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Numerical modelling of the effects of compensation grouting

C. Kummerer & H.F. Schweiger
Institute for Soil Mechanics and Foundation Engineering, Computational Geotechnics Group, Graz University of Technology, Graz, Austria

R. Otterbein
Keller Grundbau GmbH, Bochum, Germany

A. Watt
Eurostation, Brussels, Belgium

ABSTRACT: Compensation grouting is increasingly used as an active settlement control measure in shallow tunnelling in urban areas when potential damage to existing buildings is expected. Despite of that fact, the design of compensation grouting work is still mainly based on empirical considerations and experiences from past projects. In this paper, the finite element method is utilized to describe the basic effects of compensation grouting. In the proposed finite element model, the soil displacement caused by opening fractures is simulated by volumetric expansion of elements representing the grouting area. It also accounts for the stiffness increase of both the grouting zone and the surrounding soil. The model presented is validated with measurements from compensation grouting operations at the Central Station in Antwerp (Belgium).

1 INTRODUCTION

Among the variety of protective measures in near surface tunnelling underneath sensitive structures in built-up areas, compensation grouting (Mair 1994) to control excavation induced (total and differential) settlements has gained increasing importance. In contrast to alternative measures, the advantage of compensation grouting is that it is an active settlement reduction method. That means that settlements are measured during the excavation and corrected immediately at the location where they occur. Therefore grouting pipes, called Tubes á Manchette (TAMs), are installed between the tunnel and the buildings to be protected. Grouting is done in two phases:

- conditioning phase
- compensation phase

During the conditioning phase the soil is displaced and thereby compacted and stiffened. Voids are (partially) filled and the effects of the installation of the TAMs are counteracted. In normally consolidated soils, this goes along with an increase of the horizontal stress. Once the lateral stress approximately equals the vertical one, horizontal fractures will occur which will result in vertical displacements (Raabe & Esters 1993). At this stage an immediate response of the injections during the actual compensation phase is guaranteed. This pre-treatment phase is finished when heave is observed at the surface.

In general a considerable settlement reduction is achieved due to the increased stiffness of the grouted soil. According to Chambosse & Otterbein (2001a) the settlement reduction achieved by pre-treating reaches from 25 to 50% depending on the soil conditions. Consequently, the required heave is smaller during the heaving phase. Grout quantities can hardly be estimated for this phase because of the uncertainties involved (e.g. inhomogeneities and local stress states). Evaluating data from several projects, Chambosse & Otterbein (2001a) give a range from 42 to 115 l per m² for stiff to medium dense soil.

In the actual compensation (or heaving) phase, grouting is done depending on the magnitude and the distribution of the observed settlements. Immediate evaluation of settlement data based on real-time monitoring equipment is therefore an absolute necessity. Today, high accuracy automatic waterlevel systems are available for that purpose.

The efficiency of grouting at that stage, defined as the ratio of (average) heave to injected volume is typically between 5 and 20% according to Chambosse & Otterbein (2001a). These efficiency values vary considerably with stress level, i.e. the efficiency for relevelling high loaded foundations is lower than for less loaded ones. Moreover, the grouting efficiency is not constant during the grouting process (see Watt 2002).

2 NUMERICAL MODELLING OF COMPENSATION GROUTING

Modelling compensation grouting by means of numerical methods is a difficult task as grouting is done with several injections of very small quantities in many passes. A large number of fractures with different orientations develop, each fracture interacting with others. Therefore, only a "global" approach seems to be applicable to capture the overall effects of grouting. The grouted zone, enclosing the area around the injection points, has to be modelled by some elements within the analysed domain. The volume change in this zone is achieved either by internal pressure or volumetric strains.

Soga et al. (1999) gave an example for the application of artificial internal pressures. The use of a zero thickness interface representing a horizontal grouting "sheet" was examined by Kovacevic et al. (1996) for a field trial in London clay. An extension of this approach was introduced by Wisser et al. (2001) for three-dimensional situations. Due to the heavily overconsolidated character of London clay, only horizontal fractures were taken into account. Wawrzyniak (2001) examined a similar approach for normally consolidated subsoil conditions. Therefore, both horizontal and vertical "fractures" were modelled. Because the pressure actually acting in the soil is not known, the pressure to be applied in the model has to be estimated from experience. On the contrary, grout quantities can be exactly controlled.

The second group introduces volume expansion as the key parameter to simulate the displacement of the soil. Nicolini & Nova (1999) proposed a model for grouting in granular soils based on the finite difference method. The grouting process is simulated by means of inelastic strains. Depending on the subsoil conditions, both uniform expansion of the bulb and the formations of grout lenses can be modelled. Schweiger and Falk (1998) reported on the use of "thermal" expansion for numerical studies on compensation grouting for the Lisbon underground.

The approach introduced in this paper utilizes volumetric strains as the input parameter. These strains are applied to the grouting zone, which consists of two parts. In the inner area, representing the zone close to the valves, the expansion is simulated. In the outer area modified soil properties are taken into account representing the soil improvement achieved by grouting. The representation of the grouted zone in the finite element model is depicted in Fig. 4. The choice of a suitable improved zone is problem specific. Falk (1998) gave some suggestions for the size of the treated zone. The proposed model was validated by Kummerer et al. (2002) by means of an analytical solution. The application of the model for a practical problem is described below.

3 PROJECT DESCRIPTION

Built as a dead-end station at the end of the nineteenth century, the monumental Central Station in Antwerp with a height of 70 m (Fig. 1) is presently being reconstructed in order to meet the requirements of the high-speed train link Brussels-Amsterdam that is part of one of the European priority transport projects. In 2006, the high-speed trains will pass through the station on the second underground level. The tunnel section under the main station building is 80 m long and widens from two tracks to four tracks at its southern end, with a maximum width of 21 m.

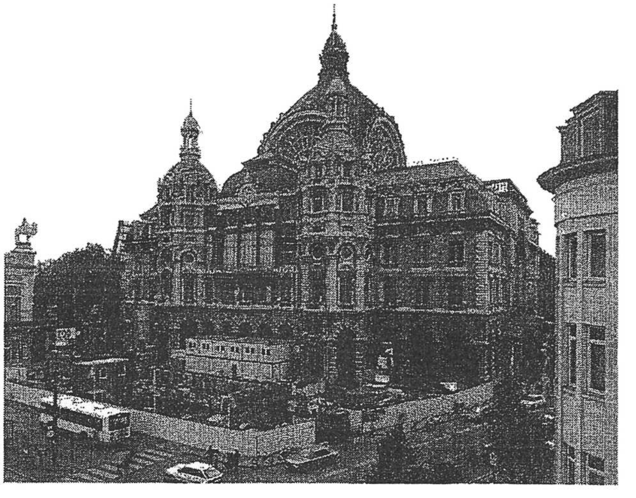

Figure. 1: Central station in Antwerp.

The subsoil consists of the so-called Antwerp sand, a tertiary slightly overconsolidated dark grey fine sand with about 10% clay content. This layer has a typical resistance of 18 to 24 MPa in the cone penetration test. The sand is intercepted by a strong shell layer of 0.5 to 0.7 m height and underlain by Boomse clay at about 30 m depth. The groundwater level is typically found 7 m below the ground surface.

The well-proved Belgian Tunnelling Method incorporating a pipe roof was considered to be most suitable for this project. These pipe roofs have been used in Belgium since the 1960s. Because of experiences with the construction of the old Metro tunnel built below the edge of the Central Station, where large settlements occurred, and settlements predictions for the actual project, which stated maximum settlements from 60 to 120 mm, compensation grouting was specified by Eurostation (designers of the tunnel) as an active settlement control measure beside the passive effects of the pipe roof. The grouting work applying the Soilfrac-technique was performed by Keller Grundbau.

Most attention was paid to the heavy dome supported by four main piers, which are founded on large footings. A cross-section through these piers is shown in Fig. 2.

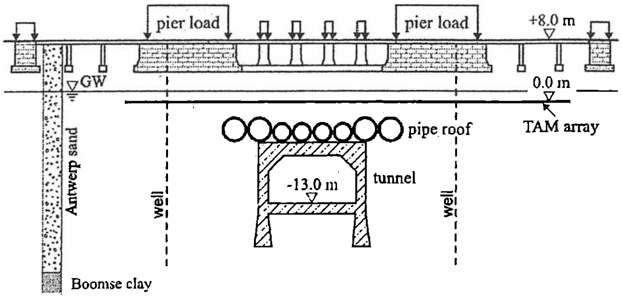

Figure 2. Cross-section through the main foundations.

The first construction phase was the lowering of the ground water by about 15 m. The excavation of two shafts on both sides of the building followed this dewatering. After that, the drilling and installation of the TAMs from the shafts was made (Chambosse & Otterbein 2001b). These grouting pipes with a diameter of 50 mm and a valve spacing of 0.5 to 1.0 m were put in place about 3.5 m below the bottom of the foundations. To control the position of these TAMs (total length 3500 m), 35% of the borings were measured with inclinometers. The drilling deviation in the sand layer was about 1%. The arrangement of the TAMs is depicted in Fig. 3.

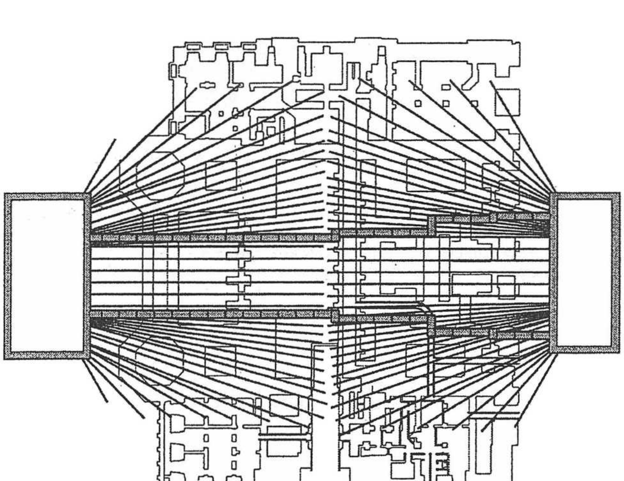

Figure 3. TAM array underneath the foundations.

Once the TAMs were installed, the conditioning phase was started. This phase was considered to be completed when the columns within the zone of influence of the works had been heaved by 2 to 5 mm. Underneath the grouted "slab" the jacking of two pipes ∅2.96 m at each side and four inner pipes ∅2.47 m was carried out. The outer pipes provided the access for the hand excavation of the trenches, which were stabilized with prefabricated concrete panels. After completion of the tunnel walls and the pipe roof by reinforcing and filling with concrete, the tunnel was excavated in sections with a maximum length of 6 m. Compensation grouting was made after each pipe jacking operation and during the construction of the tunnel walls and excavation of the tunnel.

The basic requirement of all the grouting operations is real-time monitoring of the settlement pattern related to the excavation. The GeTec-System (Otterbein 2000) with automatic water levels applied in Antwerp has an accuracy of 0.3 mm, with settlements reading available every 30 seconds. 93 water levels were installed on columns at different level within the zone affected by of the works. Beside these relative settlement measurements precise levelling was used as a control. In addition, 67 crack devices, 5 vertical extensometers, 3 horizontal and 4 vertical inclinometers were installed.

4 FINITE ELEMENT CALCULATIONS

In this section results from finite element calculations for the compensation grouting work at Antwerp Central Station are presented. In the first part, the fundamental principles of compensation grouting are demonstrated on basis of the jacking of a single pipe underneath the main footings. In the second part, a finite element model for the main construction phases for tunnelling below the Central Station in Antwerp is presented.

4.1 Basic effects of compensation grouting

In order to investigate the effects of compensation grouting with respect to the settlement behaviour of the main footings and the change of the stresses in structural elements, the jacking of one single pipe 3.0 m in diameter underneath the footings was examined. A number of simplifications regarding both the geometry of the foundations and the pipe and the soil conditions were introduced for this study. The main purpose of this calculation was not to model the real situation in great detail, but to demonstrate some principles related to the actual project.

The finite element code PLAXIS 3D Tunnel (Brinkgreve & Vermeer 2001) was utilized for all calculations. Plane strain conditions were assumed which is represented by a slice of unit thickness in the model. The finite element mesh with approximately 650 elements is depicted in Fig. 4.

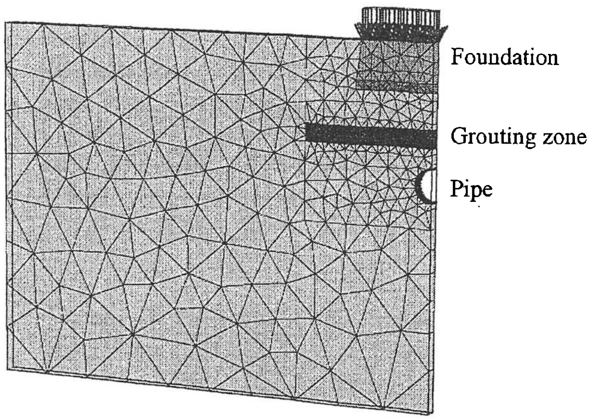

Figure 4. Finite element mesh for single pipe jacking model.

The so-called "Hardening Soil" model (see Brinkgreve & Vermeer 2001), an advanced elastic-plastic model, was utilized to describe the behaviour of Antwerp sand. This constitutive law accounts for stress-dependent stiffness, a hyperbolic stress-strain relationship and a distinction between loading and unloading/reloading. Besides shear hardening, volumetric hardening is incorporated in the model. The stiffness and strength parameters introduced in the calculation for the sand are given in Tabs. 1 and 2, respectively. E_{50}^{ref} and E_{oed}^{ref} represent the stiffness parameters for primary loading, E_{ur}^{ref} for unloading/reloading at a given reference stress p^{ref}. The power m defines the stress-level dependence of stiffness.

Table 1. Antwerp sand - stiffness parameters.

E_{50}^{ref}	E_{oed}^{ref}	E_{ur}^{ref}	ν_{ur}	p^{ref}	m
kPa	kPa	kPa	-	kPa	-
50 000	50 000	150 000	0.2	100	0.5

Table 2. Antwerp sand - strength parameters.

c^{ref}	φ	ψ	R_f
kPa	°	°	-
1.0	37	7	0.9

After generating the primary stress state and applying the foundation loads three different cases were considered:

In the reference case, settlements of the main footings resulting from pipe jacking were calculated without taking into account the effects of compensation grouting. The ground loss associated with pipe jacking was modelled in this analysis utilizing the β-method, also referred to as convergence-confinement method. A β-value of 0.45 was assumed, representing a ground loss of about 1%. In the second calculation it is assumed that the jacking is made underneath a grouted zone with increased stiffness compared to the untreated soil. According to values given by Falk (1998) a stiffness increase by a factor of 2 is considered, which can be regarded as a rather conservative value. Besides the modification of the stiffness of the grouted zone, the densification of the surrounding area caused by grouting yields a stiffer response of the soil, too. This aspect is taken into account in the third analysis. In addition to the stiffness increase in the grouted zone as mentioned above, applying small volume expansions of the treated zone simulates contact grouting. This contact heave is stopped when very small heave at the surface was observed. After that stage the pipe jacking was simulated.

The calculated settlement troughs are shown in Fig 5. Introducing a plausible stiffness increase in the grouting zone results in a settlement reduction of about 17% compared to the reference case. Taking into account the stiffness increase due to contact grouting beside the effect of a higher stiffness of the grouted zone, a settlement reduction of about 38% is obtained. This reduction factor compares very well with values given in literature where settlement reductions between 25 and 50% were reported (e.g. Chambosse and Otterbein 2001a).

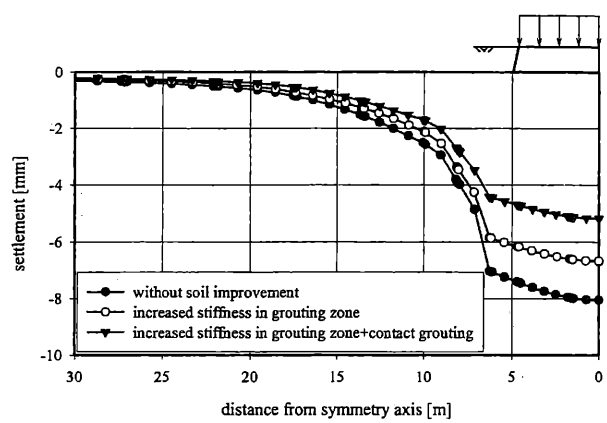

Figure 5. Surface settlements due to pipe jacking with/without contact grouting.

The influence of grouting with respect to the bending moments in the pipe is shown in Fig. 6. Therefore, the bending moment after pipe jacking (load case 1) and after the subsequent complete compensation of settlements (load case 2) was investigated. The increase in the bending moment originating from contact grouting is not significant (although done before pipe jacking it has some minor influence on the bending moments calculated because the stiffness of the surrounding soil is changed). Furthermore, it can be demonstrated that the total increase of bending moment resulting from grouting is comparatively small for this particular problem (less than 15%).

Adopting the aforementioned strategy the grouting process is simulated in a very realistic way. Nevertheless, the computational effort is acceptable for solving practical problems.

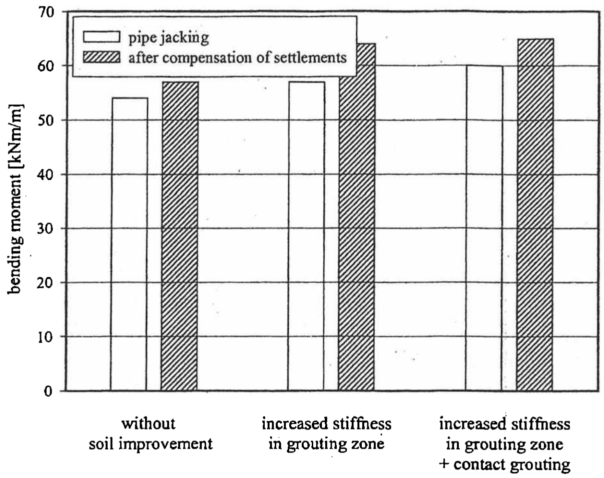

Figure 6. Maximum bending moment in the pipe.

4.2 *Finite element model of the Antwerp project*

The finite element model for Antwerp Central station is shown in Fig. 7. In this simplified model only one half of the cross section was analysed assuming plane strain conditions. The effects of the construction of the Metro tunnel were neglected. The finite element mesh consists of about 2000 elements.

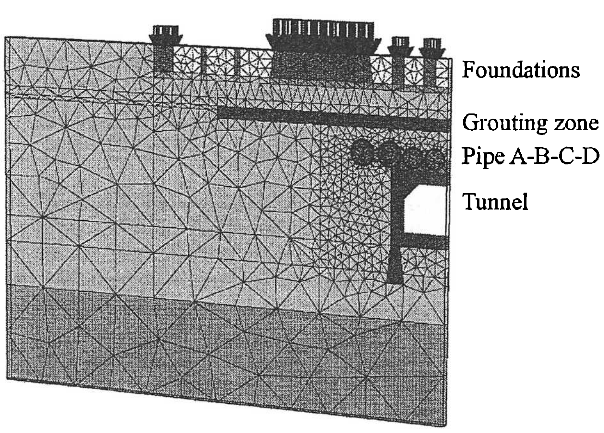

Figure 7: Finite element mesh for the Antwerp project.

Again, the Hardening Soil model was utilized to describe the behaviour of Antwerp sand. In areas influenced by the dewatering, a cohesion of 15 kPa due to capillary suction was assumed.

The following calculation steps were performed:

- Initial state of stress (the coefficient of lateral earth pressure K_0 was chosen according to $K_0 = 1 - \sin \varphi$)
- Backfill of foundations and application of loads
- Groundwater lowering (displacement reset to zero)
- Contact grouting for soil conditioning
- Pipe jacking B-D-A-C with grouting after each jacking operation
- Excavation of the side walls and the tunnel with subsequent grouting

For both the jacking of the pipes (sequence B-D-A-C) and the excavation of the tunnel including the side walls, the aforementioned β-method was applied to simulate the ground movements.

Figs. 8 and 9 show the comparison between calculated and observed vertical displacements (water level measurements) for two points of the main foundation. After the model was calibrated for the contact grouting stage, an excellent agreement between both methods can be observed.

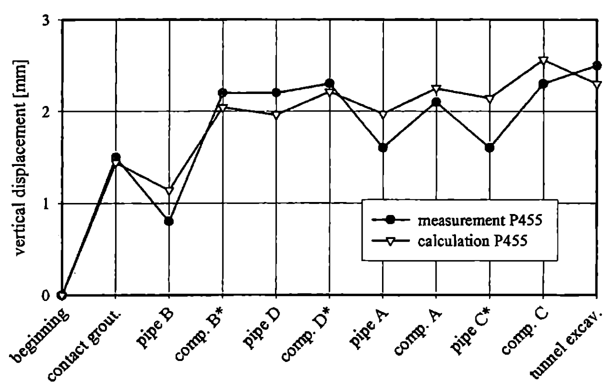

Figure 8. Comparison between measured and calculated settlements for water level P455.

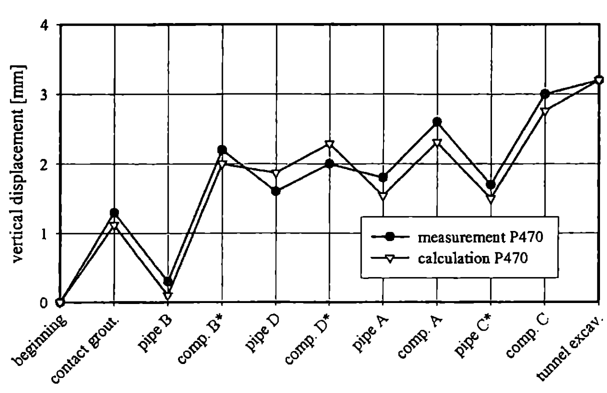

Figure 9. Comparison between measured and calculated settlements for water level P470.

In order to quantify the effects of compensation grouting, the procedure described above is performed without considering grouting. A maximum settlement of about 60 mm was obtained which is within the range of the settlement predictions. It has to be emphasized though that this value is at the lower end of the range because the numerical model does not account for construction specific uncertainties which are included when settlements are estimated from past experience.

Differences were observed between the grout volumes injected in practice compared to the ones in the finite element model. One reason is that in the contact grouting phase the grout take strongly depends e.g. on loose zones in the soil, inhomogeneities and the local stress concentrations. In most cases these effects are not known at the design stage, therefore it is not possible to introduce them in the calculation. Therefore, the model has to be calibrated for this stage. For subsequent phases it turns out that the grout intake in practice is still slightly higher than in the calculation. However, in agreement with observations in practice, the required grout volume is larger for high loaded than for less loaded foundations. Moreover, the grout efficiency is not constant during the grouting operations, but increases with the grouting progress (Watt 2002). This phenomenon is captured in the finite element model, too. An efficiency factor, describing the ratio of the grout volumes in reality and in the model, is currently investigated.

5 CONCLUSIONS

In this paper a finite element model for the description of the effects of compensation grouting was introduced. The proposed model was validated for the grouting work performed for the active settlement control for Antwerp Central Station.

The basic effects of compensation grouting with respect to settlement reduction and stresses of structural elements were demonstrated with the numerical approach adopted. It clearly turned out that a significant reduction of settlements could be expected. The additional bending moments acting on the pipe jacked underneath the footing was rather small.

The finite element analysis of the actual grouting in Antwerp could reproduce the main construction stages in a very reasonable way. Differences between calculated and injected quantities occurred, but similar tendencies for different grouting stages were observed. It can be concluded that the finite element method could be a valuable tool in the design process for future compensation grouting projects.

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