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TC28 – Underground construction in soft ground: Some recent technical advances

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1 INTRODUCTION

Much has been achieved by Technical Committee 28 (Underground Construction in Soft Ground) in recent years. In 1994, under the leadership of the then Chairman, Professor K. Fujita, a one-day International Symposium was held in New Delhi – just before the 13th ICSMGE. Following the success of this, under the Chairmanship of the author it was decided to initiate a series of two-day International Symposia on Geotechnical Aspects of Underground Construction in Soft Ground, with a third day devoted to technical site visits to underground construction projects. These International Symposia have been held every three years over the past 10 years as follows:

1996	London
1999	Tokyo
2002	Toulouse
2005	Amsterdam

The Proceedings of the London, Tokyo and Toulouse Symposia have been published, as illustrated in Figure 1. The Preprint volume of the Proceedings of the Amsterdam Symposium (held in June 2005) contained 122 papers, together with General Reports for each of the technical sessions; it is anticipated that the final Proceedings will be published before the end of 2005.

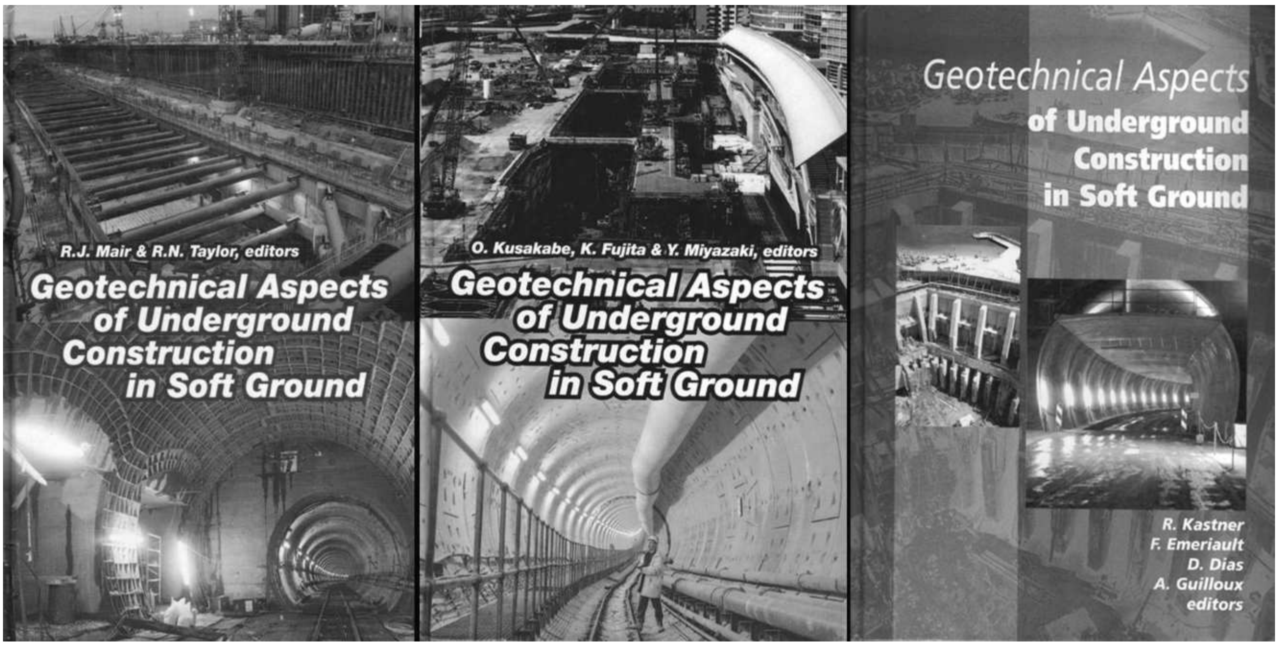
In each case there has been a wealth of information published in the Proceedings, with a total in excess of 400 papers published on a wide variety of topics of major technical interest, mainly on case histories and new research.

Significant technical advances have been made over the 10 year period 1996-2005, and in the period 2001-2005 TC28 has been particularly active through its organization of two major International Symposia in Toulouse (2002) and Amsterdam (2005).

This Report is a personal reflection of the author on some of the more significant technical advances over the period 2001-2005 arising directly from the work of TC28. Inevitably, because of constraints of space, only a few subjects have been selected.

In keeping with its Terms of Reference, the principal areas of interest of TC28 have been as follows:

1. Case histories
2. Design and construction of tunnels and deep excavations in the urban environment
3. Ground improvement schemes and displacement of surrounding ground and of adjacent structures
4. Roles of analysis, and physical and numerical modelling



London 1996 Tokyo 1999 Toulouse 2002

Figure 1. Proceedings of previous TC28 Symposia

2 SELECTED TECHNICAL ADVANCES

The selected technical advances that are addressed in this Report are as follows:

1. Bored tunnels – construction and settlement control
   - EPB machine performance
   - Tail void grouting to control settlement
2. Effects of tunnelling beneath piles
3. Tunnels – modelling and prediction
   - Centrifuge modelling
   - Numerical analysis
4. Monitoring
5. Deep excavations in soft clay

Most of the examples selected for this Report featured or were referred to in the recent TC28 International Symposium held in Amsterdam in June 2005.

3 BORED TUNNELS – CONSTRUCTION AND SETTLEMENT CONTROL

3.1 *EPB machine performance*

There have been considerable technical advances in earth pressure balance pressure (EPB) tunnelling machine performance in recent years. Figure 2 shows the essential details of an EPB machine. Of paramount importance is the control of volume loss, particularly as the diameter of tunnels under construction is increasing. In soft ground the volume loss depends on a number of factors, including the ground conditions and soil properties, but successful control also depends crucially on machine parameters, particularly on face pressure, soil conditioning, and grouting details.

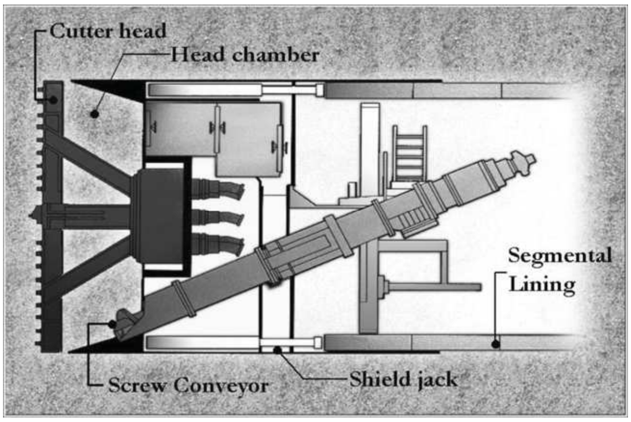


Figure 2. Schematic of typical EPB tunnelling machine

Bowers and Moss (2005) illustrate this by describing measurements of performance of EPB tunnels constructed for the Channel Tunnel Rail Link (CTRL) in the UK. Twin bored tunnels of 8.15m external diameter were constructed over a length of 18km through a very wide range of ground conditions: stiff clays of the Lambeth Group and London Clay formation, Lambeth Group sands and gravels, the Thanet Sand formation, Thames river terrace gravels, recent estuarine deposits (including alluvium and peat), and Chalk. Figure 3 shows the volume losses on one of the CTRL contracts, measured over a distance of about 7.5km. The following points are noteworthy:

1. For much of the route the volume losses averaged around 0.5% and in the best conditions prolonged runs resulted in significantly lower values.
2. At point A in Figure 3, corresponding to early in the drive, tunnelling was in mixed clays and sands; an experiment was undertaken to determine the effect of reducing the face pressure – this resulted in a rapid increase in volume loss, almost to 3%.
3. Around point B, where the tunnels were still in mixed clays and sands, further tests were conducted on the progressive reduction of soil conditioning; this resulted in the volume loss increasing until the 1% contract limit was reached. As for point A, reverting to normal operating procedures immediately resulted in small volume losses being achieved again.
4. For approximately the next 2.5 km (C on Figure 3) the tunnels were substantially in sand; bentonite injection was undertaken around the shield body and consistently low volume losses were achieved.
5. When the tunnels passed into the very stiff clays of the Lambeth Group (D on Figure 3), production slowed and the 1% contract limit was again reached. The machines were stopped and the cutterheads reconfigured, resulting in faster tunnelling and volume losses of around 0.5%.
6. Movement near the face was almost eliminated by constant maintenance of face pressure, and it was not just the mean pressure that was of significance but also the minimum level to which the pressure dropped.

In summary, the observations reported by Bowers and Moss (2005) have improved the understanding of complex EPB tunnelling machine performance. It is clear that the achievement of such machines in obtaining low volume losses depends critically on operator skill, as well as on important machine parameters such as face pressure, soil conditioning, and grouting details.

3.2 *Soil conditioning in EPB tunnelling*

A key element of successful EPB tunnel machine operation is control of the excavated spoil travelling up the screw conveyor.

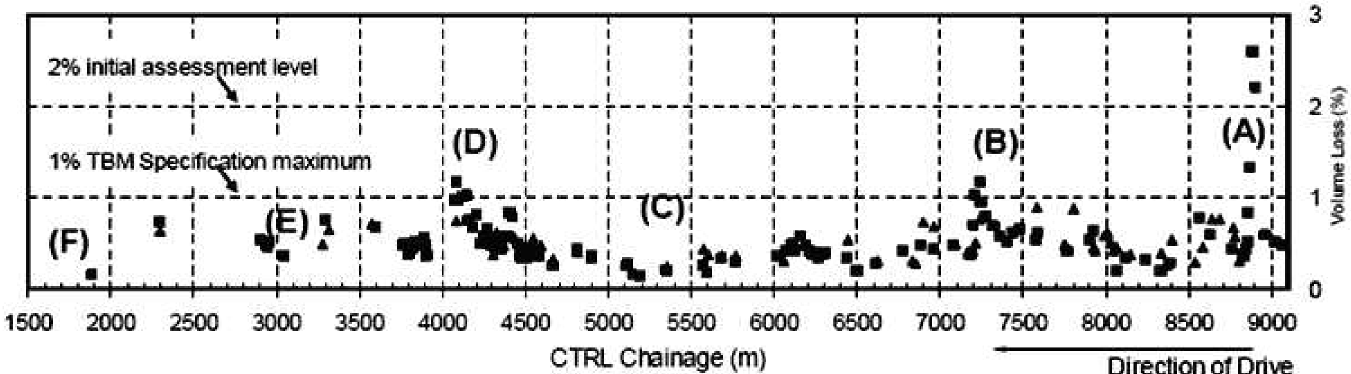


Figure 3. Volume losses for tunnelling on CTRL project in London (Bowers and Moss, 2005)

Recent research reported by Merritt et al (2003) has provided improved understanding of the mechanics of conditioned clay in a screw conveyor. Pressure changes along a model EPB screw conveyor are shown in Figure 4. The model screw conveyor was instrumented to measure normal stresses and pore pressures (as well as shear stresses). The following key points should be noted:

1. Conditioning agents injected to modify the excavated soil properties can improve tunnelling machine performance considerably
2. Foams and polymers are commonly used as conditioners
3. Ideally, the soil/conditioner mix should form a soft, plastic paste with S_u = 5 to 25 kPa
4. Foams perform poorly as conditioning agents for stiff, high plasticity clays
5. Effective soil conditioning allows controlled operation of the screw conveyor with uniform pressure drop along the conveyor
6. Linear pressure gradients along the conveyor result from constant shear stresses at the soil-casing interface
7. Pressure gradients along the screw conveyor are influenced by operating conditions (screw speed, discharge condition) and soil/conditioner undrained strength, S_u

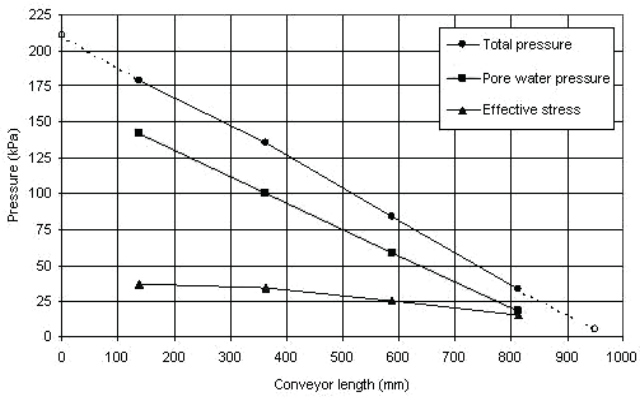


Figure 4. Pressure changes along model EPB screw conveyor (Merritt et al, 2003)

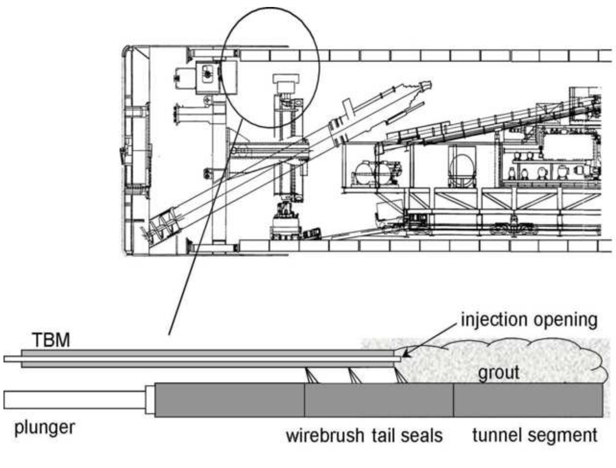


Figure 5. Tail void grouting (Bezuijen and Talmon, 2005)

3.3 *Tail void grouting to control settlement*

Successful control of settlement frequently depends on effective tail void grouting, depicted in Figure 5. Bezuijen and Talmon (2005) address the behaviour of TBM's during construction, particularly in relation to pressure distributions ahead of and at the face, and around the segments due to grouting. Grout pressures and the pressure gradients play a very important role in the history of the loading experienced by the tunnel lining. Figure 6 shows the measured grout pressures around a lining segment as a function of distance travelled by the TBM in the case of a 9.5m diameter tunnel constructed in sand below the water table. It can be clearly seen that when the TBM halts (during erection of a new lining segment) the grout pressures drop, but they increase again when the TBM advances. It can also be seen that commencement of consolidation and hardening of the grout (after the TBM has progressed about 4m) leads to an overall reduction in measured pressure, and after the TBM has progressed about 6m the grout has hardened and the pressure becomes constant (and comparable to the pore water pressure).

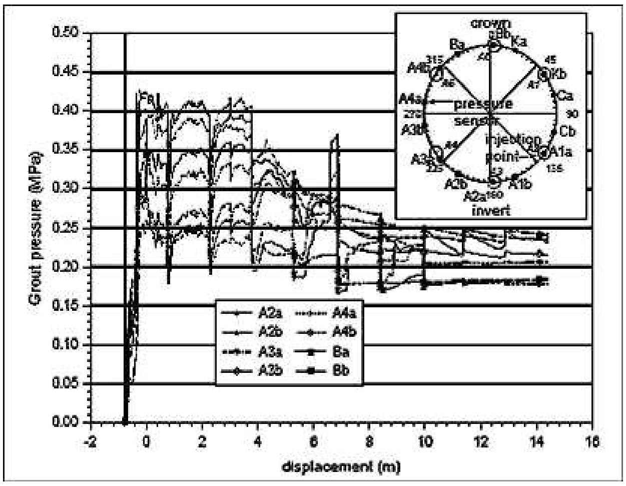


Figure 6. Measured grout pressures acting on tunnel lining segment (Bezuijen and Talmon, 2005)

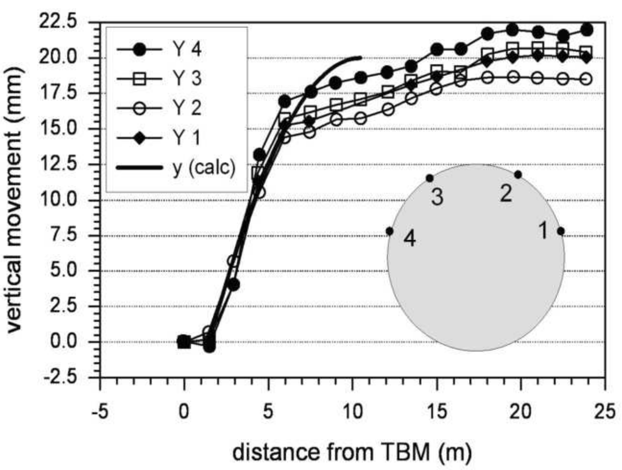


Figure 7. Vertical movement of tunnel lining segment immediately after erection (Bezuijen and Talmon, 2005)

Bezuijen and Talmon (2005) also show how completed tunnel linings can become buoyant in the fluid grout. Figure 7 shows vertical movement of the tunnel lining as a function of distance behind the TBM, for the same tunnel for which lining pressure measurements are shown in Figure 6. It can be clearly seen that the lining floats upward while surrounded by liquid grout. The idealised behaviour of the lining, showing flotation, is illustrated in Figure 8.

The measurements and interpretation of the grout behaviour reported by Bezuijen and Talmon (2005) and Talmon and Bezuijen (2005) is an important development in improving understanding of TBM performance in soft soils.

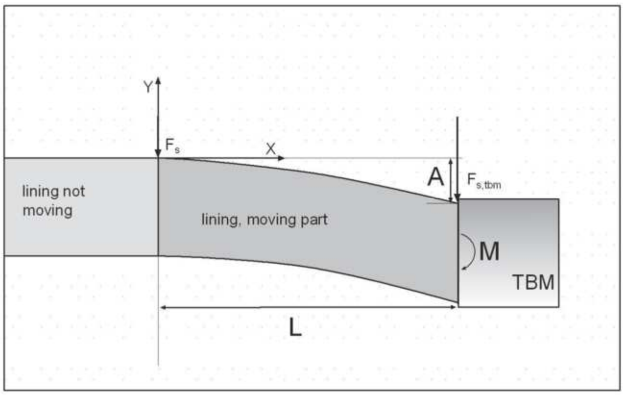


Figure 8. Idealised behaviour of tunnel lining showing flotation in fluid grout (Talmon and Bezuijen, 2005)

4 EFFECTS OF TUNNELLING BENEATH PILES

As more tunnelling schemes are contemplated for urban areas, it is becoming increasingly common to tunnel close beneath piles. It has therefore become important to understand more about the effects of tunnelling beneath piled foundations, particularly when they are heavily loaded and primarily end-bearing. Figure 9 illustrates an example of a recent case in which 8m diameter tunnels for the Channel Tunnel Rail Link in the UK were constructed beneath a bridge supported on driven piles, end-bearing in the Terrace Gravels (Jacobsz et al, 2005). The gravels were overlain by 8-12m of very soft alluvial silts, clays and organic soils, and hence the piles possessed negligible shaft capacity.

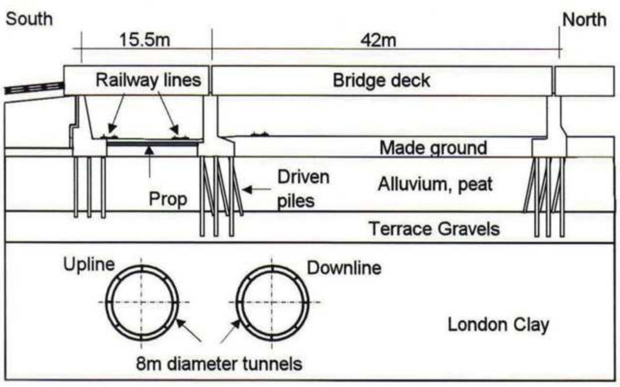


Figure 9. Case history of tunnelling beneath piles on CTRL project in London (Jacobsz et al, 2005)

Figure 10 shows the settlement of one of the bridge piers during construction of the Downline tunnel, for which the volume loss was just greater than 1%. Also shown is the settlement of the ground surface adjacent to the bridge. The surface settlement immediately after passage of the TBM was about 18mm, increasing to 20mm as consolidation occurred, with the pier settling a little less. Jacobsz et al conclude that the piles settled by the same amount as the soil at the depth of the pile bases.

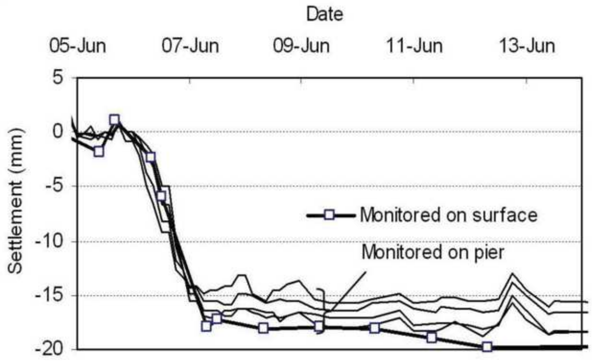


Figure 10. Observed settlement of piled bridge pier and ground surface on CTRL project in London (Jacobsz et al, 2005)

These field observations are consistent with the findings of Jacobsz et al (2004) from centrifuge tests and Selemetas et al (2005) from field tests. Figure 11 shows zones of influence – if the pile toe is located in zone A for example, the pile head settlement will be greater than the ground surface settlement, whereas in zone B it will be very similar, and in zone C it will be less.

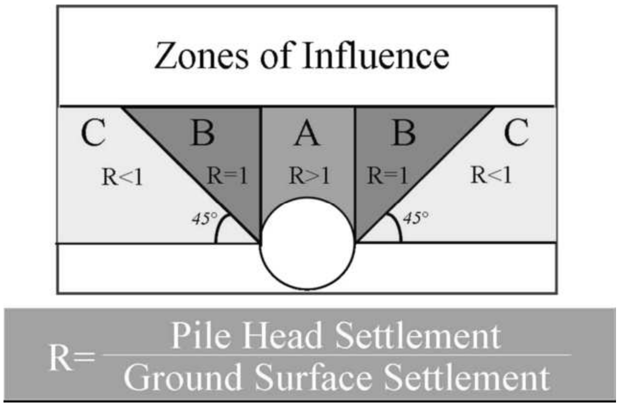


Figure 11. Zones of influence relating to pile toe locations relative to tunnel position (Selemetas et al, 2005)

These findings are also reaonably consistent with the conclusions of Kaalberg et al (2005) in their paper to this conference, based on field tests; a comprehensive field trial was undertaken during construction of the Second Heinenoord tunnel. Loaded and instrumented wooden and concrete piles were installed above a pair of 8.3m diameter tunnels to be constructed and their response closely monitored. The piles were installed in clay columns to reduce their shaft friction capacity. The volume losses for the passage of the two tunnels in the field trials were reported by the authors as 1-2% and 0.75% respectively.

In summary, zones of different pile behaviour have been identified. Depending on which zone the pile toe is located in, it is now possible to predict whether the pile will settle more than the ground surface or less. This allows improved prediction of pile settlement due to tunnelling for small volume losses.

5 TUNNELS – MODELLING AND PREDICTION

5.1 *Centrifuge modelling*

Centrifuge modelling was used by Jacobsz et al (2004) to investigate the detailed mechanics of pile response to tunnel construction beneath the pile base. The piles studied were driven piles in sand. Figure 12 shows how the observed pile settlement was less than the settlement of the ground surface at the pile head for volume losses smaller than 1.3%. This is consistent with the zones of influence shown in Figure 11. However, as the volume loss approached 1.3% the pile settlement increased rapidly and the pile failed.

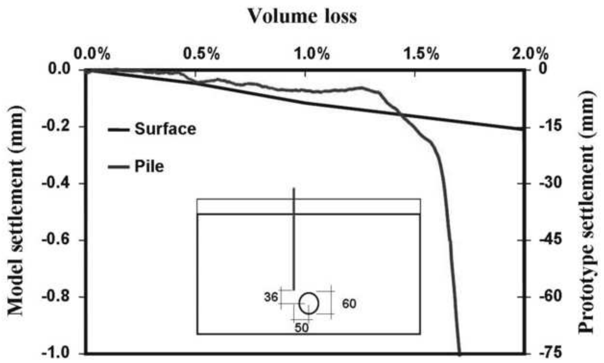


Figure 12. Effects of tunnelling near piled foundation – Cambridge centrifuge model tests (Jacobsz et al, 2004)

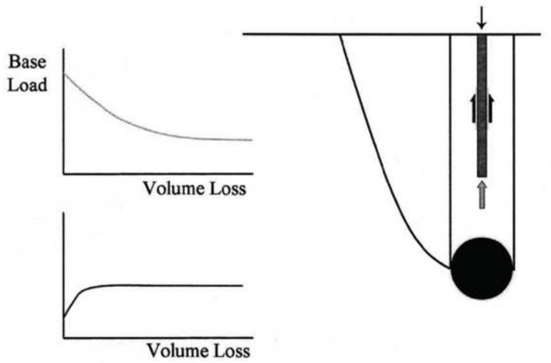


Figure 13. Mechanisms of pile load distribution changes for pile close to and above the tunnel

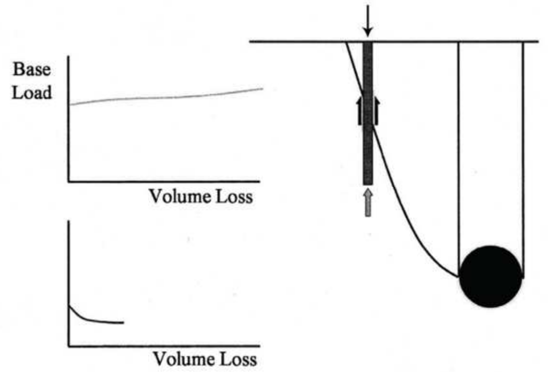


Figure 14. Mechanisms of pile load distribution changes for pile further away from the tunnel

The mechanisms relating to this are illustrated in Figures 13 and 14. In cases where the pile toe is located close to and above the tunnel (Figure 13), the base load reduces with increasing volume loss. For equilibrium to be maintained, the shaft friction increases and this is accompanied by some small settlement of the pile. When the full shaft friction capacity is mobilized, the pile is no longer in equilibrium and the pile fails. In contrast, if the pile toe is located at some distance to the side of the tunnel (Figure 14) and the toe of the pile is below the zone of ground movement, the base load increases with increasing volume loss; this occurs as positive skin friction decreases in the upper part of the pile and increases in the lower part, with an overall reduction in positive friction, resulting in only small pile settlements.

In summary, depending on the location of the pile toe, volume loss may cause a reduction in pile base load. For larger volume losses maximum skin friction may be mobilized, in which case the pile will settle rapidly with little warning. Centrifuge modelling has proved very valuable in demonstrating these effects, especially for larger volume losses.

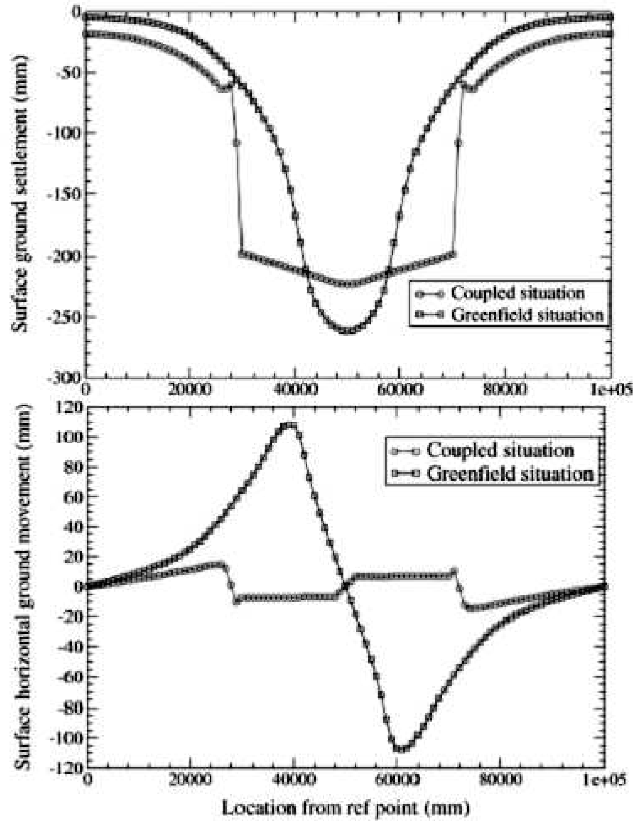


Figure 15. Influence of building stiffness on response to tunnelling-induced ground movements (Boonpichetvong et al, 2005)

5.2 *Numerical analysis*

There have been significant advances in capabilities in recent years for analyzing tunnels and deep excavations – 3D FE analyses are becoming more common, although there are difficulties associated with these and considerable skill and experience is needed if meaningful results are to be obtained. It is now possible to perform more realistic numerical modelling of interaction between ground and buildings subject to excavation-induced ground movements. Potts and Addenbrooke (1997) presented results of parametric studies of tunnelling-induced ground movements on buildings of different stiffness idealized as elastic beams.

A recent example of an analysis of tunnelling effects on a masonry building is presented by Boonpichetvong et al, 2005), who modelled the cracking of the masonry, slipping and gap-opening of the foundation-soil interface, as well as non-linearity of the soil itself. Figure 15 shows the clear influence of the stiffness of the building (shown as "coupled situation") in modifying the "greenfield" settlement profile. Of particular significance is the substantial reduction in horizontal ground movements transmitted into the building. This has been noted in field measurements of building response to tunnelling (Mair, 2003) and is of major significance when potential building damage is being assessed during the design stage of a tunnelling project: it is often the case that it is overly conservative to assume that the "greenfield" horizontal surface ground movements will be transmitted into the building foundation.

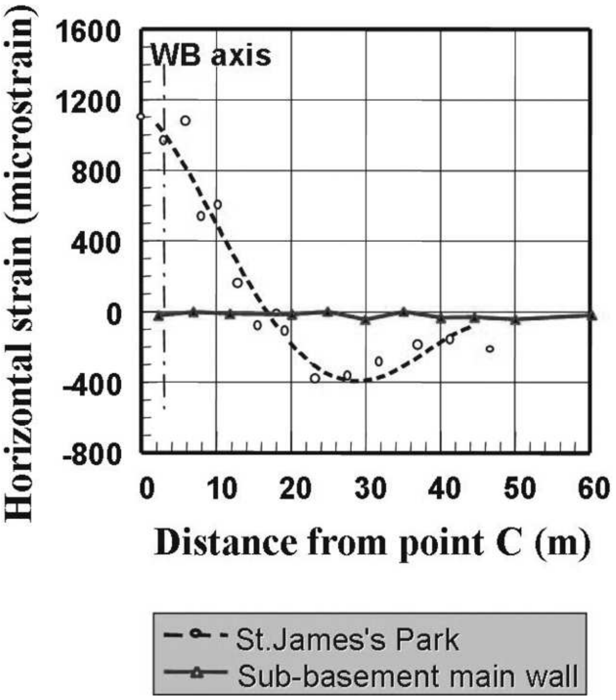


Figure 16. Comparison of building and greenfield response (horizontal strain) to tunnelling (Viggiani and Standing, 2001)

Figure 17. Automated total station in use for monitoring North-South line tunnel construction project in Amsterdam (Van der Poel et al, 2005)

6 MONITORING

The importance of monitoring is paramount, especially in complex cases of soil-structure interaction where the possible effects of the tunnel or deep excavation on a building might not be fully understood. A good example of this is shown in Figure 16, in which the response of the Treasury building in London in terms of the horizontal strain induced by tunnelling was compared with the "greenfield" response measured in an adjacent park (Viggiani and Standing, 2001). Almost insignificant horizontal strains were transmitted into the building, which is of important practical significance, as noted by Mair (2003).

There have been significant advances in resolution and accuracy of measuring systems, especially with automated devices that provide real-time readings. Figure 17 shows an automated total station in use for the North-South line construction in Amsterdam (Van der Poel et al, 2005). Figure 18 illustrates the very comprehensive monitoring system in place for one of the underground stations: for the whole project thousands of optical targets were mounted on surface structures and read by 74 robotic total stations of the type illustrated in Figure 17. Van der Poel et al describe how the data are stored, presented and updated on a GIS database, which also includes trigger levels so that warnings can be given when movements approach certain pre-determined values.

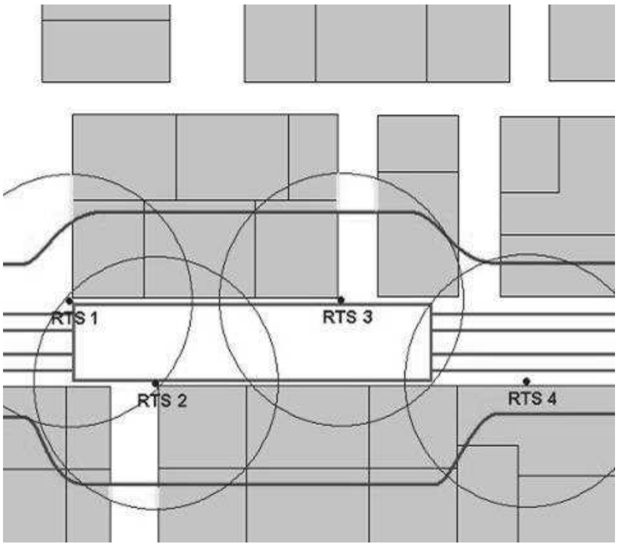


Figure 18. Coverage of total station monitoring for an underground station under construction in Amsterdam (Van der Poel et al, 2005)

Another significant new development in monitoring is described by Take et al (2005): a method of real-time image analysis has been developed involving remote digital photography and a Particle Image Velocimetry (PIV) processing technique in conjunction with data transfer and automated web-based update systems. The system was developed at Cambridge University and has been successfully applied to monitoring of a masonry retaining wall influenced by tunnelling for the Channel Tunnel Rail Link project in London. This very economic system shows considerable promise for future projects in view of its versatility, accuracy and real-time acquisition capability.

7 DEEP EXCAVATIONS IN SOFT CLAY

In recent years the depth of excavations in soft clay has increased. Deep deposits of near normally or normally consolidated clay can lead to major design and construction problems for deep excavations. Many lessons can be learned from the considerable experience of deep excavations in soft clay in Singapore in recent years. A number of innovative solutions have been developed to ensure basal stability, including the following (Shirlaw et al, 2005):

1. Underwater excavation
2. Lime piles
3. Deep soil mixing
4. Formation of buried slabs using jet grouting

Shirlaw et al conclude that for excavations in excess of about 6m in depth basal stability becomes an issue if the walls are not taken to hard strata. They highlight a number of significant failures over the last 10 years or so, and list the following four main causes of failures:

1. Inadequate understanding of the geological and hydrological conditions
2. Poor design and construction details and poor standard of workmanship, particularly in respect of support systems
3. Construction operations and sequences that differ from those envisaged in the design
4. Inadequate control of site operations, including inadvertent presence of excessive surcharge

The Nicoll Highway collapse in April 2004 is referred to by Shirlaw et al following the issue of the Final Report of the Committee of Inquiry. There are many lessons that can be learned from this collapse. An important issue was the use of numerical analysis for design of deep excavations, what type of soil model is appropriate, and how the numerical analysis should be incorporated into the final design. Shirlaw et al also highlight the need to consider carefully the design and construction implications of jet grouted slabs or other forms of ground treatment.

8 CONCLUDING REMARKS

TC28 has been remarkably active over the period 2001-2005. Two major International Symposia have been organized, one in Toulouse in 2002 and one in Amsterdam in 2005. The key commitment of TC28 has been the collection and exchange of information and experience of all geotechnical aspects of underground construction in soft ground.

This exchange of information and experience - whether it relates to design, construction, research or analysis - is of vital importance to the geotechnical engineering profession as increasing emphasis is being placed by society on improving the environment by construction underground, either by tunnelling or by deep excavations.

PROCEEDINGS OF INTERNATIONAL SYMPOSIA ORGANIZED BY TC28 IN PERIOD 2001-2005

1. ***Proceedings of International Symposium at Toulouse, 2002***: Geotechnical Aspects of Underground Construction in Soft Ground, Kastner, Emeriault, Dias,Guilloux (eds)© 2002 Spécifique, Lyon. ISBN 2-9510416-3-2 (145 papers)

2. ***Pre-print volume of Proceedings of International Symposium at Amsterdam, 2005:*** Geotechnical Aspects of Underground Construction in Soft Ground (122 papers)

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Van der Poel J.T., Gastine E. and Kaalberg F.J. 2005. Monitoring for construction of the North-South Metro line in Amsterdam. Pre-print volume of Proceedings of International Symposium at Amsterdam, 2005: Geotechnical Aspects of Underground Construction in Soft Ground

Viggiani G. and Standing J.R. 2001. The Treasury. Ch. 26 of Building response to tunnelling. Case studies from the Jubilee Line Extension, London, Vol.2, Case Studies. Burland J.B., Standing J.R. and Jardine, F.M. eds (CIRIA Special Publication 200, CIRIA and Thomas Telford), pp 401- 432