. The modelling of the grouting phase presents however still unsolved difficulties in the a priori determination of the amount of the anelastic strain, in the modelling of the interface between grouted and natural soil and in the determination of the constitutive parameters of the improved soil.

7 ACKNOWLEDGEMENTS

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