line 2 project performed successfully. Ground movements were held within limits which were typically much smaller than what would be expected with a conventional shield. Typical long-term surface settlement is in the range of 33 to 41 mm with the maximum surface settlement being not more than 55 mm. Principal conclusions drawn from the observed performance are as follows:

1. The advance of the EPB shield at the instrumentation site led to initial outward movements from the shield, movements that were largely lateral and confined to the soil immediately to the side and in front of the shield. The magnitude of the initial heaves correlated directly with the level of the earth pressure measured inside the soil chamber in the shield. The earth pressure developed inside the soil chamber can be represented by the earth pressure ratio (EPR). When the EPR is in the range of 0.95 to 1.05, the initial ground movements induced by the approaching tunnel are very small. Excessively high EPR would induce initial ground heaving, while excessively low EPR would result in initial ground settlement.

2. From the observation of ground movements, the ground response to an advancing EPB shield can be classified into five distinct phases, namely: (i) Phase I - the approach of the tunnel; (ii) Phase II - the passing of the shield body; (iii) Phase III - tail void closure; (iv) Phase IV - consolidation of grouting material and disturbed soil ; and (v) Phase V - secondary consolidation of disturbed soils. The relative proportions of settlements at each phase are highly dependent upon the construction controls adopted during the shield tunneling. A properly controlled tunneling process could successfully reduce the magnitude of surface settlement.

3. The control of shield alignment during the advance of the shield remains one of the most important tunnel control parameters. Most importantly, the actual volume of the grouting material injected into the tail-void created by the over-cutting actions of the shield as reflected by the grout filling ratio (GFR) is the key parameter to limit the long-term settlement developed at the ground surface. For the Shanghai Metro Tunnel, increasing the GFR to about 1.1 to 1.15 would significantly reduce the long-term surface settlement.

ACKNOWLEDGMENTS

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The effects of compensation grouting on segmental tunnel linings

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ABSTRACT: The effectiveness of compensation grouting in mitigating surface settlements due to tunnelling subsidence has been widely proven. However, excessive grout pressures may cause deformations of tunnel linings and heaving of the ground surface. At the London Docklands Light Railway Lewisham Extension tunnelling site, the contractor has carried out an extensive monitoring programme to measure ground and building movements due to tunnelling works incorporating compensation grouting. The movements of a building and the tunnel lining due to tunnelling and compensation grouting are presented. Preliminary results from finite element analyses simulating the influence of compensation grouting are also presented.

1 INTRODUCTION

The project described in this paper is the London Docklands Light Railway (DLR) Lewisham Extension. Figure 1 shows the site layout.

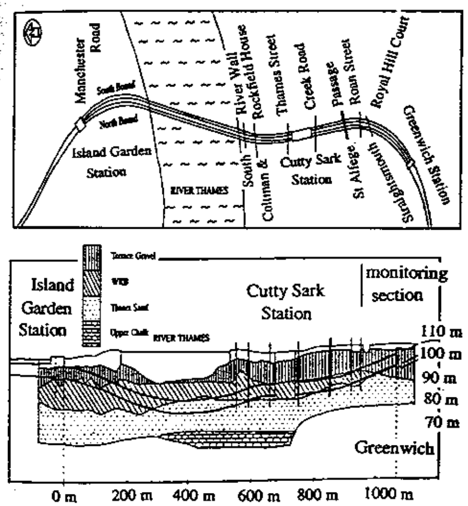

Figure 1. London Dockland Light Railway (DLR) Lewsiham Extension site layout.

Two 5.2m internal diameter tunnels, southbound and northbound, were constructed (using a slurry TBM) from Island Gardens station on the southern end of the Isle of Dogs to Greenwich station in the south east of London. The ground strata are Made Ground underlain by alluvium, Terrace Gravel, Woolwich & Reading Beds (WRB), which is now known as the Lambeth Group, and Thanet Sand. The tunnels are mainly located in Terrace Gravel and WRB except under the River Thames where they are in Thanet Sand for part of their length. The tunnel cover (distance from tunnel crown to the ground surface) varied from a few metres to a maximum of 20m. The allowable movement of sensitive structures was a maximum of 5mm for heaving and 15mm for settlement, and the angular distortion was to be less than 1:1000 to 1:1500 depending on the structure.

Tunnelling in cohesionless soils will result in instantaneous settlement. In order to maintain net movements within the above limits, compensation grouting was used at some locations, particularly where the tunnels were in the Terrace Gravels. The compensation grouting can be used either to jack up the structure after it has settled, provided the incremental settlements and heaves are acceptably small, or to prevent the structure from ever settling by more than a specified amount. The contractor, Nishimatsu Construction Ltd, preferred the latter approach as the rate of settlement in gravel is high. The contractor and Cambridge University initiated a research project to investigate ground movements due to the DLR tunnelling project and the effects of

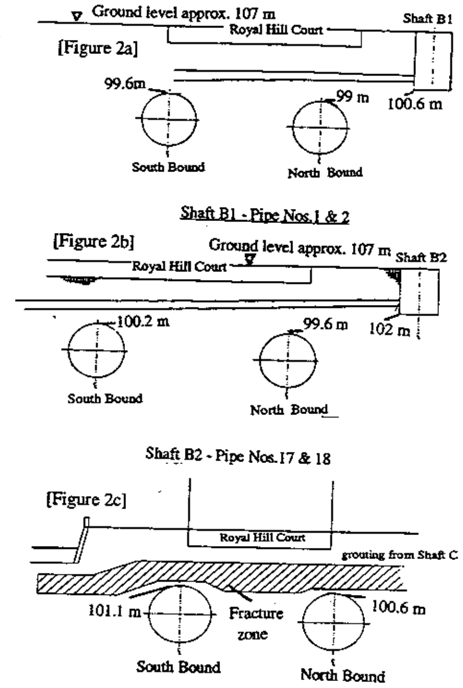

Figure 2. Typical grouting array at Royal Hill Court.

compensation grouting on the tunnel lining.

This paper presents the field measurements together with finite element simulations of ground movements incorporating compensation grouting.

2 COMPENSATION GROUTING

The specialist sub-contractor undertaking the compensation grouting was Keller. The grout used was a cement/bentonite based mix at relatively low water cement ratio with a proportion of silicate so that it can set very fast. The grout was pumped from tube-a-manchette grout injection pipes which were installed, between the tunnels and building foundations, from shafts (see Figure 2).

The grouting layout was designed to cover the whole foundation area and was injected from more than one shaft depending on the size of the building. The grout injection was stopped when a building just started to lift.

There were two stages in the grouting: conditioning and compensation. Prior to tunnelling, the ground above the tunnel was conditioned by

Table 1. Properties of compensation grout.

Material	Quantity
Cement (kg)	50
Bentonite (kg)	5
Pulverised Fuel Ash (kg)	50
Silicate (litre)	1.4
Water (litre)	110
Compressive strength 28days (N/mm^2)	3 - 5

Table 2. Properties of cavity grout.

Material	Quantity
Cement + Slag (kg)	230
Clay + bentonite (kg)	30
Stabilizer (kg)	3
Water (litre)	830
TG accelerator (litre)	80
Liquid flow value (sec)	8.5 - 10.5
Gel time (sec)	6 - 10
Compressive strength 28days (kgf/cm^2)	26

Table 3. Grouting programme.

Process	Quantity
Injection vol. (litre / sleeve)	30
Injection rate (litre / min)	5 - 20
Grouting pressure (bar), 0-1m from tunnel crown	5
Grouting pressure (bar), 1-2m from tunnel crown	7
Grouting pressure (bar), > 2m from tunnel crown	>7
Grout volume (litre/ m length of tunnel)	705

pumping grout into the soil. The conditioning was essential to tighten up the ground so that the subsequent compensation grouting would be more efficient.

After the conditioning, the compensation grouting was implemented to remedy any settlement occurring instantly after the advance of the TBM face.

In addition to this, a thixotropic gel grouting system was used to fill the void between the ground and the lining segment. This cavity grouting limits further settlement and also stops water from seeping into the tunnel.

The properties of the compensation grout and cavity grout are detailed in Table 1 and Table 2. Details of the grouting programme are given in Table 3.

3 FIELD MEASUREMENTS

The movements due to tunnelling were measured on transverse cross sections at various locations. The monitoring was carried out using precise levelling survey techniques. The measurements include ground, building and tunnel lining deformation due to the excavation and compensation grouting. Only building movements and tunnel lining deformation are reported in this paper.

3.1 *Building movements*

The building movements at the Royal Hill Court (RHC) are presented here. The soil profile beneath the RHC is 2.5m Made Ground, followed by 11.5m Terrace Gravel, 0.5m WRB and Thanet Sand. The tunnel is in Terrace Gravel with its axis at a depth of about 11m. Because of the sensitivity of the structure, it was decided to use compensation grouting to limit settlements due to tunnelling.

The Royal Hill Court is a 2-storey reinforced concrete frame building founded on shallow foundations in the Terrace Gravel. Figure 2 shows typical cross sections through the grouting array. Grouting tubes from various shafts covered the footprint of the foundations of the building. The grouting was about 2-3m from the tunnel crown.

The settlements of six points (611- 616) located on the edge of the building are shown in Figure 3. The solid vertical lines in the Figure indicate the onset of grouting, each episode taking approximately half a day. The effectiveness of compensation grouting in reducing settlements can be seen. The grouting from shaft B1 lifted the building by about 11mm. Similarly, grouting from shafts B2 and C caused an average lift of 7mm.

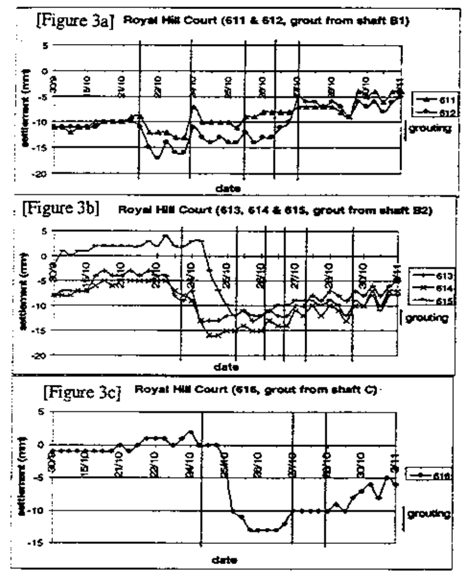

Figure 3. Settlement at Royal Hill Court before and after compensation grouting.

In general compensation grouting has limited the final settlement to less than 8mm for all points concerned. Hence, the compensation grouting has resulted in the construction of the tunnels with building settlements less than 15mm, and the angular distortions less than 1:1000, thereby satisfying the specified design criteria.

3.2 *Tunnel lining deformation*

Segmental tunnel linings were used on the project. The lining was made of 50 N/mm^2 concrete and strengthened by high yield reinforcement. The segments were 250mm thick, 5.2m internal diameter, and 1.2m wide. Each ring consists of 1 key segment, 2 top segments and 3 ordinary segments and they are bolted together with 20mm diameter curved bolts. All longitudinal joints are fitted with bituminous packers to facilitate some joint rotation. The lining has been designed for a 120 year design life.

As shown in the previous section, compensation grouting reduced the building settlement. However, compensation grouting will not only reduce the settlement at or near the ground surface but may also increase the stress on the lining and deform it. It is very important to monitor the tunnel lining deformation, especially when the grouting is close to the tunnel crown as in this project.

Initial attempts were made to measure the lining deformation using only a tape extensometer. However, it was difficult to measure changes in vertical diameter mainly because of the presence of the TBM. Then a combination of theodolite, tape and staff was used, see Figure 4. The theodolite measured movements of targets 1 - 6. A tape extensometer was used to measure vertical movements of targets 4 and 5. Horizontal movements were measured using a staff.

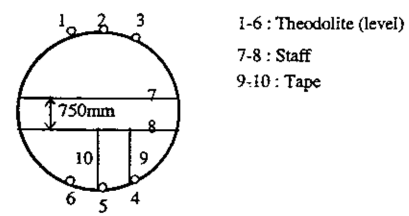

Figure 4. Measurement points on tunnel lining.

Examples of the effect of compensation grouting on the tunnel lining are shown in Figure 5. The hatched zones indicate the grouting periods. The Figure shows the changes of vertical and horizontal diameter of selected tunnel rings as the tunnel face advances away from the ring and new rings are erected. The numbers of the new rings erected are shown on the x-axis. The tunnel rings selected are Ring 833 (southbound), Ring 888 (southbound) and Ring 820 (northbound), in all the Royal Hill Court area.

Following the first grouting episode, the vertical diameter of ring 833 shortened by 1.75mm and the horizontal diameter elongated by 0.5mm, approximately. However, after the second grout injection the vertical diameter increased. No information on the horizontal diameter change is available following the second grout injection.

For Ring 888, compensation grouting resulted in an increase of the vertical diameter and shortening of the horizontal diameter. In other words, the lining deformed in a vertical elongation mode. This is contrary to what is expected.

Similar to Ring 833, Ring 820 of the northbound tunnel depicted the expected type of lining deformation, i.e. vertical squatting, when grouting was applied. The decrease in vertical diameter was 4.8 mm and increase in horizontal diameter was 1.1 mm.

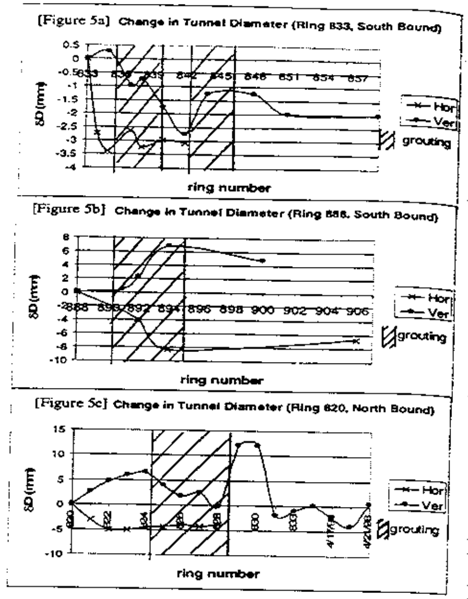

Figure 5. The effect of compensation grouting on tunnel lining.

4 FINITE ELEMENT ANALYSES

Finite Element (FE) analyses were carried out to simulate the effect of compensation grouting. First, FE analyses were used to back-analyse ground movements due to tunnelling alone. The back analyses were to gain confidence in selection of a suitable soil model and its parameters in the simulation of the tunnelling. The FE analysis was then used to simulate effects of compensation grouting.

The FE analyses were plane strain. Reducing approximately 30% of the in-situ stress around the tunnel simulated the tunnel construction. The lining was then placed and the remaining in-situ stresses were removed. This is to account for 3D effects which cannot realistically be simulated in plane strain.

The ABAQUS (HKS, 1998) FE code was used for all the simulations. The Schofield soil model (Schofield, 1980) was used for all soils except Thanet Sand which was modelled by a Mohr Coulomb model. The Schofield model has an original Cam Clay yield surface on the wet side, and a tension cut off and the Hvorslev surface on the dry side. This model was selected because the overconsolidated nature of the soils at London Docklands site can be modelled well by the Hvorslev surface. The tension cut-off criterion is to avoid the minor principal stress becoming negative.

Recent publications suggest that it is necessary to use non-linear stiffness inside the yield surface in order to realistically reproduce the surface settlement profile due to tunnelling. This was modelled using non-linear equations suggested by Dasari (1996).

The parameters for the non-linear equations were determined to fit triaxial laboratory data published by Hight et al. (1993) for Terrace Gravel and Linney and Page (1996) for WRB. The shear stiffness-strain curves used for Terrace Gravel and WRB in FE analysis together with experimental data are shown in Figure 6. Thanet Sand was modelled as a Mohr Coulomb material.

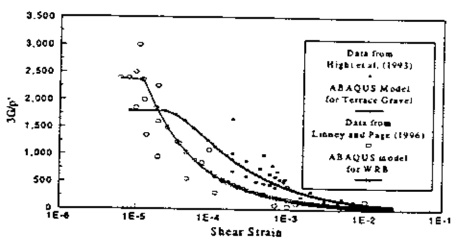

Figure 6. Shear stiffness-strain curves for Terrace Gravel and WRB.

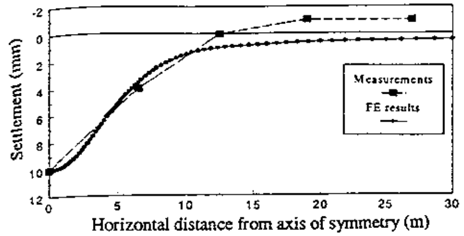

Figure 7. Surface settlement at RHC.

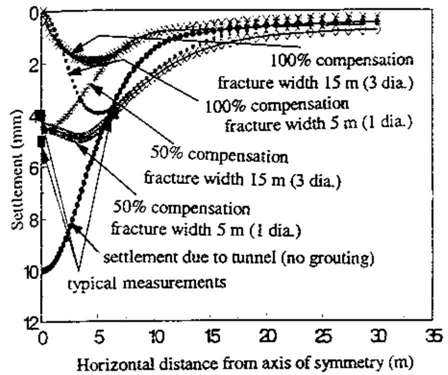

Figure 8. Predicted compensated settlement with grouting.

4.1 *Results from FE analyses*

The building settlement measurements at the RHC due to the southbound tunnel together with FE predictions of the surface settlements are shown in Figure 7. There is a reasonable agreement of the predicted and observed maximum settlement. It is well known that FE simulation predicts excessive settlements in the far field. The effect is not so pronounced using the non-linear model employed here. The good agreement between FE results and measurements could also be due to the stiffness of the RHC building, which would tend to make the settlement profile wider (Mair and Taylor, 1997).

The FE program was then used to predict the effect of compensation grouting at the RHC. It was assumed that compensation grouting would cause horizontal fracture. The grouting was then modelled by applying a constant pressure along the pre-defined horizontal fracture. The grouting pressure should have been applied in stages as the settlement occurred (rigorous approach) rather than applying grouting at the end to compensate for the whole settlement (simple approach). Kovacevic et al. (1996) carried out numerical analyses with these simple and rigorous approaches and reported that both methods gave similar results. Here the simple approach has been adopted.

Two analyses were carried out, with two fracture widths 5m and 15m corresponding to one and three tunnel diameters, respectively. In each case grouting pressure was increased until the maximum surface settlement had been fully compensated. The pressure required to fully compensate the settlement in each case was very close to the initial total vertical pressure at the grout level. The surface settlement profiles, for the 5m and 15m fracture width, due to tunnelling and compensation grouting, are shown in Figure 8. Two profiles correspond to 50% of compensation and two more correspond to full compensation. The measurements from the RHC are also included in this Figure. There is a reasonable agreement between measured settlements and FE results with 15m fracture width assuming 50% compensation. It can be seen that 5m fracture width predicted a concentrated settlement profile. This is unfavourable as it will result in more angular distortion. The angular distortion due to 15m fracture width is smaller. Therefore, it is better to grout over a wider area.

The lining was modelled as an elastic material with reduced integration elements. The predicted horizontal and vertical movements of the tunnel lining due to compensation grouting are shown in Figure 9. The vertical diameter of the tunnel reduced and the horizontal diameter increased. In other words, the lining deformed in a vertical squatting and a horizontal elongation mode. The magnitude of movements is quite small compared to field measurements. It may be that field measurements included effects of the tunnel build which were not modelled in the FE analyses.

The predicted increase of hoop stress due to compensation grouting is shown in Figure 10 in terms of the average stress at the centre of the lining

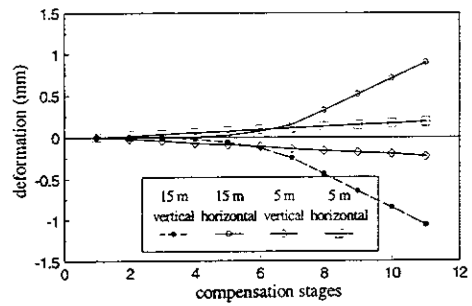

Figure 9. Predicted vertical and horizontal deformation due to compensation grouting.

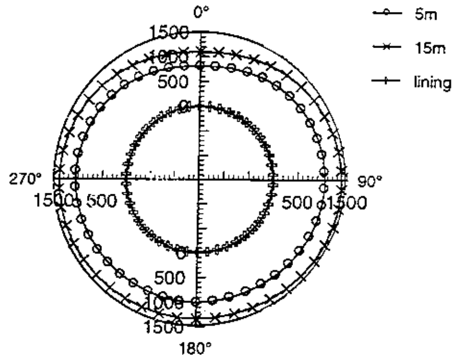

Figure 10. Predicted increase of hoop stress (kPa) due to compensation grouting.

elements. The lining elements near the crown were in sagging mode (tension on tunnel side) and the elements near the springing were in hogging mode (tension on soil side). This will result in a variation of hoop stress across the section and create bending. The maximum increase in hoop stress was about 1.4 MPa. This increase of hoop stress plus the existing overburden pressure are well below the lining concrete limit stress.

5 CONCLUSIONS

The measured settlements due to tunnelling and compensation grouting at the Royal Hill Court in London Docklands are presented. In the absence of compensation grouting the maximum settlement due to tunnelling was only about 15mm. This corresponds to a volume loss of less than 1%. Thus it can be concluded that good control of tunnel construction had been implemented. The use of compensation grouting helped to reduce building settlements to less than 8mm. The observed lining deformation mode was variable. The lining deformations were small, being less than 5mm. Finite element simulations were also carried out to simulate the influence of compensation grouting. The results from the FE analyses were generally in reasonable agreement with measurements.

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Pipe jacking through soft alluvial clay near London: A case history

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ABSTRACT: A new sewer tunnel was required as part of a major scheme linking two pumping stations near London. The tunnel, in very soft alluvium close to the River Thames, was constructed using the pipe jacking technique and was monitored as part of the Pipe Jacking Research Programme at the University of Oxford. This paper describes the monitoring work and presents some of the main findings from the interpretation of field measurements made. The fieldwork primarily comprised the measurement of jacking forces, normal and shear stresses at the pipe-soil interface, and the magnitude and extent of ground disturbance.

1 INTRODUCTION

Pipe jacking, including microtunnelling, has for some time been an important technique for the construction of small diameter tunnels, generally up to 2m, in urban areas. Craig (1983), in his review of pipe jacking procedures, identified a need for research into the technique and a programme in the UK was initiated.

The Pipe Jacking Research Programme at Oxford University commenced in 1986 and continues today. The overall objectives of the research programme follow the recommendations listed in Craig's report (loc cit) but they have been added to and refined to reflect advances made since the 1980s. Five phases of research have been completed, the current phase being the sixth. The second and third phases involved monitoring the construction of pipe jacked tunnels under construction. Nine tunnels were monitored between 1989 and 1996 and this paper provides a case history of the final monitoring exercise.

The sewer tunnel that was monitored was required by Anglian Water as part of the Thurrock Southern Trunk Sewerage Scheme. The overall scheme connected new and existing pumping stations east of London on the north bank of the River Thames.

The route of the monitored length of tunnel is shown in Figure 1. The 1.5m internal diameter tunnel was driven from shaft L to N and generally passed beneath public parkland but ran close to a large residential area. The River Thames was very close, passing approximately 300m south of the drive. The tunnel inverts at shaft L and N were 7.0m and 7.2m below ground level respectively, and average depth to axis was 6.4m. It was proposed that an intermediate shaft (shown as M in Figure 1) would not be required during construction. The driven length of tunnel was therefore 260m.

Hydraulically, only a 1.0m pipe was required for the sewer but the method of construction accepted by the Client was a 1.5m pipe jack, with excavation by an Iseki Unclemole (slurry pressure balance shield) and using steel-banded concrete pipes, to be relined with glass reinforced plastic (GRP) pipes.

The aims of the monitoring work on the pipe jacked tunnel can be summarised as:

i. collect information on jacking loads to contribute to the existing body of data
ii. analyse and interpret contact stresses at the

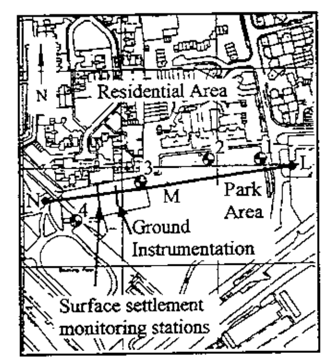

Figure 1. Location of monitored tunnel.