K.J. Bakker, A. Bezuijen, W. Broere & E.A. Kwast, editors

GEOTECHNICAL ASPECTS OF UNDERGROUND CONSTRUCTION IN SOFT GROUND

GEOTECHNICAL ASPECTS OF UNDERGROUND CONSTRUCTION IN SOFT GROUND

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Geotechnical Aspects of Underground Construction in Soft Ground

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Preface

In 1989, the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE) established a committee known as Technical Committee 28 (TC28) "Underground Construction in Soft Ground", with the main purpose to provide a forum for interchange of ideas and discussion amongst representatives from many countries with an active interest in tunnelling and deep excavations in the urban environment.

In 1994, TC28, chaired by Professor K. Fujita, organized its first symposium, which coincided with the ISSMGE's International Conference in New Delhi. Professor R.J. Mair then succeeded Professor Fujita as Chairman of TC28, and three day symposia were subsequently held in London (1996), Tokyo (1999) and Toulouse (2002). This volume contains the Proceedings of the fifth symposium of the series, held in Amsterdam in June 2005.

The terms of reference for Technical Committee 28 are: to provide a data source of information about the design, construction and analysis of deep excavations and tunnels, with particular emphasis on the development, effects and control of ground movements; to encourage the production of national reports on the design and monitoring of deep excavations and tunnels by ISSMGE Member Societies; and to disseminate and discuss these reports at international symposia on Geotechnical Aspects of Underground Construction in Soft Ground.

In agreement with these terms of reference, the following themes were established:

- Tunnelling in soft ground
- Mitigating measures
- Numerical analysis of tunnels and deep excavations
- Monitoring of underground constructions
- Deep excavations

For practical purposes, the first theme was split into two presentation sessions: Design Methods for Tunnels, Influences on Foundations; and The Construction of Bored Tunnels.

The first two days of the Symposium were dedicated to technical discussion sessions, with presentations of general reports and individual papers. The opening lecture was given by Professor Arnold Verruijt, and two invited special lectures were given. The first, by Professor Robert Mair, was an overview of the key technical advances achieved during the first ten years of TC28; and the second, on Deep Excavations in the Singapore area, was presented by Mr. Nick Shirlaw.

The third day of the Symposium was dedicated to technical visits. Field trips were organised to the Groene Hart Tunnel, to the construction site of the new North/South Metro Line in Amsterdam, and to the Randstad Rail site in Rotterdam, all of which were under construction at the time.

This volume of the Proceedings contains 131 papers, including the general reports, special lectures and notes on the discussions in the technical sessions. All the papers have been reviewed both by the Scientific Committee and by members of TC28. The collection of papers and reports published in this volume provides a major source of reference on underground construction in soft ground.

The success of the Symposium, must be attributed to the authors of papers, the speakers at the Symposium and the delegates that came to Amsterdam to present their work, either orally or in the poster session as well as participating in the discussions.

Special thanks go to the Dutch Public Works Department and to the companies, both from The Netherlands and from abroad, that were willing to give financial support to the organization of this Symposium; a list of the sponsors is included in this proceedings.

K.J. Bakker & R.J Mair
Chairmen of the Symposium

A. Bezuijen, W. Broere & E.A. Kwast
Editors

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The Symposium of Technical Committee 28 on Underground Construction in Soft Ground was organized by the Royal Institution of Engineers in The Netherlands KIVI/Niria under the auspices of the International Society for Soil Mechanics and Geotechnical Engineering.

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Special lectures

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TC28 – tunnelling: reflections on advances over 10 years

R.J. Mair
Cambridge University, UK

ABSTRACT: Technical Committee TC28 on Underground Construction in Soft Ground is one of the most active technical committees of the International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE). Significant technical advances have been made over the 10 year period 1996–2005, and TC28 has been particularly active through its organization of four major two-day International Symposia: in London (1996), Tokyo (1999), Toulouse (2002) and in Amsterdam (2005). This Lecture is a personal reflection on some of the more significant technical advances in recent years arising directly from the work of TC28. The Lecture focuses on a number of selected technical advances relating to the following areas concerned with bored tunnels: construction and settlement control, the effects of tunnelling beneath piled foundations, modelling and prediction (centrifuge modelling and numerical analyses), and monitoring. Examples from research and practice are presented.

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 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 pre-print volume of this Amsterdam Symposium contains 122 papers, together with General Reports for each of the technical sessions; we look forward with anticipation to the final Proceedings being published.

In each of the four International Symposia 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.

This Lecture is a personal reflection of the author on some of the more significant technical advances over the 10 year period 1996–2005 arising directly from the work of TC28. In view of constraints of space, only a few subjects have been selected. These few subjects inevitably do not fully reflect the very wide range of topics and areas of interest covered in the four International Symposia, but they have been selected as being of particular significance, in the author's personal view.

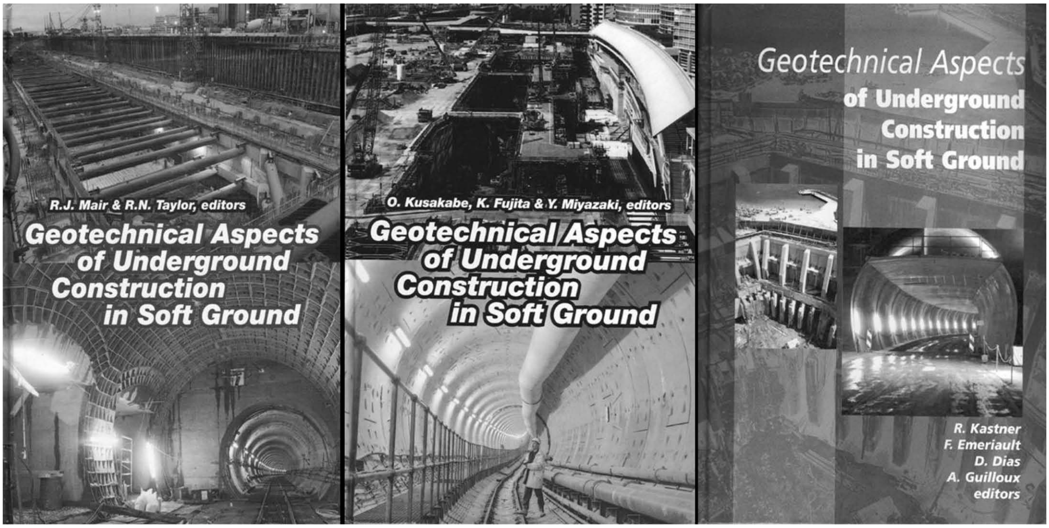
In keeping with its Terms of Reference, the principal areas of interest of TC28 have been as follows:

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.

2 SELECTED TECHNICAL ADVANCES

The selected technical advances that are addressed in this Lecture 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



London (1996) Tokyo (1999) Toulouse (2002)

Figure 1. Proceedings of previous TC28 Symposia.

Most of the examples selected for this Lecture are featured or are referred to in this TC28 Amsterdam Symposium.

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. There have been considerable advances in soil conditioning technologies in recent years and a wide variety of foams and polymers are now available for use with EPB machines.

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 8.15 m external diameter were constructed over a length of 18 km 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 7.5 km. The following points are noteworthy:

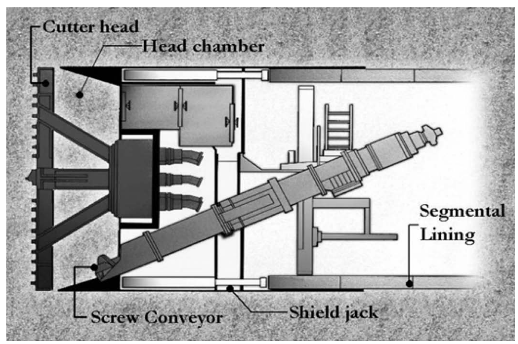


Figure 2. Schematic of typical EPB tunnelling machine.

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;

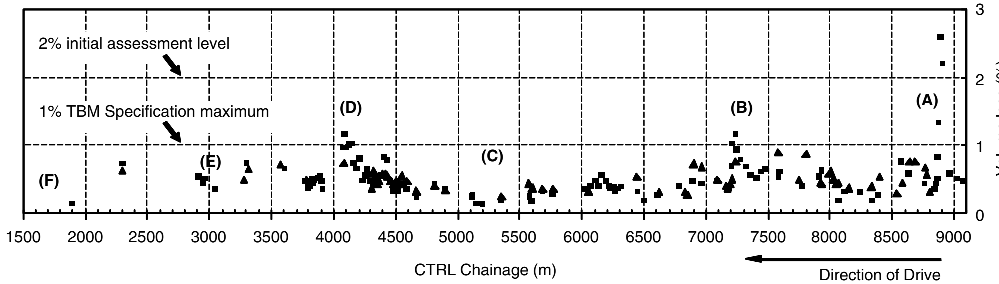


Figure 3. Volume losses for tunnelling on CTRL project in London (Bowers and Moss, 2005).

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. The importance of operator skill for successful EPB tunnelling, and in particular for the avoidance of sinkholes, is emphasized by Shirlaw et al (2003).

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. 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

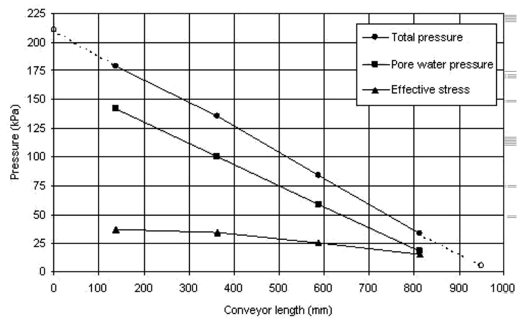


Figure 4. Pressure changes along model EPB screw conveyor (Merritt et al, 2003).

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

3.3 *Tail void grouting to control settlement*

Successful control of settlement frequently depends on effective tail void grouting, depicted in Figure 5.

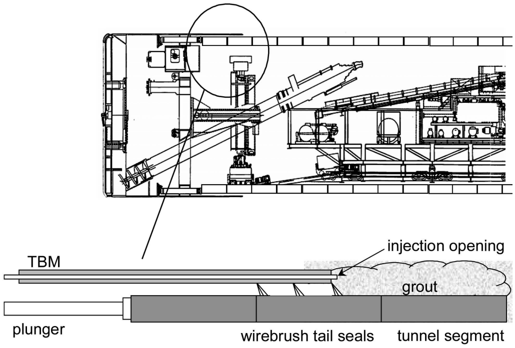


Figure 5. Tail void grouting (Bezuijen and Talmon, 2005).

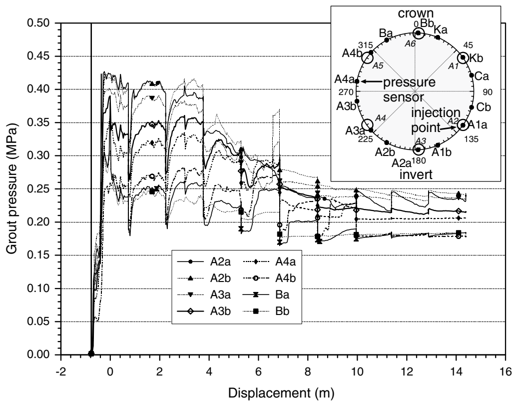


Figure 6. Measured grout pressures acting on tunnel lining segment (Bezuijen and Talmon, 2005).

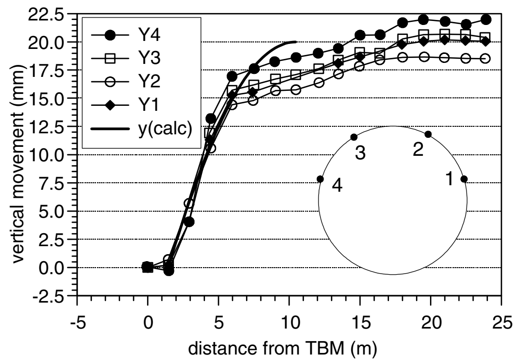


Figure 7. Vertical movement of tunnel lining segment immediately after erection (Bezuijen and Talmon, 2005).

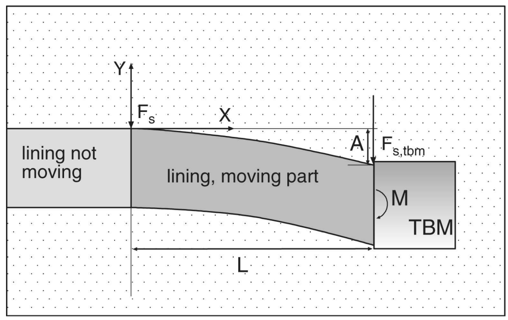


Figure 8. Idealised behaviour of tunnel lining showing flotation in fluid grout (Bezuijen and Talmon, 2005).

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 9.5 m 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 4 m) leads to an overall reduction in measured pressure, and after the TBM has progressed about 6 m the grout has hardened and the pressure becomes constant (and comparable to the pore water pressure).

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.

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

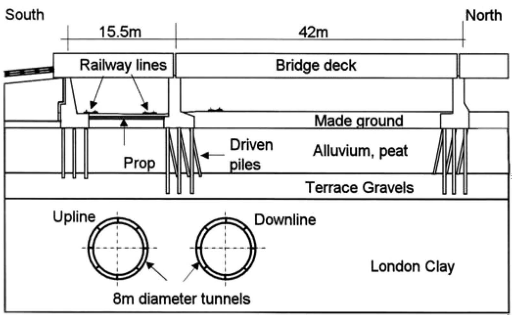


Figure 9. Case history of tunnelling beneath piles on CTRL project in London (Jacobsz et al, 2005).

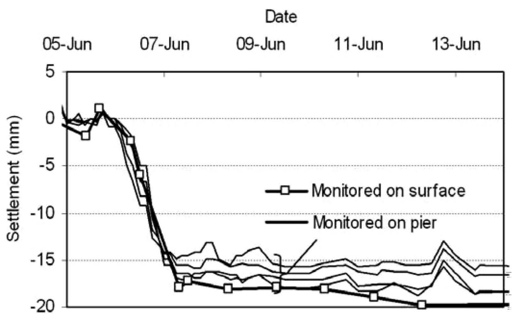


Figure 10. Observed settlement of piled bridge pier and ground surface on CTRL project in London (Jacobsz et al, 2005).

they are heavily loaded and primarily end-bearing. Figure 9 illustrates an example of a recent case in which 8 m 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 8–12 m of very soft alluvial silts, clays and organic soils, and hence the piles possessed negligible shaft capacity.

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 18 mm, increasing to 20 mm 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.

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

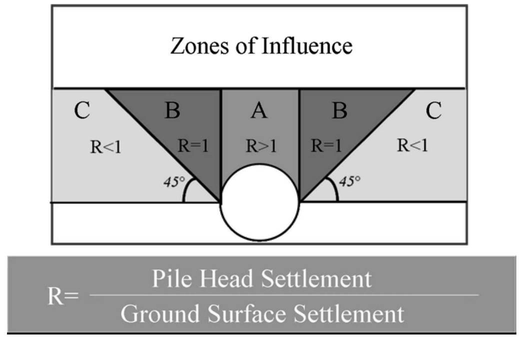


Figure 11. Zones of influence relating to pile toe locations relative to tunnel position (Selemetas et al, 2005).

zone B it will be very similar, and in zone C it will be less.

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 8.3 m 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.

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

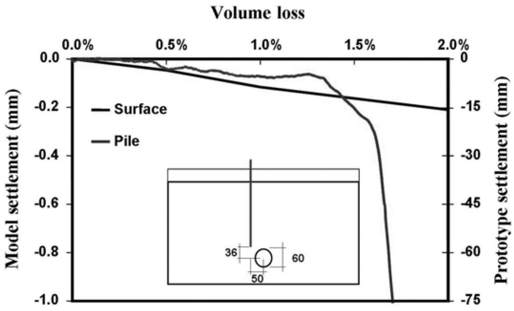


Figure 12. Effects of tunnelling near piled foundation – Cambridge centrifuge model tests (Jacobsz et al, 2004).

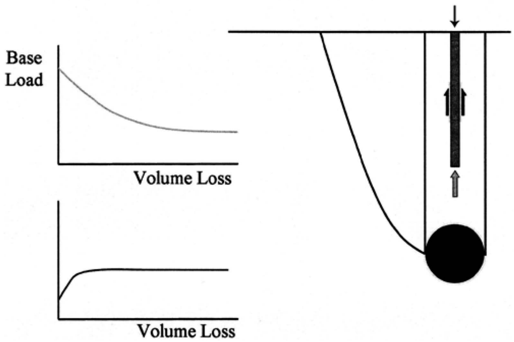


Figure 13. Mechanisms of pile load distribution changes for pile close to and above the tunnel.

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.

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

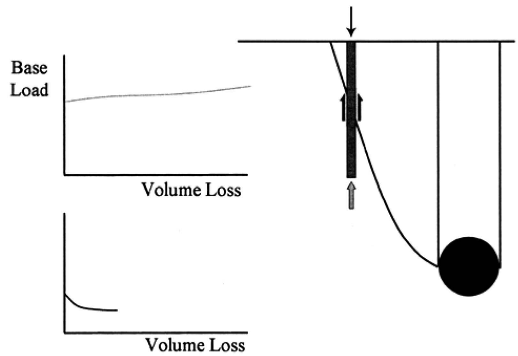


Figure 14. Mechanisms of pile load distribution changes for pile further away from the tunnel.

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 300 mm thick wall of 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. Viggiani et al (2005) describe an interesting example of sophisticated FE analysis of the effects of tunnelling at a skewed angle beneath a masonry wall. They too found that the stiffness of the wall affected not only the magnitude of the computed displacements but also (and more importantly) the pattern of the displacement field and associated deformation.

6 MONITORING

The importance of monitoring is paramount, especially in complex cases of soil-structure interaction where

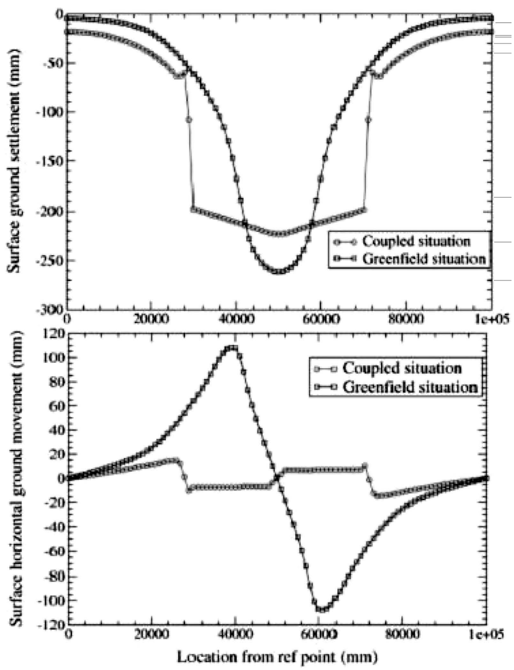


Figure 15. Influence of building stiffness on response to tunnelling-induced ground movements (Boonpichetvong et al, 2005).

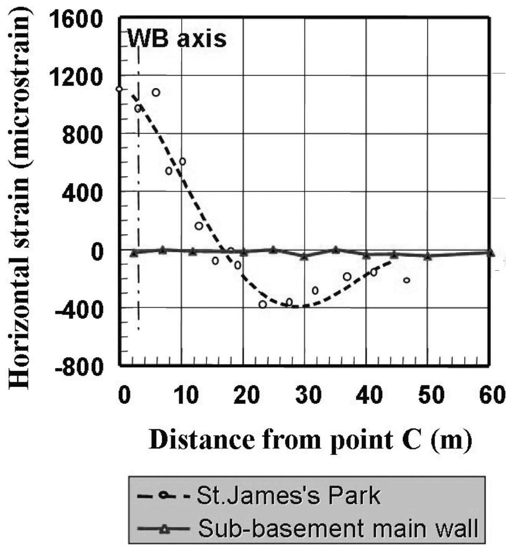


Figure 16. Comparison of building and greenfield response (horizontal strain) to tunnelling (Viggiani and Standing, 2001).

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

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.

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.

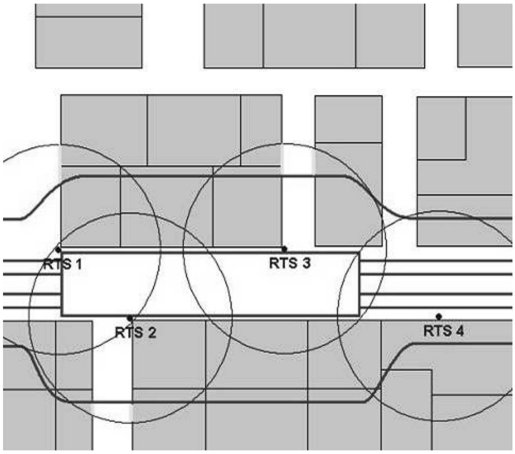


Figure 18. Coverage of total station monitoring for an underground station under construction in Amsterdam (Van der Poel et al, 2005).

7 CONCLUDING REMARKS

TC28 has been remarkably active over the 10 year period 1996–2005. Four major International Symposia have been organized: in London in 1996, Tokyo in 1999, in Toulouse in 2002 and now this 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 1996–2005

1. ***Proceedings of International Symposium at London, 1996*:** Geotechnical Aspects of Underground Construction in Soft Ground, Mair and Taylor (eds), © 1996 Balkema, Rotterdam, ISBN 90 54 10 856 8
2. ***Proceedings of International Symposium at Tokyo, 1999*:** Kusakabe, Fujita & Miyazaki (eds) © 2000 Balkema, Rotterdam, ISBN 90 5809 1 066
3. ***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)
4. ***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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Deep excavations in singapore marine clay

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ABSTRACT: Significant areas of the financial and commercial districts of Singapore have been constructed over deep deposits of near normally consolidated marine clay. Numerous deep excavations have been required for urban development and the supporting infrastructure. The design of these excavations needed to consider basal stability and (for walls taken to hard strata) net active forces below excavation level. A number of innovative solutions to these problems have been applied. Solutions have included the use of underwater excavation, lime piles, deep soil mixing and the formation of buried slabs using jet grouting. Generally, these have been successful. However, there have been a number of major excavation failures over the last 10 years. Examples of successful excavations are contrasted with some of the failures, and the causes discussed.

1 INTRODUCTION

Deep deposits of normally or near normally consolidated clay pose particular problems in the construction of deep excavations and tunnels. A significant proportion of the older urban areas of Singapore are built over deposits of soft marine clay. In particular the old Chinatown, Little India and Arab Street areas are built over deposits that extend typically to 20 m to 35 m below ground level. Reclamation south of the old Beach Road, carried out over the last 50 years, has resulted in areas for new development where the marine clay can extend to 45 m or more below ground level. The clay can still be consolidating under the weight of the reclamation fill twenty or more years after reclamation. The marine clay is the main constituent of the Kallang Formation.

Figure 1 shows the areas of Singapore where the Kallang Formation is encountered, and the areas of post war reclamation, which are underlain by soils of the Kallang Formation.

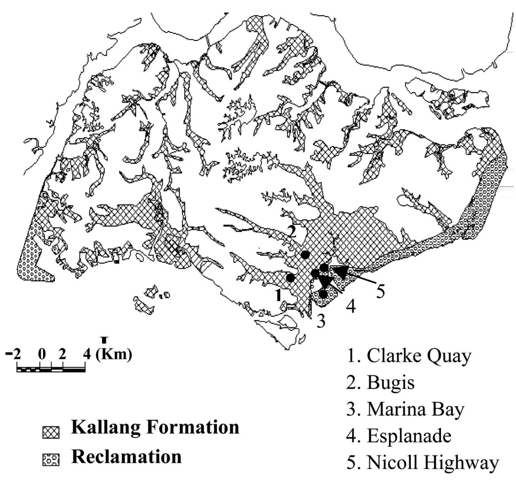

Figure 1. Singapore, showing areas of Kallang Formation deposits and recent reclamations.

The rapid development of Singapore over the last 40 years has required the construction of many deep excavations, for mass transit, underground roads and commercial and residential space. Many of these excavations have been in areas of deep marine clay. In the older urban areas, the large inward movement of the excavation support walls and the consequent settlement of the traditional Singapore ‘shophouses’, founded on shallow foundations or short timber piles, have been of particular concern. In the newly reclaimed areas the depth and very low strength of the clay have posed particular problems.

Some of the innovative solutions that have been successfully adopted in the construction of deep excavations in the marine clay will be described, using brief case studies of completed excavations. In addition to the many successful excavations, there have been a number of failures of excavations, and the details of some of these will also be given.

2 THE MARINE CLAY

The marine clay is a unit of the Kallang formation. The Kallang Formation is a recent deposit consisting of soil of marine, alluvial, littoral and estuarine origins and covers about 25% of Singapore Island (PWD, 1976). Marine clay is the main constituent of this formation. In some instances, particularly under reclaimed areas, it can be over 40 m in thickness. The marine clay includes a Holocene deposit, referred to as the Upper Marine Clay (UMC), and a Pleistocene deposit, known as the Lower Marine Clay (LMC). The two beds are typically separated by a stiffer intermediate layer, considered to be the desiccated crust of the lower marine clay (Tan et al., 2002). The clay fraction of the marine clays is usually high, typically more than 50%. The principal mineral is kaolinite, although at some locations smectite is also prominent.

A study on Singapore Marine Clay by Tan et al. (2002), was based on samples from two sites. One, on the main island, was for the Singapore Arts Centre (SAC), now known as the Esplanade. The other was from the sea channel off the eastern coast (PT clay). The SAC site is within the central business district of Singapore, close to many of the deep excavations that have been carried out for urban development. The PT clay is in an area well away from urban development. As there are significant differences between the clay in the two areas, this paper will use only the data from the SAC site.

The liquid limit of the clay at the SAC site was 60%–80%. The activity of SAC clay is ∼0.8 which is only slightly more active than Norwegian Drammen clay, a well-known lean clay.

Using high quality samples (Tan et al., 2002) and constant strain rate test, the compression behaviour shows the presence of high compressibility immediately after the preconsolidation pressure (p'_c) before joining the normally consolidated line, an indication of some microstructure. This behaviour after p'_c can be described using two compressibility indices, namely C_{C1} the compression index immediately after p'_c and C_{C2} the usual value along the normally consolidated line well after p'_c. At the SAC site C_{c1} ranged from 0.4 to 1.8. The value of C_{c1}/C_{c2} was about 1.2 for standard oedometer tests and about 1.7 for constant strain rate tests.

The clay was found to be slightly over-consolidated, with OCR averaging 1.2 from oedometer test and 1.5 from CRS test.

The undrained shear strength ratio (c_u/σ'_{vo}) measured using UCT (Unconfined Compression) tests was about 0.22 (Figure 1). Samples tested after isotropic or Ko consolidation gave a higher ratio, of about 0.3, as did tests in extension. Data from Direct Simple Shear (DSS) tests, reported by Cao et al., gave a lower strength ratio of 0.18 (for the UMC) MC) and 0.23 (for the LMC). Typically, vane tests give an undrained shear strength ratio of about 0.25, in terms of current effective stress. This ratio takes into account the small degree of overconsolidation found in areas other than the more recent reclamations. Tan et al report that the ratio between the undrained shear strength and the maximum past effective consolidation pressure (p'_c) is 0.21, based on both DSS and field vane tests. The design of excavations is commonly based on the results of field vane tests.

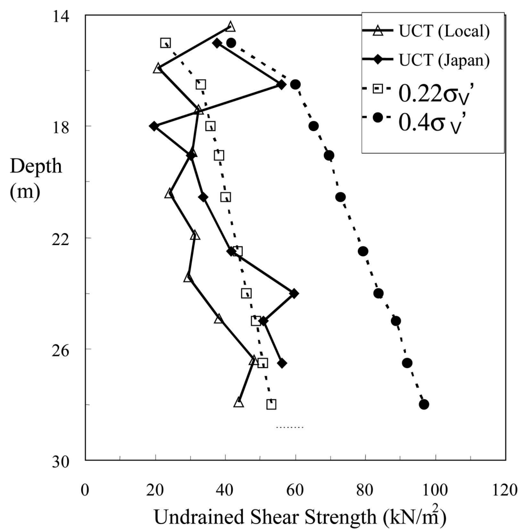

Figure 2. Effect of sampling method on undrained shear strength (UCT) for SAC clay.

Significant areas of Singapore have been formed by reclamation, often over deep deposits of marine clay. Although the SAC site studied by Tan et al. was reclaimed from the sea, both the thickness of fill placed and the thickness of the marine clay were lower than in some other areas. In areas of thick marine clay it can take 25 years or more for the clay to consolidate under the weight of the fill; many of the more recent reclamations still exhibit positive excess pore pressures in the clay. Due to the low effective stress in the clay, the strength can be significantly lower, at a given depth, than the values measured at the SAC site. A number of shear strength/depth plots used for design at various sites are summarized on Figure 3. Bugis and Clarke Quay Stations were constructed in old urban areas. The Esplanade and Marina Bay Station were built in reclamations that had been completed 23 and 10 years before excavation, respectively. A section

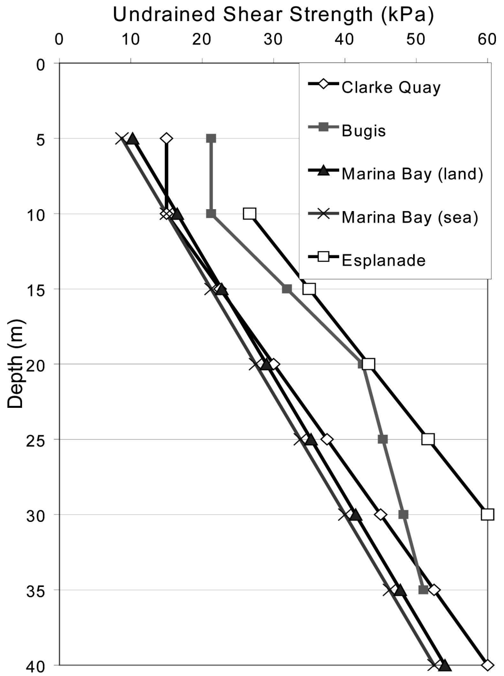

Figure 3. Undrained shear strength/depth profiles in the marine clay at various sites in Singapore, as used for design.

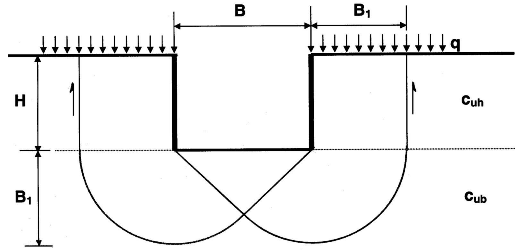

Figure 4a. Terzaghi's method (1943).

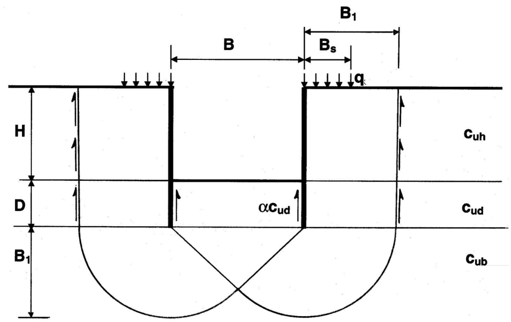

Figure 4b. Modified Terzaghi's method (Wong and Goh, 2002).

of the Marina Bay tunnels was built, by cut-and-cover methods, through a sea inlet (the Telok Ayer basin) that has subsequently been reclaimed.

The design shear strength profiles in Figure 3 are after the application of vane correction factors of 0.85 (for Bugis), 0.81 (for the Esplanade), and 1 at the other sites.

3 ISSUES FOR EXCAVATIONS

3.1 *Basal stability – undrained failure, hydraulic uplift*

For excavations into soft clay, where the soft clay extends to below the base of the excavation, there are two issues related to basal stability that have to be addressed. These are undrained basal heave and hydraulic uplift on the remaining clay below the base of the excavation.

3.1.1 *Undrained basal heave*

Where the clay below the base of the excavation has a low shear strength, and the walls do not extend to stiffer strata, then there is the potential for undrained base heave to occur (Figure 4).

There are many published methods for the assessment of basal heave stability of excavations in clay. Wong & Goh (2002) reviewed the methods proposed by Terzaghi (1943), Bjerrum & Eide (1956) and Goh (1994) and compared them with results from finite element analysis using Sage Crisp. Results indicated that Terzaghi's method would yield reasonable factors of safety for excavations involving flexible sheet-pile walls in clay but conservative results for stiff diaphragm walls. They have proposed an extension of Terzaghi's method to take into consideration the effect of wall stiffness. The modified method was validated against the finite element results.

Terzaghi's method (1943) catered for wide excavations with the width (B) greater than the depth (H). The failure mechanism is shown in Figure 4a. The failure surface extends from the ground surface to a depth of 0.7B below the formation level or to the top of the underlying hard stratum whichever is shallower. Wall penetration beyond the formation level is ignored. The effect of wall stiffness is not considered. Even though it is developed under plane strain condition, it is applicable to wide excavations of rectangular shape. The factor of safety F_s can be computed using Equation (1).

$$F_S = \frac{5.7\, c_{ub} B_1}{\gamma H B_1 - c_{uh} H} \qquad (1)$$

where c_{uh} is the average undrained shear strength above the formation level; c_{ub} is the average undrained shear strength within the failure zone below the formation level; γ is the average bulk unit weight of the soil above the formation level; H is the maximum excavation depth; and B_1 is equal to 0.7B or depth to hard stratum below formation level (T) whichever is smaller.

This equation is expressed in terms of soil resistance over the net driving force. An alternate approach (Chang, 2000) is to keep all resisting forces in the nominator and all driving forces in the denominator as shown in Equation (2).

$$F_S = \frac{5.7\, c_{ub} B_1 + c_{uh} H}{\gamma H B_1} \qquad (2)$$

Results of the finite element analysis are in excellent agreement with those obtained from Eq. 1 and 2. Eq. 2 has no noticeable improvement over the original formulation (Eq. 1). Bjerrum & Eide (1956) pointed out that Terzhagi's method is reliable for shallow excavations ($B \geq H$) in a homogenous soil. The results may be unreliable when the surface clay has a stiff, desiccated, crust or when the excavation is narrow (i.e. $B < H$). Mana & Clough (1981) reported cases where excavations were successfully completed even when the calculated factor of safety was below unity. These field observations imply that Terzaghi's method may yield conservative results in some situations.

When a very stiff wall is used, the clay has to flow around the toe into the excavated area. Wong & Goh (2002) extended Terzaghi's method to accommodate this phenomenon as shown in Figure 4. The extension is based on Equation (2) where all the resisting forces are on the nominator and the driving forces at the denominator.

$$F_S = \frac{5.7\, c_{ub} B_1 + c_{uh} H + (1+\alpha)\, c_{ud} D}{\gamma H B_1 + q\, B_s} \qquad (3)$$

where α is the adhesion factor; $B_1 = 0.7B$ or (T-D) whichever is smaller; T is clay thickness below formation level; and B_s is the width of surcharge loading where $B_s \leq B_1$. Two examples were used to verify the reliability of this method. The computed factors of safety for are summarized in Table 1. The computed F_s values using the modified method are in excellent agreement with those obtained from the finite element analyses. However, this method is only valid if the wall has enough strength to resist the potentially large forces imposed on it.

Due to the low undrained shear strength of the marine clay in Singapore, excavations where the retaining walls do not penetrate below the base of the excavation would typically fail by base heave at a total excavation depth of about 6 m, based on the shear strength typically found in the older urban areas.

Table 1. Factors of safety for 'rigid' walls in clay.

Case	Terzaghi Eq. 1	Modified Terzaghi Eq. 3
1	0.81	0.96
2	0.76	0.95

3.1.2 *Hydraulic basal stability*

The marine clay is typically underlain by fluvial sand (Bird et al., 2003), and then by Old Alluvium (Chiam et al., 2003) or the weathered rocks of the Jurong or Bukit Timah Granite Formations. These materials can be significantly more permeable than the overlying marine clay, so consideration has to be given to the potential for hydraulic uplift. This will occur if the water pressure in the more permeable layer is higher than the weight of the remaining soil in the passive zone of the excavation. Pressure relief wells have been used in Singapore to control this problem.

3.2 *The 'net active' pressure problem*

One way of avoiding undrained basal instability is to lengthen the supporting walls for the excavation, providing adequate penetration into a competent bearing stratum. However, for excavations in areas of deep soft clays, the wall deflection and induced bending moment can be exceptionally high. This is due to the 'net active' pressure problem, as described in Davies & Walsh (1984). 'Net active' pressure occurs when the active pressure on the wall exceeds the limiting passive pressure in the zone below the base of the excavation. This occurs once the stability number at the base of the excavation exceeds the critical stability number (Karlsrud 1986).

The undrained shear strength/depth profile for Clarke Quay (Fig. 3) is typical of those commonly used for design in Singapore, in areas other than those that have been recently reclaimed. 'Recent' in this context refers to areas that have been reclaimed in the last 25 years; due to the thickness and low permeability of the clay, primary consolidation generally takes 25 years or more to take place.

Figure 5 shows the calculated net active pressure (the active pressure minus the limiting passive pressure) below the base of a long, 18 m deep excavation, for the Clarke Quay strength/depth profile, assuming marine clay to a depth of 40 m. The calculation is based on:

$$\text{Net Active Pressure} = \gamma H + q - N_{cb}\, c_u \qquad (4)$$

Where q is the design surcharge and the other terms are defined above. It can be seen that there is a net active pressure on the wall almost throughout the marine clay, even below the base of the excavation. In the passive zone the wall has to transfer this load, upwards

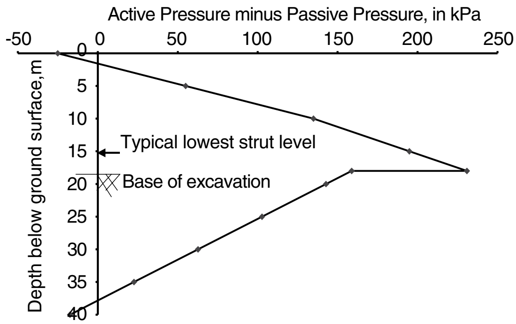

Figure 5. The 'net active' pressure on the wall of an 18 m deep excavation. The undrained strength profile used is that for Clarke Quay in Figure 3.

to the struts and downwards into the underlying hard stratum. In the example given the wall is effectively unsupported between the lowest strut level and the hard stratum, a span of 25 m. It is this large unsupported span that leads to the large deflections that are discussed below.

3.3 *Underdrainage during excavation*

Deep marine clays in Singapore are typically underlain by fluvial sands, and then by the Old Alluvium or the weathered sedimentary rocks of the Jurong Formation. Typically, these underlying units have a bulk permeability that is two to four orders of magnitude higher than that of the marine clay.

Stress relief during excavation results in large negative excess pore pressures developing in the marine clay in the passive zone. These reduced pore pressures are then transmitted outside the excavation through the more permeable underlying units. The confined nature of these aquifers and the high horizontal permeability of the fluvial sands can transmit the reduction in pore pressure for hundreds of metres from the excavation. A reduction in pore pressures in the fluvial sands outside the excavation is followed by underdrainage of the overlying marine clay, and resulting consolidation settlements. The process is driven by stress relief, not by active groundwater control, and can continue for months after the base slab has been completed. Shirlaw & Wen (1999) and Wen & Lin (2002) provide examples of this problem.

4 PRACTICAL IMPLICATIONS FOR EXCAVATIONS IN SINGAPORE

As discussed in section 2, the marine clay is generally, apart from the PT area, close to normally consolidated, or, in areas of post war reclamation, still consolidating. The shear strength is low, and consequently the factor of safety against basal heave, even for relatively shallow excavations, is low. Net active pressures develop for even quite shallow excavations. Very large lateral movements have been experienced with sheetpiled excavations. In Table 2, a number of cases of sheetpile supported excavations are summarised. The excavations listed were inland from the current coastline, and not in the more difficult recent reclamation areas, where the clay is weaker (as it is still consolidating) and thicker than at these sites. The depth to the base of the marine clay at these four sites was 14.7 to 24 m. It can be seen that, despite the relatively shallow excavations, the wall deflections were large in all four cases.

Table 2. Sheetpile wall deflections at four sites in Singapore.

Name	Exc. Depth (m)	Wall type	Levels of struts	Max. Wall Deflection (mm)
MOE	7	YSP IV	4	315
Rochor Complex	6.3	FSP IIIA	3	150
CTC Building	11	FSP IIIA	6	230
Novena Station	15.5	FSP IV	6	270

The maximum deflection of the wall at these four sites occurred below the base of the excavation. This shows that the large deflections measured were influenced by the 'net active' pressure that develops once the excavations exceed about 6 m in depth.

As discussed above, excavation induced stress relief results in large pore pressure changes, which in turn results in underdrainage of the marine clay outside the excavation. Consolidation settlements of 100 mm or more have been recorded during the construction of Newton Station (Nicholson 1987), Bugis Station (Shirlaw & Wen, 1999) and Farrer Park cut-and-cover tunnels (Wen & Lin, 2002).

The rapid development of Singapore has led to the need for some very deep excavations in the marine clay, often in areas where movement has to be controlled to low levels, due to adjacent structures. This has resulted in some innovative excavation systems being used. Most of these have been very successful, but, unfortunately, there have also been a number of failures. The following sections describe, first, some of the successes, and then some of the failures.

5 INNOVATIVE EXCAVATION SYSTEMS

5.1 *Diaphragm walls and lime columns (Bugis)*

Bugis Station was constructed between 1986 and 1989, as part of Phase 2 of the subway system in Singapore. The station excavation was 18 m deep. The depth from ground surface to the base of the marine clay varied along the station from 17 m to 37 m. Underlying the marine clay was a thin bed of fluvial deposits and then the Old Alluvium (Fig. 6).

The excavation was carried out in an old urban area of Singapore, near Bugis Steet. It was specified that diaphragm walls should be used. 1.0 and 1.2 m thick walls were constructed, up to 54 m in depth, depending on the thickness of the marine clay. Up to 7 levels of strutting were required to support the walls for the 18 m deep excavation. In addition, lime columns were used to improve the soil at and immediately below final excavation level (Kado et al., 1987). Although the lime columns increased the average strength of the soil at formation level, by 60% to 100%, the installation of the columns, using a mandrel, caused some reduction in the strength of the clay above the treated area.

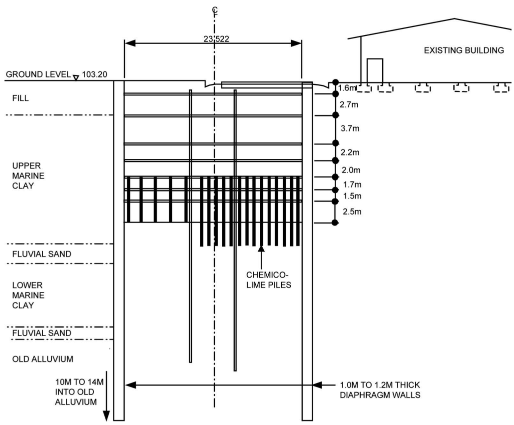

Figure 6. The excavation support system for Bugis Station.

The lateral movement of the diaphragm wall when the excavation was at final excavation level was 150 mm, in the section where the marine clay was thickest. Total settlement was up to 250 mm, of which about 100 mm was consolidation settlement.

The excavation resulted in up to 8.2 m of pore pressure reduction in the Old Alluvium outside the excavation, which caused underdrainage of the marine clay. Despite the installation of 11 recharge wells, the consolidation settlement was still continuing 9 months after the completion of the base slab. No pressure relief wells or deep pumping were carried out during the excavation.

5.2 *Underwater excavation (Marina Bay)*

Marina Bay Station was built between 1986 and 1989 in an area that had been reclaimed from the sea about 10 years before. The tunnels connecting the station to Raffles Place station were built across the Telok Ayer basin (Fig. 7), a tidal basin connected to the sea. The station and tunnels across the basin were constructed using underwater excavation (Clarke & Prebaharan, 1987). A substantial retaining wall consisting of 610 × 305 mm × 149 kg I sections welded to BXN sheetpiles,

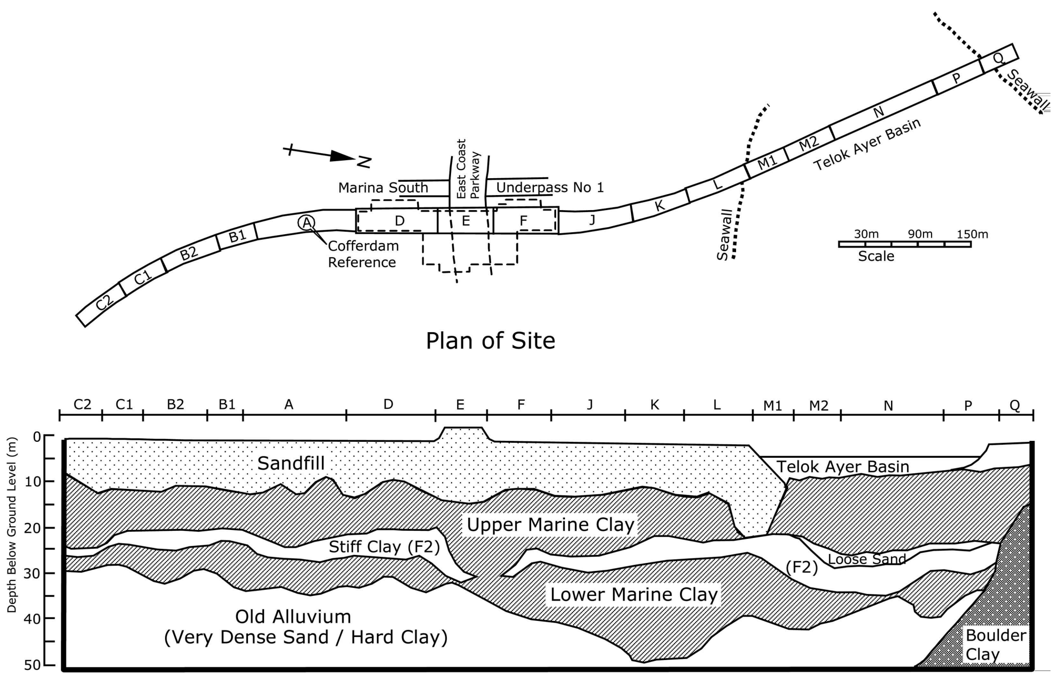

Figure 7. Marina Bay Station and tunnels (after Clarke & Prebaharan, 1987).

was installed. The walls were not taken right through the marine clay, but were stopped at a depth of 23 m to 30 m below ground level, in the stiff clay layer intermediate between the UMC and the LMC.

The excavation sequence is shown in Figure 8. Excavation to a depth of 6 m was carried out in the dry, and two levels of struts were installed. The rest of the excavation, to a depth of about 19 m, was carried out underwater. The water in the cofferdam was maintained at 1 m above ground level in the reclamation areas. The total weight of water within the cofferdam at final excavation level was calculated to be slightly greater than the total weight of the soil excavated after the initial, dry, excavation stage. As a result the deflection of the wall, 100 to 200 mm at the end of the dry excavation stage, hardly changed during the underwater excavation.

The permanent bored piles, required to support the station, were then installed. The piles obtained bearing in the Old Alluvium, and were designed for both compression and tension. A 1.5 m thick mass concrete slab was then tremied into the base of the excavation. Shear connectors were provided between the permanent steel casings of the piles and the tremie slab. The tremie slab was designed to provide lateral restraint to the walls and to resist base heave pressure, tied down by the permanent piles, following dewatering.

The base heave pressure was calculated using the formula:

$$P = q + \gamma H - F_1 N c_U - \frac{F_2 \pi d(h_1 + h_2 + h_3) c_U}{LB} \qquad (5)$$

Where:
q is the surcharge
γ is the average bulk density of the soil above the level of the base of the excavation
F_1 and F_2 are strain compatibility factors
N is the critical stability number for base heave (Bjerrum & Eide, 1956)
c_U is the undrained shear strength of the clay
h, h_1, h_2 and h_3 are as defined in Figure 9, and depend on the critical failure surface for the particular ground conditions and excavation geometry at the site
L is the spacing of the piles along the excavation
B the excavation width.

The tremie slab was unreinforced, but had to be able to withstand bending, due to the uplift force, between the piles. The bending capacity of the slab was derived from the compressive force exerted on the slab by the retaining walls.

Four pressure cells were installed below the base of the excavation. The calculated and measured total uplift pressure on the centre of the slab is given in Table 3.

In all fifteen, linked, rectangular cells, 11 in the reclamation and 4 in the Telok Ayer basin, were successfully constructed in this manner.

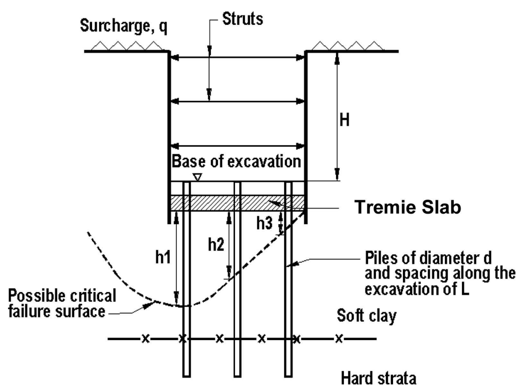

Figure 9. Basis for the estimation of uplift pressures on the tremie slab, Marina Bay Station.

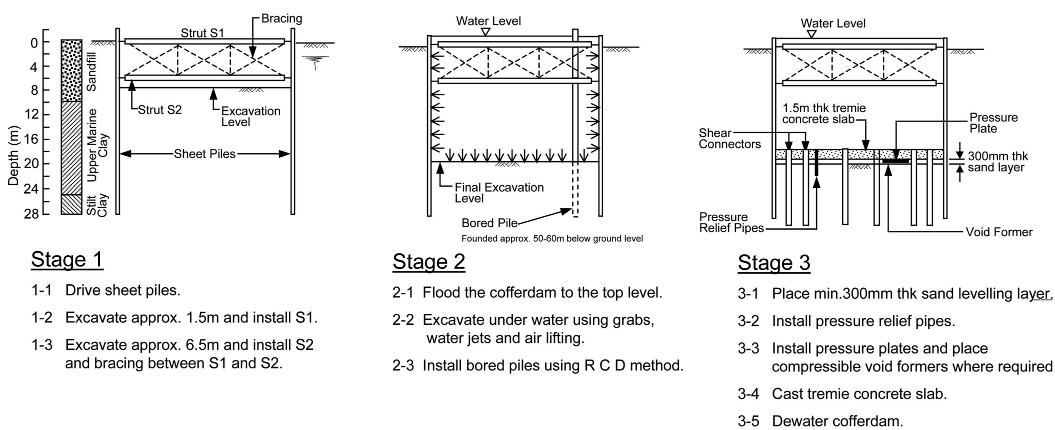

Figure 8. Excavation sequence for Marina Bay Station and tunnels.

Table 3. Predicted and measured uplift on the tremie slab, Marina Bay (after Clark & Prebaharan, 1987).

Cofferdam	Predicted uplift (KN/m^2)	Measured uplift (KN/m^2)
A	85	78, 78
F	83	63, 60, 57, 42
J	83	100, 70
M2	72	62, 57

5.3 *Diaphragm walls and jet grout slabs (Little India)*

The cut-and-cover tunnels between Little India and Farrer Park Stations formed part of the North East line of Singapore's MRT system. The total depth to the base of the marine clay was between 18 and 35 m, while the depth of excavation was 17.5 m. The retaining system consisted of 0.8 m thick diaphragm walls with 6 levels of struts.

In order to reduce the deflection of the wall, and therefore the settlement to adjoining properties, a jet grouted slab was formed at the base of the excavation. This was done only in those areas where the adjacent buildings were particularly sensitive to settlement. Sections of slab were either 1.5 m or 2 m in thickness, and the slab spanned between the two diaphragm walls (Fig. 10).

A comparison has been made of the measured deflections with those for other excavations, using diaphragm walls, where no jet grout slab was used (Fig. 11). It can be seen that the slab had little benefit when the hard stratum was close to the base of the excavation. Much greater benefit was obtained, in terms of reduced wall movement, when the hard stratum was deeper.

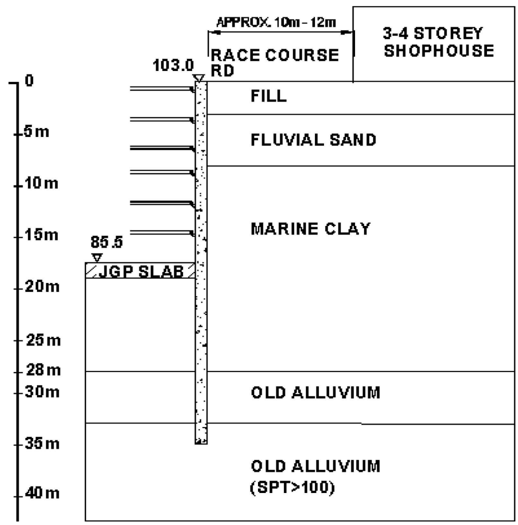

Figure 10. Jet grout slab used as a buried strut, cut-and-cover tunnels along Race Course Road.

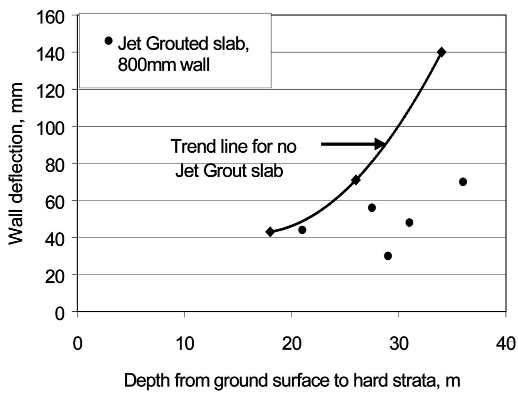

Figure 11. A comparison between the recorded deflections for 17.5 to 18 m deep excavations, with and without a single jet grout slab just below the base of the excavation.

5.4 *Floating sheetpiles and piled jet grout slabs (Clarke Quay, Kallang/Paya Lebar Expressway)*

Another excavation for the North East Line was that for Clarke Quay Station. The marine clay in this area was underlain by weathered rocks of the Jurong Formation; the depth to the weathered rock from ground surface was 20 m to 27 m. The main station box was excavated using strutted diaphragm walls taken into the Jurong Formation. However, two of the entrances were constructed using a support system that combined many of the features of the Marina Bay excavation with the jet grouted slab, as used at the Little India to Farrer Park tunnels.

The jet grouted slab was used in place of the tremie slab, and designed in a similar way. The sheetpiles for the 9 m deep excavation were driven to a depth of 12 m, so the excavation and support system 'floated' in the marine clay. The excavation was carried out in the dry, using 3 levels of strutting as well as the jet grout slab (Fig. 12). The design of the jet grout slab was based on equation 5, previously used of the tremie slab at Marina Bay. Measurements of the heave of the slab during excavation gave a value of only 3 mm. Further details of the design and construction issues for jet grouted slabs are given in Shirlaw et al (2000a), Shirlaw et al. (2000b) and Shirlaw (2003).

Similar 'floating' retaining systems, but for deeper excavations, have since been used for some of the excavations for the Kallang and Paya Lebar Expressway, which commenced in 2002. One example is shown in Figure 13. The excavation was much larger than those

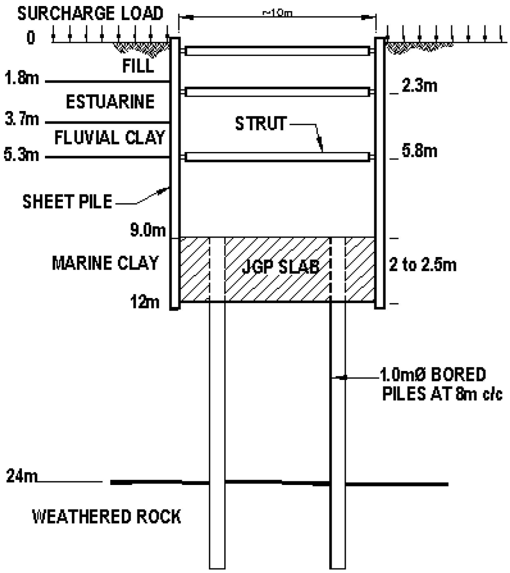

Figure 12. The ‘floating’ retaining system, Entrances 1 and 2, Clarke Quay Station.

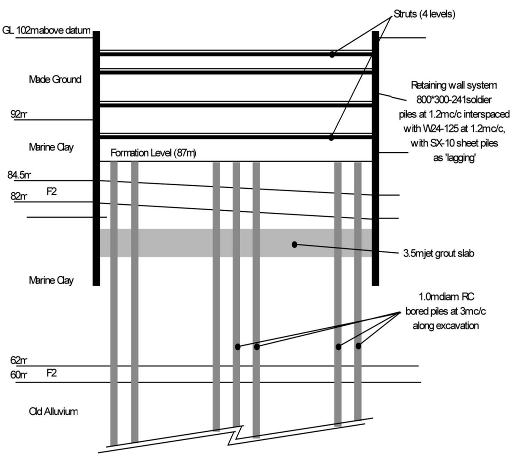

Figure 13. An example of a ‘floating’ wall system, used for part of the KPE construction.

of the entrances at Clarke Quay. The width of the excavation was 40 m and the depth 15 m. In this case the monitored heave on the jet grouted slab, at the end of excavation, was 47 mm. The position of the jet grout slab was chosen to ensure that the grouting was in the marine clay and not in the stiff intermediate (F2) clay. The column diameter that could be achieved in the stiff clay was significantly smaller than that in the marine clay. In order to form a slab by installing multiple, overlapping columns, it was considered advantageous to ensure that the slab was in the marine clay.

Jet grouted slabs have been in use as part of excavation support systems in Singapore for about 20 years. In numerical models the jet grout is generally treated ground as a strong, perfectly elastic/perfectly plastic soil of considerable strength. However, as discussed in Shirlaw (2002a), it is advisable to also assess the jet grout slab as a weak structure. Strain compatibility between the soft clay and the jet grout, and the potential for a brittle ‘post peak’ behaviour of the jet grout also need to be considered. Once the jet grout slab is considered as a weak structure, it is apparent that the interfaces between the jet grout slab and the walls and piles are critical to the effectiveness of the slab. These interfaces are, in turn, dependent on the layout and size adopted for the jet grout columns. The designer of the jet grout slab therefore needs to be involved in the details of the jet grouting, to check that the construction practice is going to provide the resistance assumed in the design.

5.5 *Diaphragm walls, piled jet grout slabs and cellular excavation (Esplanade)*

The Esplanade – Theatres on the Bay is touted to be the landmark of Singapore with the two unique domes as shown in Figure 14. It is a world-class cultural and arts centre. The 1,600-seat concert hall boasts the biggest reverberation chamber in the world and the 2,000-seat theatre has an adjustable proscenium arch and orchestra pit with two full-sized ancillary stages. The arts centre covers an area of about 18,000 m^2 and has the shape of a segmented semi-circle with a radius of about 90 m (Fig. 15).

There are no floor slabs at the ground and first basement levels. The aim was to create a large opening spanning across the entire site with a height clearance of about 10 m to accommodate a theatre and a concert hall. A 1 m thick diaphragm wall was constructed along the perimeter of the semi-circular arch. The wall was supported by a series of buttress walls at 8 to 10 m intervals perpendicular to the wall. The idea was to transfer the soil pressure behind the wall onto the slab at the second basement level and then down to the bored piles supporting the slab. The design configuration is shown in Figure 16 (Wong et al., 2002). The site is located along the sea front near the central business area. The 2 hectares site was reclaimed in 1975.

The ground water level was about 2.5 to 3.0 m below surface. The soil condition was very variable across the site. The soil profile along the perimeter is shown in Figure 17. The fill consisted mostly of gravelly sand to clayey and silty sand with thickness varying from 4 to 15 m. The average standard penetration blow count (SPT ‘N’) was about 10. The thickness of upper marine clay varied from 0 to 5 m and the lower marine clay had a thickness up to 20 m near the southwest corner. Marine clay was absent at the northwest corner. The

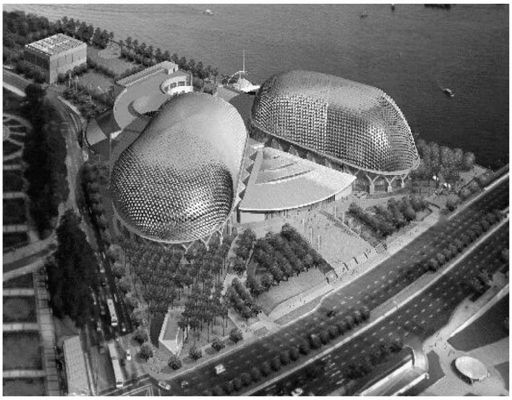

Figure 14. Artist's impression of the completed project.

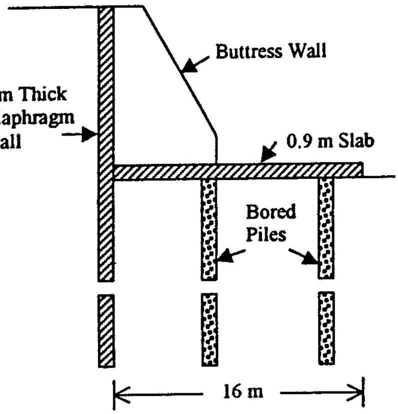

Figure 16. Configuration of permanent wall.

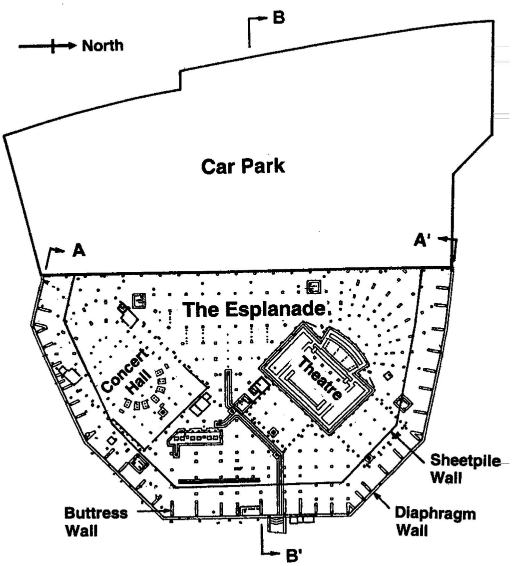

Figure 15. Site plan and layout of car park and centre proper.

clay has a liquid limit of around 75 and a plasticity index of 45 existing fill. The measured c_u/p' ratio was about 0.25. Underlying the Kallang Formation was the Old Alluvium Formation, which consisted mainly of silty sand and sandy silt. The depth to hard stratum, where SPT 'N' $\geq$ 100, varied from 27 to 35 m below ground surface.

Results of consolidation tests indicated that the clay was essentially fully consolidated under the weight of the fill.

The final design took advantage of the semi-circular shape by constructing a 16 m wide concrete slab around the inner perimeter of the site butting against the diaphragm wall at formation level. It was intended for this concrete slab to act as a compression ring to keep the diaphragm wall in-place as illustrated in Figure 18.

In order to construct the 16 m wide concrete slab around the perimeter at formation level, a trench 18 m wide and 10 m deep was dug around the perimeter. A row of temporary sheetpile wall was installed at 18 m parallel to the diaphragm wall as shown in Figure 5. Results of the finite element analysis indicated that the maximum wall deflection due to trench excavation alone would exceed the allowable limit of 75 mm. In order to reduce the wall deflection, jet grouting was considered. With the presence of a 2 m thick jet grout slab, the computed deflection reduced from 82 to 38 mm. Further analyses were carried out to optimize the design of the diaphragm wall in terms of: (i) extend of jet grouting; (ii) wall length and (iii) wall bending moment. These analyses resulted in several wall types (I, II, IIA, IIB and IV) as shown in Figure 18.

The buttress wall was designed to provide permanent support to the diaphragm wall to resist the earth pressure. The buttresses were 1 m thick and 6 m wide, and spaced at 8 to 10 m. A continuous waler beam connected all the buttresses at mid-height of wall. The buttresses acted partly as a wall stiffener and partly as a medium to transfer the soil pressure to the base slab and then onto the supporting bored piles as illustrated in Figure 16. The construction sequence is summarised below:

Stage 1: Construct 1 m-thick diaphragm wall around the perimeter and install the sheetpile wall 18 m away from the diaphragm wall.

Stage 2: Install a 2 m-thick jet grout slab between the diaphragm and sheetpile walls below the formation level. Because of favourable soil conditions at the northeast corner, a small section at that area was not treated. The jet

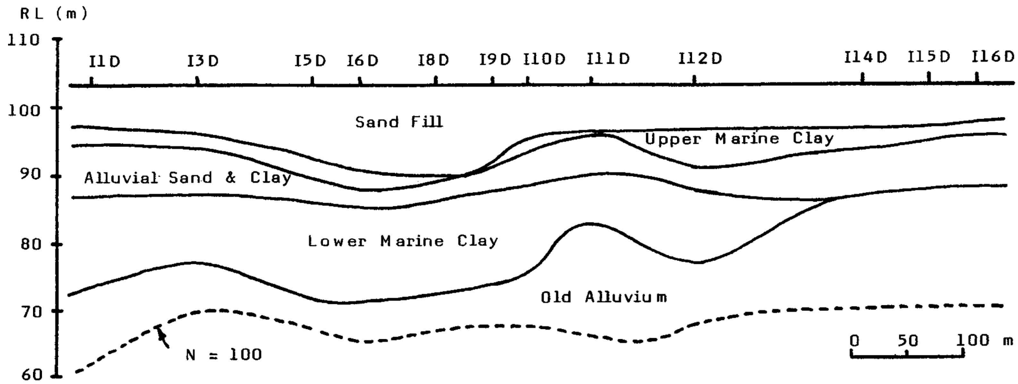

Figure 17. Soil profile along the perimeter of excavation from south to north (Section A-A′).

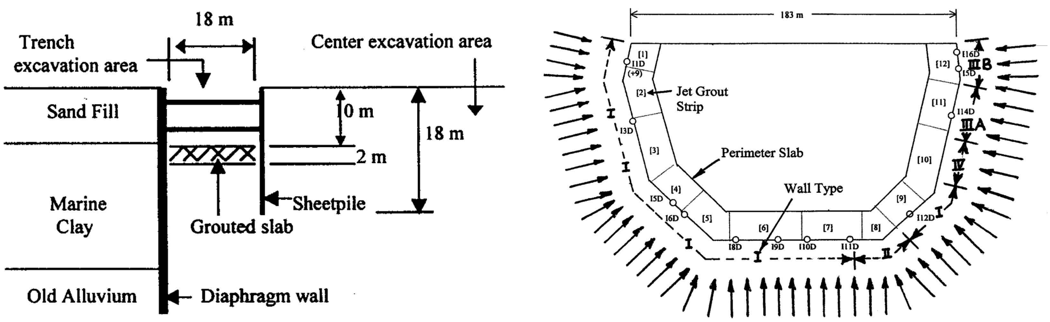

Figure 18. Perimeter concrete slab (16 m wide and 0.9 m thick) at formation level acting as a compression ring.

grouting work was completed by mid-June 1997.

Stage 3: Install bored piles.

Stage 4: Excavate the soil between diaphragm and sheetpile walls to formation level at Reduced Level (RL) 92.5 m. Struts were installed at RL 101.3 m and 97.0 m. The trench excavation was completed by early November 1997.

Stage 5: Excavate pits for pile cap construction.

Stage 6: Construct concrete slab (16 m wide and 900 mm thick) and buttress walls. At the interface between car park and art centre proper, the slab was connected to the lower basement slab of car park. The construction of the buttress walls was completed by the end of 1997.

Stage 7: Excavate the central area bounded by the sheetpile wall uniformly. Remove the struts as excavation level reached below the strut level.

Stage 8: Excavate to formation level and remove the sheetpile. Excavation work in the central area started in early January 1998 and completed in April 1998.

This project was heavily instrumented. The wall deflections were monitored using in-wall and in-soil inclinometers around the perimeter. Earth pressure cells were installed in two diaphragm wall panels to measure the earth pressure acting on both sides of the wall. In-soil and in-pile inclinometers were installed in the central area to monitor the pile and soil movements during excavation. Piezometers and water standpipes were installed to monitor the changes in pore pressure and ground water table respectively. Strain gauges were mounted on the perimeter slab and the buttress walls to measure the compression forces in these elements. Settlement markers and survey points were installed on the ground surface to monitor the ground movement.

At the end of trench excavation, the maximum inward wall deflection varied from 20 to 33 mm. The displacement increased to 27 to 50 mm during the construction of pile caps and buttresses. Since the wall was pushed outward during jet grouting, the net wall deflection was relatively small at this stage.

During excavation in the central area, the ground surface was lowered ‘uniformly’ downward. Strict control was exercised to keep the difference in ground level not to exceed 1 m during excavation. The aim

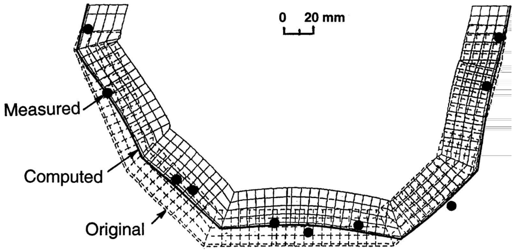

Figure 19. Computed and measured deflections.

was to minimize movement of the installed piles. The maximum wall deflection due to excavation in the central area alone varied from −5 mm (outward) to +25 mm (inward). When combining with deflections during trench excavation, the total deflection after jet grouting varied from 37 to 65 mm. Due to further excavation work at the lyric theatre and local pile caps, there was an increase in movement by about 3 to 5 mm. If the wall movements due to jet grouting were taken into consideration, the final wall deflection from the original vertical position varied from 29 to 70 mm.

Another item of interest is the performance of the perimeter slab acting as a compression ring. The computed and measured horizontal displacements of the perimeter slab are shown in Figure 6. The measured maximum displacements at the crown varied from 10 to 17 mm and the computed displacement was 15.6 mm. The overall deformation patterns are in good agreement.

5.6 *Secant piles and DSM berms (Toa Payoh)*

For the construction of a 3-level basement car park for the HDB Centre at Toa Payoh, a 12 m deep excavation running approximately 150 m parallel to a Rapid Transit Structure (RTS) was needed (Tan et al., 2001). A very stringent limit of not more than 15mm was imposed on the resultant movement allowed for the RTS structure. There were also two key features in this project which require special attention, namely the wide span of the excavation, approximately 180 m, making it difficult to install horizontal braced struts and that the excavation will be in an area with marine and organic clay. As a solution, inclined struts were used. To facilitate this, the ground some distance away from the retaining wall was to be excavated first, and a soil berm was left to restrain the inwards movement of the retaining wall. However, initial design analyses suggest that this would mean almost no movement could be tolerated during the installation of the diaphragm wall, clearly not a satisfactory solution.

To reduce the likely movement during the excavation to provide some additional room for the construction of the diaphragm wall, the berm was improved using lime columns to provide adequate support to the

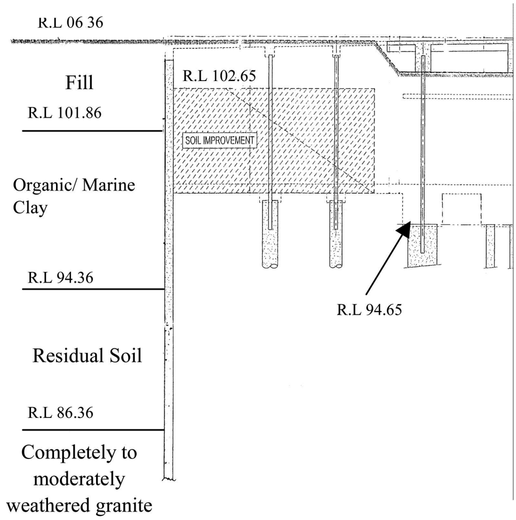

Figure 20. Use of deep soil mixing to provide an improved berm, HDB Centre at Toa Payoh.

retaining wall to limit movement (Fig. 20). The existing soil needs to be improved only a little. The main chemical improving agent used is quick lime (CaO), which absorbs water from the surrounding causing the lime to swell and formed slaked lime ($Ca(OH)_2$). According to Tan et al. (2001), the process of slaking leads to a volume expansion which in turn causes a lateral consolidation effect on the soil adjacent to the lime columns, and moisture content in the soil is reduced. As a longer term chemical effect, the calcium ion in the slaked lime are then absorbed by the negatively charged surface of clay minerals, thereby acting as the cementing agent leading to the improved strength of the soil. The shear strength of the original soil is around 15 kPa. The shear strength of the individual pile is about 150 kPa and the shear strength of the composite treated soil is improved to 20 to 25 kPa. More importantly, the Young's Modulus of the composite soil was improved to 8 MPa, significantly higher than the value of the original unimproved marine clay, which will be about 1 to 2 MPa. However, such improvement is considered low compared to more conventional soil improvement technique such as jet grouting mainly because of the very wide spacing between columns. The measurement during construction indicated that the final resultant movement was just over 10 mm. This indicates that the choice of a low intensity improvement was just right for this job.

6 FAILURES

Although there have been many successful excavations in Singapore, failures do occur. A Workshop on

‘Avoiding Failures in Excavation Works’, was organised by the Building Control Authority (BCA) in 2003. Included in the workshop were 13 brief case studies of excavation failures that had occurred over the previous 10 years. All of the cases involved relatively shallow excavations, from 2 m to 8.5 m in depth. The majority involved excavations in marine or estuarine clays of the Kallang Formation. The cases involved either outright collapse of part of the retaining system (ULS failures), or movements that caused severe damage to adjacent structures (SLS failures). In many cases, the adjacent buildings were so badly damaged as to require demolition and rebuilding. The lateral deflection of the retaining walls ranged from 200 mm to complete collapse. In several cases insufficient consideration of hydraulic pressures was blamed for the failure.

One of the cases cited in the BCA seminar is discussed in outline, in 6.2 below. Two other major collapses, not included in the BCA workshop, are also discussed.

6.1 *Central Services Tunnel*

The Central Services Tunnel (CST) was built by cut-and cover methods through the Telok Ayer basin (shown in Figure 7). The area was reclaimed from the sea following the construction of the tunnels from Marina Bay Station.

The reclamation took place in about 1992, 10 years before construction of the CST. Under 8.5 m of fill there was 17m of upper marine clay, 7 m of fluvial deposits, 3 m of lower marine deposits and then Old Alluvium (Fig. 21).

The 16 m deep excavation was constructed within a combined sheetpile/solider pile retaining wall. The retaining wall was taken to 3 m below final excavation level, in the upper marine clay. To provide basal stability, a 2.5 m thick slab was to be constructed at the base of the excavation. The jet grout slab was tied down by two rows of king posts, driven into the Old Alluvium. The king posts also provided restraint against buckling for the four levels of strutting.

The failure occurred during excavation for the fourth level of strutting (Lim & Tan 2003). The jet grout slab suffered a failure in bending along the centre of the excavation over a length of about 50 m. Buckling of the jet grout slab was accompanied by the king posts punching upwards, effectively taking out the strutting system, and accompanied by inward rotation of one wall of the excavation. Photographs taken after the failure are shown in Figures 22 and 23.

Other areas that had been designed on a similar basis were successfully excavated. According to Lim & Tan (2003), the major differences between the failed area and the other, successfully, completed areas were:

1. There was a stockpile of soil adjacent to this section of the excavation, amounting to about six times the design surcharge of 20 kPa.

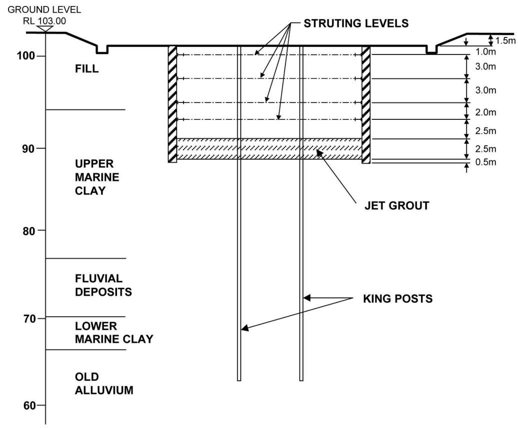

Figure 21. The excavation for the Central Services Tunnel (after Lim & Tan (2003)).

Figure 22. The failed section of the excavation for the Central Services Tunnel, showing the failed base just above the jet grout slab and the north wall rotated inwards.

Figure 23. The failed excavation during recovery. Note the kingposts in the failed section, some of which had been pushed upwards by 3 m, and the rotational failure of the retaining wall.

2. The design precut of 1.5 m, was not carried out on the side of the excavation with the large surcharge.
3. The jet grout slab was constructed to a thickness of 2 m, rather than the design 2.5 m.
4. The king posts were driven to a set, rather than the design penetration in Old Alluvium. As a result, the king posts stopped just at the top of the Old Alluvium, and therefore did not have the capacity to resist the uplift on the slab.

The combination of a massive additional surcharge, and the failure to construct the jet grout slab and to tie it down as required in the design, provide a ready explanation for this failure.

6.2 *Lorong Limau*

This failure occurred during the construction of the underground parking levels for a residential condominium. The planned 8.1 m deep excavation was in 10.5 m of recent deposits, mainly marine clay. The excavation was supported by a combined sheetpile/H-pile retaining wall, and three levels of struts. Due to the shape of the excavation, there were a large number of corners, where the wall was supported by angled struts. At one of these corners, there was no continuity of the walers around the corner. Although the walers were welded to the wall, there was insufficient resistance along the back of the wall to take the component of force along the walers (Figure 24). The return wall and the walers racked into the excavation. The struts on the main wall were therefore ineffective, and the retaining wall deflected by an estimated 600 mm+, with a similar magnitude of settlement on the road and pressurised gas main immediately adjacent to the excavation. The gas main did not rupture, and no one was hurt.

This problem, involving the axial component of force, in walers, from angled struts, is a common cause of excavation failures in Singapore, and is not limited to marine clay.

Figure 24. The walers in this photograph had moved longitudinally, allowing the wall to deflect by over 600 mm.

Figure 25. Nicoll Highway area after the collapse.

6.3 *Nicoll Highway*

On the 20th April 2004 a section of cut-and-cover tunnel near Nicoll Highway collapsed, killing four people working at the site (Fig. 25). The Committee of Inquiry into the collapse commenced in August 2004 and issued a final report on 13th May 2005. The following brief summary is based on the final report (Magnus et al., 2005), of the Committee of Inquiry and on a report in the New Civil Engineer (Mylius 2005).

At the time of the collapse, excavation was in progress to allow installation of the 10th, and final, level of struts, and was over 30 m deep (Mylius, 2005). Nine levels of struts had already been installed to support the 800 mm thick diaphragm walls. Two jet grout slabs had previously been installed. One of these jet grout slabs, placed above the ninth strut level, was sacrificial and had been removed. The other slab was positioned just below final excavation level.

Expert witnesses testifying at the court of inquiry identified problems in the design and the construction of the work. Design problems involved both the structural design and the geotechnical analysis (Mylius, 2005).

The structural design problems related to the design of the struts and the strut/waler connections. Omission, on the drawings, of splays at the end of the struts (which were assumed in the design calculations), and errors in the design of the strut/waler connection resulted in a strutting system which had about half of the ultimate design capacity that it was supposed to have. This was compounded by the substitution of 'C' channel for plate stiffeners in the level 7 to 9 walers. This substitution resulted in a system that failed in a brittle, rather than ductile, manner. The final failure was initiated by buckling of the webs of the walers at the 9th level of struts.

The geotechnical analysis involved an incorrect soil model for the marine clay in the finite element analysis used for the design. The analysis used effective stress parameters with a Mohr-Coulomb model, with the material type set as 'undrained'. This was compounded by the use of a pore pressure distribution in the passive zone that was hydrostatic with respect to the base of the excavation. The results of these modelling errors were:

- The design moment capacity of the wall was about half of what it should have been
- The predicted movement of the wall was about half of what it should have been
- The jet grout slabs experienced higher strains than predicted
- The design load in the 9th level of struts was about 10% lower than it should have been. However, the total design load of the 1st to 9th levels of struts was about 20% higher than would have been required in the analysis recommended by the Committee of Inquiry.

As a result of the under-design of the wall, most of the experts considered that the wall had formed a plastic hinge by the 17th April, three days before the collapse.

In addition the Committee of Inquiry reported on many other factors. These included:

- Insufficient penetration of the diaphragm wall into the main bearing stratum (the Old Alluvium). The original design did not allow for sufficient penetration due to the incorrect modelling; several of the wall panels in the area of the collapse failed to achieve even the penetration required by the design
- 66 kV cables crossing the excavation, which resulted a weak point in the support system
- The diaphragm walls were curved in plan
- There was a deep buried channel which was not fully identified by the designers
- A poor instrumentation and monitoring system. A lack of experience and skill in many of the personnel involved was particularly mentioned
- Delayed installation of the 10th level of stuts; 8 bays were not installed at the time of the collapse
- Poor quality control
- The lower jet grout slab was weaker and thinner than assumed in the design

The report commented on problems, and a lack of clarity, in the chain of command and communication within the builder's organisation. Although the report states that there was insufficient evidence to assess the state of the remaining jet grout slab at the time of the collapse, a number of issues concerning the design and construction of jet grout slabs were covered, including recommendations on limiting strains in the jet grout.

7 CONCLUSIONS

The deep deposits of near normally or normally consolidated marine clay that occur in Singapore can present significant problems for the design and construction of deep excavations. For excavations in excess of about 6 m in depth basal stability becomes an issue, if the walls are not taken to hard strata. Where the walls are taken to hard strata, then the walls have to be designed for 'net active' conditions in the marine clay, and consideration has to be given to consolidation settlements due to under-drainage of the clay. A variety of special techniques have been used to allow safe excavation and to restrict wall movement and settlement. These techniques have included jet grouting, lime piling, underwater excavation and sequenced construction, often used in combination.

While the basic problems of carrying out deep excavations in the marine clay are well known, there have been a number of significant failures over the last decade. Many of these failures fall neatly within the four main causes of failure suggested in CIRIA Report C580 (2003). These include:

- Inadequate understanding of the geological and hydrological conditions (Several of the cases cited at the BCA seminar in 2003, Nicoll Highway)
- Poor design and construction details and poor standard of workmanship, particularly of support Systems (Central Services Tunnel, Lorong Limau and Nicoll Highway)
- Construction operations and sequences that differ from those in the design (Nicoll Highway)
- Inadequate control of site operations including excessive surcharge (Central Services Tunnel, Nicoll Highway)

The Nicoll Highway case also involved issues related to the use of numerical methods for design of deep excavations. These issues included the soil model used, and how the numerical analysis should be incorporated into the final design. Such issues are generally not covered in current codes. With the increasing use of numerical methods as part of the design process, detailed consideration needs to be given to how, and to what extent, numerical methods should be incorporated into the design of deep excavations.

Another important geotechnical issue arising from the Nicoll Highway collapse involves the design and construction of jet grouted slabs (or other forms of ground treatment), where these are a part of the excavation support system. Although these are commonly modeled as hard soil, consideration also needs to be given to them as weak structures, and to the potential for a brittle mode of failure. The construction procedures for such slabs should be reviewed by the designer, to ensure that the resistances assumed in the design can be achieved in practice.

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The effects of tunnelling on existing structures

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ABSTRACT: This paper presents a general report regarding the effects of tunnelling on existing structures based on the papers presented at the TC 28 symposium held in Amsterdam 2005. This report is subdivided in experimental research, modeling and monitoring. Moreover the report distinguishes between piled and non-piled structures.

1 INTRODUCTION

In most metropolises the number of urban underground constructions is increasing steadily. These activities are related to replacing old structures by new ones, often high-rise buildings with underground parking facilities, or infrastructure works such as motorways, railways and subway tunnels. To assess the impact of these construction operations on existing structures is increasingly important for designers. In particular tunnelling near piled foundations is a subject that recently has gained the interest of the geotechnical research world, mainly because of a number of tunnels that are projected in city centres with piled foundations such as the North/South Line in Amsterdam and the Channel Tunnel Rail Link in London. A better insight into the response of piled structures to tunnelling is needed to carry out these projects without harming monumental buildings.

This general report focuses on the results of research on this topic. This general report discusses 14 papers, directly addressing this topic. The papers originate from 7 different countries, authors come from both the academic world as from consultants. This report is restricted to bored tunnel operations and does not deal with the effects of excavations on existing structures. An overview of the papers that are discussed in this general report is given at the end of this report.

In the innovation cycle three stages and type of research can be distinguished:

- experimental research
- modeling, (predictions and calculations)
- monitoring (field testing and case histories)

This report will be subdivided in these three types of research. Moreover this report will distinguish between non-piled structures (6 papers) and piled (8 papers). The piled structures refer to buildings and bridge piers, the non-piled structure to buildings on shallow foundation and tunnels, sewers etc.

When considering the effects of tunnelling on existing structures and more in particular to estimate the potential damage to buildings three aspects are of importance:

1. the assessment of the magnitude of the ground deformations induced by the tunnelling process,
2. the response of the structure to the ground movements and
3. the tolerable deformations of the buildings.

Most papers presented to this symposium address two out of these three aspects. Determining the ground deformations is the first step; the response of the structure to these deformations is the next. Analyzing this response one distinguishes between piled and non-piled foundations. Papers dealing with ground movements only are reported in one of the other general reports.

2 EXPERIMENTAL RESEARCH

Experimental research into the tunnel-soil-structure interaction is often carried out in a geotechnical centrifuge. In this symposium **Carporaletti, Burghignoli & Taylor (2005)** and **Kaalberg, Teunissen, Van Tol & Bosch (2005)** studied the impact of tunnelling on respectively non-piled and piled structure in a geotechnical centrifuge, while **Standing & Leung, 2005** describe in their paper presented to this symposium photo-elasticity experiments to analyze the reverse process: the stresses developed around a tunnel during piling.

Standing & Leung (2005) describe photo-elasticity experiments to analyze the stresses developed around a tunnel during piling. Photo-elasticity techniques were used earlier to investigate stress distributions in geotechnical problems. Allersma (1987) for example studied the stresses around pile tips with this technique. This technique requires for soil mechanics applications a particulate material surrounded by a

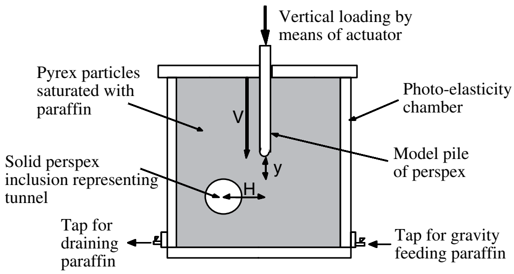

Figure 1. Test set-up (Standing & Leung, 2005).

liquid medium of the same refractive index. Standing and Leung used Pyrex for the particles, Perspex for the tunnel and pile and paraffin for the liquid. In a clear-sided chamber these materials are transparent, but block fully polarized light. Stressed elements become visible by refraction showing stress paths and stress intensities. Figure 1 shows the schematically the test set up. The pile was penetrated by force at different positions relative to the tunnel up to a horizontal distance from the side of the tunnel equal to the tunnel diameter. The tunnel behaved relatively stiff. Based on the experiments the authors conclude that the impact of piling on the tunnel is similar to the influence of tunnelling on a piled foundation as presented by Jacobs et al. (2003). Moreover they conclude that piles installed at a distance less then 0.5 D from the side of the tunnel transfer a considerable part of the load from the pile toe to the tunnel lining. Although the applied technique provides only qualitative results the authors consider the technique applicable to assess boundary values, as for example the distance at which load transfer starts.

Experimental research into the tunnel-soil(-building) interaction in a geotechnical centrifuge research has been carried out by a number of researchers. Grant & Taylor, (1996) studied tunnelling-induced ground movements. Bezuijen & van der Schrier (1994), Loganathan et al. (2000) and Jacobsz et al. (2003) researched the effects of tunnelling on piled structures. One of the important issues in this kind of centrifuge research is the modeling of the tunnelling process. Grant & Taylor used a cavity lined with a rubber bag and simulated the tunnelling process by reducing air pressure in the bag. Bezuijen & van der Schrier applied a cylinder with a decreasing diameter. Loganathan et al. Jacobs et al. also used a cylinder but with a membrane around the cylinder with respectively oil and water in between and simulating ground loss by decreasing the liquid volume.

Caporaletti et al. (2005) report in their paper to this symposium the research, also performed in a centrifuge, on the effects of tunnelling in layered ground on existing non-piled structures, by modelling a completely buried thin 'historical wall' perpendicular to the tunnel axis. The purpose of the study was to assess ground movements and strains at different depths and the potential damage on pre-existing structure. The set-up was plane strain. A stratum of 12.8 m sand overlies a 22 m layer of clay (prototype scale). The tunnel was cut in a stiff over-consolidated kaolin clay layer and lined with a rubber bag and represented a real tunnel of diameter equal to 8 m with an axis located about 23 m underneath the ground surface. Similar to Grant & Taylor the excavation process was simulated by reducing the air pressure in the tunnel liner bag. The model wall (of an existing building) corresponded to a prototype wall 15.2 m high and 4 m width. It was placed within the upper sand layer with its foundation just at the sand/clay interface. The sand stratum consisted of medium dense sand ($D_r = 60 - 70\%$). The water level was set at about 4 m above the sand/clay interface.

The first test modelled the greenfield condition and subsequent tests used model walls of different strength and stiffness to investigate the interaction problem. The most significant tests (with the weakest wall and with strongest wall) were presented and analysed at fixed reference values of volume loss ($V_L = 5.0\% - 10.0\% - 15.0\% - 20.0\%$). The distance i_z of the inflexion point from the tunnel centreline for the settlement troughs was evaluated at every horizon, both in sand and in clay, and was found to remain constant at each horizon during the process of simulated tunnel excavation.

All the measured distance i_z can well be fitted by using the equation presented by Moh et al. (1996) for drained soils. No evident differences were noted between the behaviour of the weak wall and the strong wall. The patterns of movement were independent of magnitude of volume loss. In order to assess trends for the greenfield test and ground-structure interaction tests all values of i_z and H (= focus of displacement vectors) are plotted against depth in Figure 2. This figure demonstrates that the theoretical trends addressing normalized parameter of settlement trough against normalized depth from ground surface suggested from literature (O'Reilly & New, 1982 and Mair et al., 1993) underestimate the values assessed from experimental data, both on the ground surface and subsurface.

The authors also evaluated contours of volumetric strains in the wall. Quite different contours were found for the weak wall test compared to the greenfield and strong wall tests. Maximum shear strains in the wall-ground interaction tests are similar both in shape and in magnitude. The pictures in Figure 3 present the failure mechanisms of the weak and strong wall. The failure mode of the weak wall is characterized by bending deformation causing cracking due to direct tensile strain. In contrast, the strong wall is characterized by a shear deformation with cracking due to diagonal tensile strains.

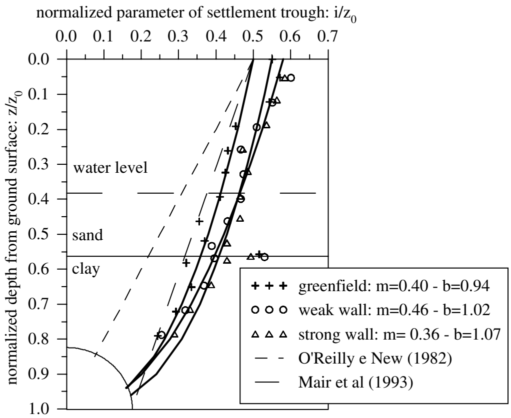

Figure 2. Normalized parameter of settlement trough (i/z_0) against normalized depth from ground surface (z/z_0), (Carporaletti et al., 2005).

The paper from **Kaalberg et al., 2005**, to this symposium presents a research overview with the results of centrifuge research, already reported earlier (Bezuijen & van der Schrier, 1994). In that research large pile settlements were observed due to tunnelling also outside the triangular zone as presented later by Kaalberg et al., (1999) and by Jacobs et al. (2003), respectively based on field tests and centrifuge tests. Also the results of the field tests reported by Kaalberg et al., 2005 confirm the validity of the theory that piles with a toe position outside the triangular zone are not affected by tunnelling, at least with small values of ground loss. The contradiction between these results and the former centrifuge test results can probably be explained by the magnitude of the volume loss in the former centrifuge tests. Outside the triangular zone large pile settlements may occur in case of large volume loss (more than 3%).

3 MODELLING

3.1 *Impact on non-piled structures*

Three papers presented to this symposium address modelling of different aspects of the tunnelling-soil-building interaction. **Simic (2005)** focuses on the modification factors for the deflection ratio as presented by Potts & Addenbrooke (1997) using the monitoring data from three tunnel projects in Madrid and compared these data with numerical simulations. **Franzius & Potts, Burland (2005)** studied the twist deformation of buildings along a tunnel trajectory due to the 3D aspects of tunnelling by 3D FEM parametric analysis in order to assess modification factors for twist. **Netzel (2005)** focuses on damage criteria for buildings analyzing the limiting tensile strain method.

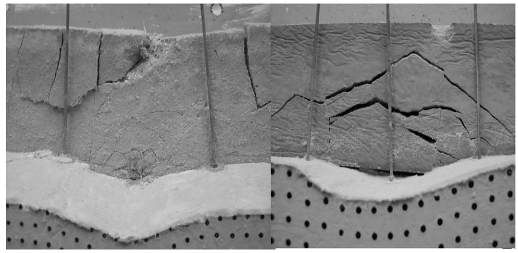

Figure 3. Failure mechanism of the weak (left) and the strong wall (right), (Carporaletti et al., 2005).

The investigations in **Simic's (2005)** paper concern both the discussion of different models to quantify ground movements and the damage induced to buildings. The author processed the measurement data of three tunnels bored in the city of Madrid to compare greenfield values with the corresponding settlement of buildings along the tunnel route. The interaction effects have been analyzed in terms of the building situation in respect of the settlement trough and of the structure type. The three tunnels were excavated and supported in two stages using the traditional Madrid method with different excavated sizes: 49 m^2, 78 m^2 and 125 m^2. The three tunnels were bored under urban areas in medium dense to dense silty sands with occasional layers of clayey sands that originate to the Miocene. Settlements were measured in different sections along the tunnels routes, either in greenfield conditions and below existing buildings. A total of 26 buildings were surveyed during the tunnel construction.

The surface settlement points were evaluated by fitting to a Gaussian distribution to obtain the percentage of volume loss. The building settlements were compared to the greenfield settlements in an equivalent situation of tunnel depth and geotechnical conditions to assess the modification factors of the deflection ratio for hogging and sagging (Potts and Addenbrooke, 1997).

However, the recommended assessment of the bending stiffness of the building by Potts and Addenbrooke implicitly assumes that the columns of the building are infinitely rigid, grossly overestimating the stiffness of the structure. Simic uses three dimensional slab and column elastic analysis to obtain the real stiffness $(E_c\ I)_{flex}$ of the building. The back-analysed values of the modification factors for the deflection ratio M^{DR} for sagging and hogging are shown in Figure 4, together with the parametric curves proposed by Potts and Addenbrooke (1997).

The author concludes that although there is a wide scatter: *i* buildings in sagging tend to behave less stiff than the parametric model, yielding higher modification factors, *ii* buildings in the hogging mode tend

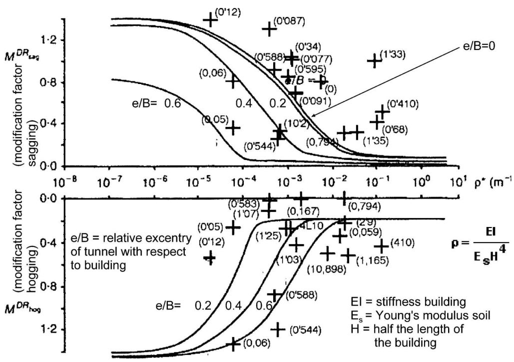

Figure 4. Comparison of the back analyzed values of modification factors with theoretical values from Potts and Addenbrooke (1997), (Simic, 2005).

to follow more closely the parametric model, except for some particular cases of structures of load bearing walls which, due to brittle failure.

In addition a three dimensional finite difference model (FLAC 3D) was run to simulate the tunnel construction below one of the buildings in hogging mode. The excavation process was modelled by imposing a ground loss that resulted in a 'greenfield' settlement similar to that measured in a control section. The calculated settlements of both ground and building were in good agreement with the measured values. In this model, the tensile strains of the building yielded a maximum value of 0.046% which was very close to the result of the equivalent beam analysis (by Burland & Wroth, 1974 and Mair et al., 1996) using the modification factors of Potts and Addenbrooke (1997).

The analyses of Potts and Addenbrooke and of Simic concerned the settlement assessment assuming a 2D deformation of the building. Case studies have shown that buildings can be distorted three-dimensionally (3D) during the process of tunnel construction. The paper by **Franzius et al. (2005)** tot this symposium presents a 3D Finite Element (FE) parametric study of twist deformation of buildings due to tunnel induced ground movement. For this paper a definition of twist derived from expressions used in structural engineering was adopted relating the settlements of four corners of a building to the width and length. This expression for twist has the dimension [1/length]. FE modelling of the tunnelling process by different researchers is discussed below together with the impact on piled structures.

In the 3D FE calculations the tunnel, with a diameter of 4.1 m, was placed at a depth of 20 m. The tunnel lining was represented by elastic shell elements. Tunnel construction was modelled by successively excavating 2.5 m long 'slices' of soil elements in front of the tunnel face and activating one ring of tunnel lining after each excavation step (i.e. the soil remained unsupported over 2.5 m behind the face). The tunnel excavation was modelled over 100 m, while all buildings were in plan perpendicular to the tunnel axis. The soil profile consisted of London Clay represented by a non-linear elastic plastic constitutive model. The initial coefficient of earth pressure at rest was $K_0 = 1.5$. The soil was modelled to behave undrained.

The building structure was represented by elastic shell elements and the stiffness parameters were chosen to represent 1-, 3-, 5- and 10-storey buildings. 8 different building scenarios were analysed, to investigate the behaviour of building twist, varying building stiffness, building geometry and tunnel depth. The twist behaviour was quantified by introducing twist modification factors M^{Θ}, similar to M^{DR} adopted by Potts & Addenbrooke (1997). The twist modification Factor M^{Θ} is defined as the ratio between the peak twist of the building case over the maximum twist in the corresponding greenfield scenario. The study revealed that these factors M^{Θ} reduce with increasing building stiffness and increase with increasing building geometry an increasing tunnel depth. This trend of smaller structures leading to smaller values of twist deformation shows that smaller buildings show a stiffer response than large structures. In contrast to the small twist deformation, it was found that the stiffer building response leads to an increase rigid-body motion of the whole building, i.e. the entire structure rotates towards the tunnel face as it approaches and rotates back as the excavation passes beyond the buildings. After 70 m of tunnel face passage the magnitude of the twist remains constant.

Netzel (2005) focusses on damage criteria for buildings analyzing the limiting tensile strain method. The limiting tensile strain (LTS) method is an empirical, analytical method to predict the degree of possible settlement damage on the buildings. The calculated diagonal and bending strains in the beam are consequently compared to different limiting tensile strain values representing different damage classes with an indication of degrees of damage. Design charts are developed by Burland & Wroth (1974) and Boscardin & Cording (1989). Netzel presents the results of a fundamental analyses of the basic assumptions made by the developers from structural point of view and its impact on the calculated tensile strains in the buildings. The current LTS-method uses equation (1) presented by Timoshenko to calculate the tensile strains in the structure but with a shear form factor of 1.5.

$$\Delta/L = \frac{P \cdot L^2}{48 \cdot EI} + \frac{P \cdot 1.2}{4 \cdot G \cdot A} \quad (1)$$

Where Δ is the maximum deflection of the beam, E,G and I are building parameters (respectively: Young's modulus, shear modulus and the moment of inertia), L the considered span length of the building, and P the fictive vertical point load at midspan of the beam, causing the impose deflection profile.

Equation (1) is based upon the maximum shear strain at the neutral axis of the beam. Timoshenko, however also presents solutions where the influence of the shear strain distribution over the height of the beam is properly taken into account, thus resulting in a shear form factor of 1.2 for a rectangular beam as presented in equation (1). The influence of the different shear form factors on the calculation of tensile strains is analysed by Netzel. The correct shear form factor leads to an increase of the strains between 20 and 25% for practical ranges of L/H (between 0.7 and 1.5). Netzel therefore recommends to use a shear form factor of 1.2 when predicting the building damage using the tensile strain method.

If a building is longer than the influence area of a settlement trough, the current LTS only considers the part of the building inside the influence area (cut off for the building at 1 mm settlement line). Obviously neglecting the 'cantilever' effect of long structures. In his paper Netzel shows that the 'cantilever' effect can cause a significant increase of the strains (up to 80%) and has therefore to be considered especially, when the L/H-ratio of a building in the hogging zone is smaller than 3 (which is the case in most situations) and the proceeding length of the structure beyond the 1 mm line is more than one time the length of the structure inside the 1 mm area.

Netzel emphasizes in his paper that the design chart by Boscardin & Cording (1998) and the design chart by Burland et al. are only applicable for the case of a massive bearing wall in the hogging zone and the L/H-value of 1. A fundamental difference between the two charts is the use of the parameter deflection ratio or the angular distortion as measure for the building distortion due to vertical (differential) settlements. Netzel investigated the influence of these two approaches on the determination of the tensile strains and thus the damage class. The author recommends to use the maximum angular distortion for the determination of the diagonal tensile strains and the deflection ratio to determine the bending tensile strains with the LTS for Gaussian formed settlement profiles.

3.2 *Impact on piled structures*

In the field of the prediction of tunnelling-induced deformation of piled constructions two approaches can be distinguished:

1. staged calculation methods containing
   a. the estimation of the tunnelling-induced ground deformation (with taken any foundation into account, also named free field ground movement)
   b. the analysis of the response of the pile or pile group to these deformations
2. fully numerical 3D analyses

In this category four papers were presented dedicated to prediction models. **Surjadinata, Carter, Hull & Poulos (2005), Kitiyodom, Matsumoto & Kawaguchi (2005)** and **Matsumoto, Kawaguchi & Kitiyodom (2005)** describe staged calculation approaches and **Lee & Ng (2005)** a FE modelling. In two other papers prediction models were described and used in combination with monitoring, **Jacobsz, Bowers & Moss (2005)** applying stage calculation method and **Kaalberg et al. (2005)** FE analysis. The prediction models used in these two papers will shortly be discussed here.

3.2.1 *Staged calculation models*

This approach was presented by Chen et al., (1999 and 2000). They estimated the free field movements with empirical models. Loganathan and Poulos (1999) applied this approach with a closed form solution for the free field deformations. In both papers boundary element analyses were used for the pile response. **Jacobsz et al. (2005)** applied in their paper with different case histories a similar approach, analyzing the free field deformations with an empirical model (New & Bowers, 1994) and using simple straight forward analyses of the pile response.

The paper of **Surjadinata et al. (2005)** presented to this symposium continues with this approach but using a FEM analyses to establish the free filed movements. Because of the combination with the boundary element method to analyse the pile response they name it the FAB-method (**F**inite **A**nd **B**oundary element method). The tunnelling-induced deformations and were modelled with the FEM by prescribing displacements at the tunnel boundary corresponding with a pre-defined ground loss. The calculation of the free field deformations was validated with two well documented case histories. The authors focus in their paper in particular at the horizontal pile displacements as the tunnels are situated well above pile toe level. Their calculation results are compared with predictions from Chen et al. (1999). The agreement between both approaches is reasonable, but it has to be mentioned that the benchmark Case of Chen et al. (1999) is not validated with measurements. Nevertheless Surjadinata et al. did compare their findings also with the case history presented by Lee et al. (1994) in which horizontal pile displacements were indeed measured. Figure 5 show the results. They conclude that their so-called FAB method provides slightly conservative predictions.

Kitiyodom et al. (2005) and **Matsumoto et al. (2005)** present in their respective papers similar approaches using the computer model PRAB to analyze the pile, pile group and piled raft response. PRAB uses an analytical solution based on an extension of Mindlin's solution for passive loading f.e. ground movements. To estimate these free field ground movements empirical and analytical solutions (Sagaseta, 1987, Verruijt and Booker, 1996 and Loganathan & Poulos, 1998) as well as a numerical method are applied. The calculation results were compared with fully numerical analyses using FLAC3D. In their models the volume loss was induced using the gap factor

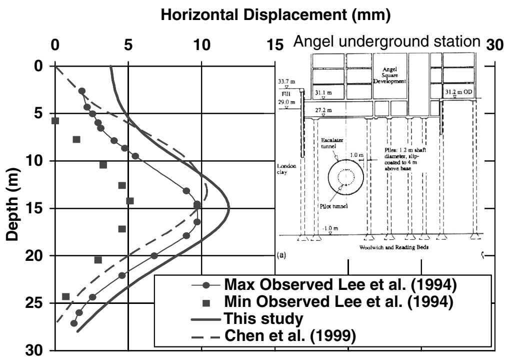

Figure 5. Measured and predicted horizontal pile displacements 5.7 m from the centreline of the tunnel for the Angel Underground Station, (Surjadinata et al., 2005). The corresponding cross-section (Chen et al., 1999).

presented by Lee et al. (1992). Kitiyodom et al. apply this model to predict the effect of tunnelling on pile groups and Matsumoto et al. use the same model for the effects on piled raft foundations. The calculation results for the pile group are compared with calculation results from Loganathan et al. (2001), see Figure 6.

Matsumoto et al. compare their results with FLAC3D, see Figure 7. In general there is a reasonable agreement.

Although fully numerical 3D analyses are more and more feasible due to hardware and software developments they are still very time consuming and therefore expensive and not particular fitted to make numerous calculations for example for different cross-section, parametric studies and different building and foundation configurations etc. Thus the staged calculation methods are still attractive because of the relative good quality of the predictions especially in combination with validation with case histories or a fully numerical method and the fact that the time of these predictions is mere a fraction (approximately 1/1000th) of the time needed for a full 3-D FEM approach, Surjadinata et al. (2005).

3.2.2 *Fully numerical 3D analyses*

Soil-tunnel-pile interaction has previously been investigated by fully numerical methods, in 2D by

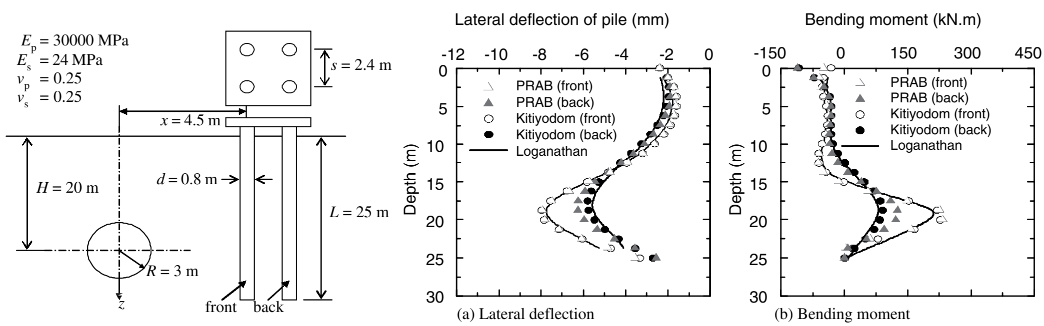

Figure 6. Problem that has been analyzed (left), comparison of calculation results (right), the makers named PRAB are the results of this study (Kitiyodom et al., 2005).

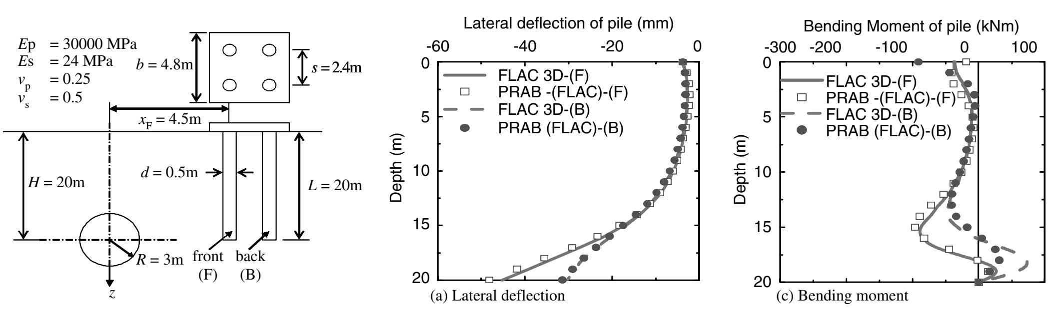

Figure 7. Problem that has been analyzed (left), comparison of calculation results (right), (Matsumoto et al., 2005).

Vermeer & Bonnier, (1991) and in 3D by Van Dijk, (1998) and Mroueh & Shahrour (1999). The first two simulated the tunnelling process with a contraction model, Mroueh & Shahrour (1999) with a staged construction where in the second stage the soil was removed and the lining was activated. As stated above the fully numerical 3D calculations are in development and applied increasingly. In this symposium three papers deal with fully numerical 4D predictions of the effects of tunnelling-induced deformation of constructions, wherein 4D means 3D with an advancing TBM. **Franzius et al. (2005)** and **Lee et al. (2005)** combine the advancing TBM with an unsupported span in stiff clay, respectively for the effect on non piled and piled structures. **Kaalberg et al. (2005)** performed numerical 3D predictions of the effects of tunnelling-induced deformation of piled constructions with an advancing TBM in combination with front pressure and tail grouting.

The authors mention the use of different computer codes respectively ABAQUS and DIANA. This general report will only address the specific aspects related to the topic of this report, such as the schematisation of the tunnelling process and features of the pile foundation.

Lee et al. (2005) simulate the construction of an open face tunnel excavation in stiff (London) clay with an unsupported span of 3 m before a shotcrete lining is installed. The excavation rate amounts 3 m/day. The numerical analysis takes this advancing process and the related consolidation into account. The pile in their study was a 'wished' in place concrete pile, with a diameter of 0.80 m. The pile is loaded up to a factor of safety equal to 3.0. The calculated surface settlement after passage of the tunnel (plain strain) is compared with results of the centrifuge tests by Loganathan (2000) and seems after a correction for the differences in volume loss reasonable. The paper shows pile head settlements as a function of the advancement of the tunnel, expressing the additional pile settlement as a percentage (in this case 0.12%) of the tunnel diameter. This additional pile head settlement (167% of the initial pile head settlement) is translated into an equivalent of the applied working load, from which the authors conclude that the FOS is reduced from 3.0 to 1.5, see Figure 8. According to this approach the authors assume that pile load versus pile displacement behavior was not affected by the tunnelling process.

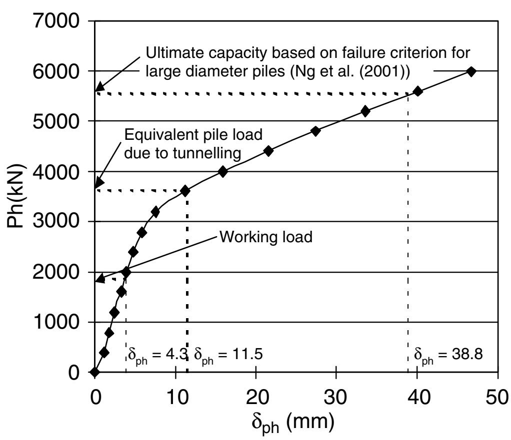

Figure 8. Pile head load versus pile head displacement (Lee et al., 2005).

Kaalberg et al. (2005) mention in their paper, concerning Dutch research, that the full-scale tests were accompanied by fully numerical calculations. In their case the tunnelling process in saturated sand with a slurry supported face was simulated and the interaction with driven piles was analyzed. In the initial analyses a contraction of the tunnel simulated the volume loss related to the tunnelling process. In subsequent calculations the tunnel process was simulated in time and taking front pressure and the grout pressure in the tail into account. To the opinion of the authors the simulation of the grout pressure distribution in the tail void leads to a more realistic input of the tunnelling process and much better results concerning the stresses around the tunnel after TBM passage resulting in a steeper settlement contour perpendicular to the tunnel axis and a more realistic longitudinal trough at the surface. This 4D model was calibrated using the monitoring data from the Heinenoordtunnel.

Another important aspect of their analyses is the stress condition around the driven piles. As they were in particular focussing on possible stress relieve around the pile tips a realistic stress condition in these zones was required. Therefore a calculation model based on an Eulerian principle (Berg, 1994) was used to simulated larges strains related to pile installation.

4 MONITORING

4.1 *Effects non-piled structures*

Two papers were presented to this symposium containing monitoring data from existing tunnels due to tunnelling at short distances. The paper of **Kwast & Van Oosterhout (2005)** presents the predicted and measured impact of the boring of the second tube of the Pannerdensch Canal Tunnel on the first tube. This tunnel is part of the Betuweroute, a new cargo railway link between the harbour of Rotterdam and Germany. The tunnels were driven with a TBM type slurry shield. The minimum distance between the tubes is rather small (<0.5 D) and no mitigating measures were taken before boring. The outside diameter is 9.5 m and the lining thickness is 0.42 m. At the monitoring location the distance between the tubes varies from 3.5 m to 5 m (ca. 0.35 D to 0.5 D) and the cover above the bore tunnel varies between 1.0 D and 0.6 D. At this location the tunnel was bored through a refilled sand pit with mainly middle to well compacted uniform sand. Predictions were performed by 2D FEM analyses showing a maximum horizontal and vertical deformation of the outer diameter of the first tube of respectively −0.02% (−2 mm) and +0.02% (+2 mm) during construction of the second tube. Deformation measurements were carried out at 3 cross sections with distances between the tubes equal to 5.0 m, 4.0 m and 3.5 m. The measured diameter deviations of the first tube (horizontal and vertical) during passage of the second were negligible (+/−2 mm), which was in accordance to what was predicted.

The maximum calculated change of lateral ground stresses during grouting of the second tube was 35 kPa at a distance of 1.0 m out of the lining of the second tube decreasing to nil at a distance of 4.0 m. Two cross-sections were analysed with regard to changes in lateral ground stresses by spade cells. The increase of ground pressure during TBM passage of the first tube at an angle of 45°, by spade cells 2 and 6 is respectively 200 kPa and 125 kPa, see Figure 9. The rather high increase of ground pressure at these locations is caused by a nearby located grout injection point of the TBM. The maximum increase of water pressures during TBM

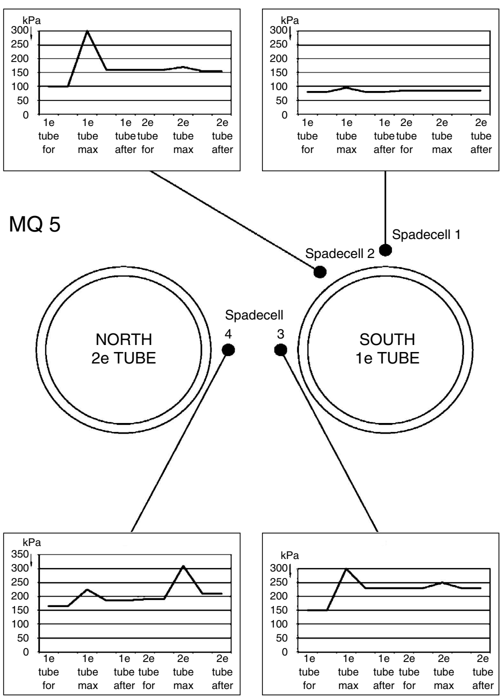

Figure 9. Results of spade cells (in kPa), (Kwast, E.A. & G.P.C. van Oosterhout, 2005).

passage is 40 kPa and disappears just after passage of the TBM.

The main conclusion of this research project was that no measurable influences were observed on the first tube due to the construction of the second tube at a distance of only 3.5 m to 5.0 m from the first tube.

Moss, N.A. & K.H. Bowers (2005) report the effect of new tunnel construction under existing tunnels. Section 2 of the Channel Tunnel Rail Link (CTRL) high-speed railway includes 36 km twin running tunnels driven through mixed soft ground conditions beneath east London. The tunnels were excavated using a TBM with an Earth Pressure Balance (EPB) shield and an excavated diameter of 8.15 m. Controlled pressure grouting through the tailskin was carried out concurrently with excavation. Additionally the annulus around the shield itself was supported with pressurised fluid. A 350 mm thick steel and polypropylene fibre reinforced precast concrete tunnel lining was erected inside the TBM tailskin to form the final lining.

The CTRL tunnels pass beneath buildings, bridges, surface railways, an underground station and under six existing operational metro tunnels. The existing metro tunnels included a variety of different types of metro tunnel constructions. The philosophy for the CTRL tunnelling was based on minimizing the ground movement at the TBM and systematically assessing the risks to determine if any supplementary measures were necessary. The assessment process for the effects of CTRL-tunnelling involved three stages, aiming to reduce the number of structures by eliminating those demonstrated to be at low risk.

Stage 1 assessment of settlement contours showing greenfield settlement contours, using the Gaussian distribution model described by New & O'Reilly (1991). Assumed Volume loss of 1% (contract requirement) and 2% as conservative starting point.

Stage 2 assessment comprises damage categorisation according to the model described by Boscardin& Cording (1989) and the more recent model proposed by Mair et al. (1996).

The stage 3 approach applied to the assessment of the metro tunnels (being extraordinary structures), commenced with ground movement predictions, according to the model proposed by New and Bowers (1994), for a credible range of volume losses and values of the trough width parameter. The results of the calculations were then applied to the metro tunnels using simple beam and spring models. The capacity of the metro tunnel lining system was defined in terms of a limiting bending curvature. Once the critical curvature had been defined, an iterative process was undertaken to determine the matching volume loss and hence the tolerable vertical settlement. In cases where the calculated capacity of the metro tunnel was deemed insufficient to resist the predicted settlement mitigation works were undertaken in advance of the tunnelling. The paper by Moss and Bowers describes the stage 3 assessment for the six metro crossings.

Ground movement during tunnelling was limited by close TBM control and intensive monitoring of the metros tubes. Systems of monitoring trigger levels and associated contingency plans were prepared to manage any adverse events.

The authors conclude *i*. that Metro crossing are conducted without disruption of operation of existing railways *ii*. the detailed specification of TBM and risk engineering contributed to remain within budget and time schedule *iii*. the assessment of ground deformations was reasonably accurate and the magnitude of volume losses lower than initially expected.

4.2 *Effects on piled structures*

Two papers presented to this symposium report the results of full-scale tests in which the behaviour of piles was monitored during the passage of a TBM. A third paper describes case studies of a tunnel project in which the behaviour of piled constructions was monitored when the TBM advanced underneath.

Selemetas, Standing & Mair (2005) present the results of a full-scale trial, which took place during the construction of CTRL in the UK. The tunnel consists of two bored tunnels with a diameter of 8 m. The first TBM (Up-Line) passed beneath the monitoring section one month prior to the second TBM (Down-Line). Four instrumented, driven cast-in-situ, reinforced concrete piles were installed above and at an offset from the two tunnels to be bored with an Earth Pressure Balance (EPB) shield. All four piles were loaded up to 50% of their ultimate bearing capacity. The ground conditions consist of 3 m of made ground overlying about 4.5 m of soft Alluvium of peat and silty clay.

Beneath this Alluvium lies 3.7 m of dense Terrace Gravels followed by 15 m of very stiff London Clay. The groundwater level is 4 m below the ground surface. The intention was to maintain these loads during tunnelling, which at the site of interest took place through London Clay. The profile of greenfield ground surface displacement was also measured. The paper discusses the resulting changes in the pile base load and the associated pile settlement due to tunnel construction. The conclusion of this paper confirms the earlier presented approach with the triangular zones, in which pile settlement can be correlated to greenfield ground surface settlement, (Kaalberg et al., 1999). In Figure 10 their zones of influence for (EPB) shield tunnelling in London Clay is presented. In this research special attention was paid to the distribution of shaft and base resistance and the impact of tunnelling on this distribution as two piles were "end bearing" piles and two "friction" piles. Piles with their bases above the tunnel (Zone A) showed a considerable reduction in their base loads during passage of the TBM. Piles with their bases in beside the tunnel (Zones B and C) experienced small changes in their base loads. The base loads of these piles showed a small increase with time due to the negative shaft friction.

Kaalberg et al. (2005) present an overview of the Dutch research on the impact of tunnelling on pile foundations in view of the North/South Line in Amsterdam that will be bored in the coming years. A part (3.9 km) of this line crosses the historic centre of Amsterdam. The line will pass beside more than 250.000 wooden and over 2000 concrete piles of the foundations of adjacent buildings. The research started with tests in the geotechnical centrifuge of GeoDelft and comprised numerical modelling and full-scale tests. The full-scale test was carried out at the Second Heinenoord tunnel, consisting of two tunnels (diameter of 8.3 m) bored with a slurry shield. The test had two objectives: to study the effects and risks of tunnelling near, beside and under pile foundations and to validate a 3D numerical model. This computer code was used to predict pile and (sub)surface settlements. Special attention was paid to distinguish between settlement of the soil layer around the pile toe and settlement caused by stress relieve around the pile toe due to the tunnelling. Especially this last reason, causing a permanent reduction of pile capacity was feared. The soil conditions of the test site are different from Amsterdam. In Amsterdam the upper 10–12 meters consists of very soft silty clays, peat and loose sand deposits. Underneath these soft layers a sand layer is found, in which most wooden piles are founded. At the test site only the first four meters consists of soft clay, overlying fine sand. To stimulate the Amsterdam subsoil conditions large clay columns have been constructed. Subsequently the piles were rammed through the clay column and into the sand layer. The pile loads are similar to Amsterdam pile configurations. In total 43 timber piles and 20 concrete piles were placed in the vicinity and above of the tunnel. For the first passage, the pile toes were located at a distance of 0.5 D to 2.5 D

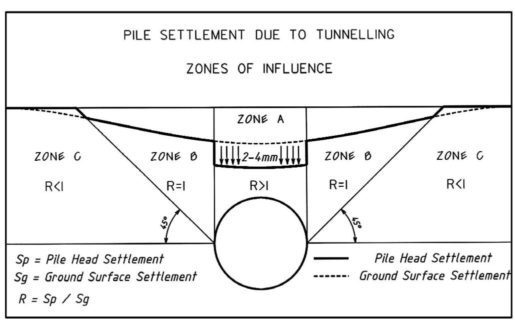

Figure 10. Zones of influence of piled settlement due to Earth Pressure Balance shield tunnelling in London Clay, (Selemetas et al., 2005).

from the tunnel tube (with diameter D). Most piles were loaded by sand-filled containers on top. Through a load-controlled jack, the weight of the load was transported to the piles. The monitoring programme contained: *i*. pile, surface and sub surface vertical and horizontal deformations; *ii.* stresses in the soil and at the pile shaft; *iii.* static pile load tests and CPTs before and after TBM-passage. The paper presents for the first and second passages the relation between surface and pile settlement, corresponding with a volume loss at surface of respectively between 1 and 2% during the first passage and 0.75% during second, see Figure 11. These results have led to the triangular zones as earlier presented by Kaalberg et al. (1999).

The measured changes in effective stress, after the second TBM passage show that despite the volume loss of 0.75% during this passage that the stresses increased considerably after the passage, probably due to effective tail grouting. Close to the tunnel an increase between 100 and 150 kPa was found. It was concluded from the stress measurements at the pile shafts at a distance of 0.5 D from the tunnel that also during passage the total and effective stresses at the pile shafs temporary increased. CPTs as well as static pile load tests before and after TBM passage confirmed that a relieve of stresses around the pile shaft and toes did not occur as the cone resistances and the pile capacities even increased slightly after TBM passage. Finally it was concluded that the surface settlements and pile settlements at the long term increased by 15%

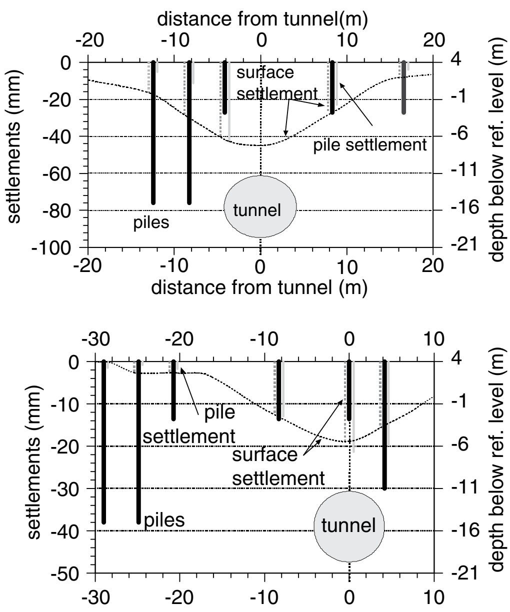

Figure 11. Surface and pile settlement at first (1st figure) and second (2nd figure) passage, (Kaalberg, et al., 2005).

compared with the immediate deformations after TBM passage.

Jacobsz et al. (2005) describe three case studies dealing with the construction of the tunnels for the Channel Tunnel Rail Link (CTRL) project in London, which gave the opportunity to obtain valuable information on the effects of tunnelling relatively close to piled bridge foundations. The paper briefly describes the assessments carried out prior to tunnelling under three bridges in London as well as monitoring results. The first bridge, Renwick Road Bridge, is supported on end-bearing piles, while the other two bridges, Ripple Road Flyover and the A406 Viaduct, are supported on friction piles. During the passage of the downline under the Renwick Road Bridge the volume loss was just above 1%, leading to surface settlements of about 20 mm and pile settlements between 15 and 18 mm. These settlements correspond with the triangular zone relating surface and pile settlements as shown above. The authors concluded, based on the prediction of sub surface settlements, that the pile behaviour of end bearing piles corresponds with the settlement of the layer in which the pile toes are installed.

A simplified cross-section of the Ripple Road Flyover bridge is presented in Figure 12. The vertical piles were of driven-cast-in situ construction, while the raking piles were bored.

The majority of the pile shafts are in the London Clay, which is overlain by approximately 4 m of Terrace Gravels. The surface is underlain by made ground varying in thickness between 1 m and 2 m. The clearance between pile base and tunnel lining was estimated to be approximately only 1 m. Mitigation works were carried out consisting of grouting around the piles within the Terrace Gravel, and additional grouting underneath the pile cap. With this solution the grouted piled bridge piers were expected to settle by the same amount as the ground surface.

The upline TBM resulted in a total settlement of between 8 mm and 10 mm, i.e. a volume loss of about

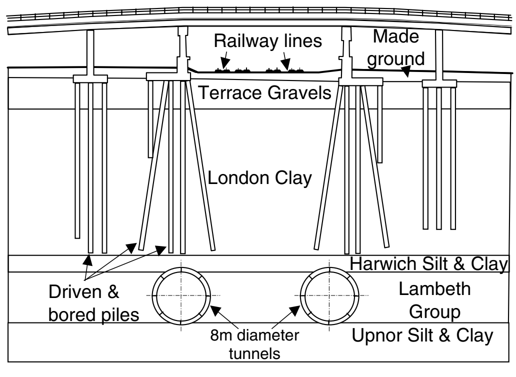

Figure 12. Cross-section Ripple Road Flyover, (Jacobs et al., 2005).

0.6%. The downline TBM passage caused approximately 11 mm of settlement just before the bridge, i.e. a volume loss of 0.8%. The face pressure was then increased, reducing the settlement to about 7 mm under the bridge. Approximately 3 mm of heave was observed ahead of the TBM face. The settlement of the bridge pier above the upline tunnel was about 8 mm and that of the pier above the downline tunnel 10 mm. These settlements are similar to the surface settlement.

From the three presented case histories the authors conclude the following: *i*. At small volume losses end-bearing piles settle by an amount equal to the greenfield settlement at the pile base *ii.* friction piles change the greenfield subsurface settlement profile and settle by an amount similar to the greenfield surface settlement *iii.* care must be taken to ensure that volume losses are kept small in areas where load cannot be distributed from the pile base to the shaft *iv.* assuming friction piles to deform with the surrounding soil provides a conservative approach and *v*.when assessing the effects of tunnelling on piled foundations a re-assessment of pile capacity should be carried out.

5 GENERAL CONCLUSIONS

The papers dealt with in this general report have rather different topics. It is therefore not possible to present overal conclusions that cover all papers. However, some general trends can be mentioned:

- The papers dealt with showed that there is some agreement how to deal with the response of structures to tunnelling. For structures without piles the theoretical work from Potts and Addenbrooke (1997) is (with some scatter) confirmed by measurements. Damage criteria for these buildings are presented. Work has started to include 3D effects.
- The influence of tunnelling on piled foundations is in several papers described with influence zones as the zones presented in Figure 10 of this report. Deviations are reported only for large volume losses and piles very close to the tunnel.
- Nowadays tunnels can be built with volume losses of less than 1%. It is shown in the papers that this presents possibilities to bore tunnels close to each other (at distances of only $0.35 \times D$ with D the diameter of the tunnel), to bridge foundations and crossing existing tube tunnels without disrupting the normal operation.

ACKNOWLEDGEMENTS

I want to express my gratitude to Mr. Thomas Viehofer and Mr. Adam Bezuijen both from GeoDelft who helped me respectively by reviewing part of the papers and by giving me useful advices. Finally the authors are gratefully acknowledged for their very interesting papers; it was a pleasure to study them.

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LIST OF PAPERS SUBJECT OF THIS GENERAL REPORT

Caporaletti, P., A. Burghignoli, and R.N. Taylor. Tunnelling-induced movements and effect of ground-structure interaction: centrifuge modelling

Franzius, J.N., D.M. Potts and J.B. Burland. Twist behaviour of buildings due to tunnel induced ground movements

Jacobs, S.W., K.H. Bowers & N.A. Moss. The effects of tunnelling on piled structures on the CTRL

Kaalberg, F.J., E.A.H. Teunissen, A.F. van Tol & and J.W. Bosch. Dutch research on the impact of shield tunnelling on pile foundations

Kwast, E.A. & G.P.C. van Oosterhuis. Measurements and evaluation of the influence of two bored tunnels at reduced distance (<0.5 D) in a homogenous sand layer

Kitiyodom, P., T. Matsumoto & K. Kawaguchi. Analyses of pile foundations subjected to ground movements induced by tunnelling

Lee, G.T.K. & C.W.W. Ng, Three-dimensional numerical simulation of tunnelling effects on an existing pile

Matsumoto, T., P. Kitiyodom & K. Kawaguchi. Three-dimensional analyses of piled raft foundations subjected to ground movements induced by tunnelling

Moss, N.A. & K.H. Bowers. The effect of new tunnel construction under existing metro tunnels

Netzel, H., Review of the limiting tensile strain method for predicting settlement induced building damage

Selemetas, D., J.R. Standing & R.J. Mair. The response of full-scale piles to tunnelling

Simic, D. Structure interaction effects on tunnelling induces settlements

Standing, J.R. & W.Y.M.T. Leung. Investigating stresses around tunnels and piles using photo-elasticity techniques

Surjadinata, J., J.P. Carter, T.S. Hull & H.G. Poulos. Analysis of effects of tunnelling on single piles

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Bored tunnels

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ABSTRACT: This General Report reviews a total of twenty five papers relating to bored tunnel construction, its influence on ground deformations and tunnel interaction with other geo-structures in the 5th International Symposium on Geotechnical Aspects of Underground Construction in Soft Ground. These papers may be broadly grouped under five sub-themes or headings: (a) case history and back-analysis, (b) study of interaction effects, (c) investigation of face stability, (d) assessment, measurement and analysis of ground deformations and, (e) influence of grout and muck properties on the construction of bored tunnels. This report is intended to highlight main features, key findings, and perhaps to raise queries and concerns of reviewed papers as appropriate. For details, readers are strongly encouraged to refer to the original papers.

1 INTRODUCTION

A total of twenty five papers, which cover a wide range of topics related to bored tunnels, are accepted and included in this session. Since the contents covered by the papers go far beyond just construction, the word "construction" is therefore dropped in the title of this General Report in order to avoid any confusion and ambiguity. Among the accepted papers, seven are from Asia, fourteen are from Europe and four are from the South and North America. However, none of these papers is from Australia and Africa. Some effort seems to be necessary to encourage contributions and active participations from these two continents in future.

According to technical contents, the papers submitted may be grouped under five sub-themes or sub-headings: (a) case history and back-analysis, (b) study of interaction effects, (c) investigation of face stability, (d) assessment, measurement and analysis of ground deformations and, (e) influence of grout and muck properties on the construction of bored tunnels. A list of papers grouped under each sub-theme is given in Table 1. A wide range of research methodologies such as field monitoring and centrifuge modelling and predictive tools including empirical method, numerical analysis, analytical and statistical approaches are used by various researchers.

This report does not intend to cover details of each paper but it highlights main features and key findings and raises queries and concerns of reviewed papers as appropriate. Readers are strongly encouraged to refer to the original papers for more details.

2 CASE HISTORY AND BACK-ANALYSIS

Autuori and Minec describe and discuss major geotechnical challenges at Groene Hart tunnel, which is part of the Trans-European high-speed line (HSL) connecting Amsterdam, Brussels and Paris. The Groene Hart tunnel is located at the Green Heart of Holland in the Netherlands. The tunnel was a 15 m diameter single tube TBM tunnel constructed in saturated loose sand under high ground water conditions. At some areas along the route, a layer of about 0.7 m thick peat was found at 8 m above the tunnel (see Figure 1). The stability of the tunnel cover and possible formation of local cracks in the peat were two major concerns for designers, in particular, the concern over possible connection of clean free water and polluted artesian water through local cracks, which might be induced as a result of excess pore pressure generated during the TBM construction. Comprehensive instrumentation including 13 settlement gauges, 2 extensometers and 19 pore pressure cells denoted by Ws in Figure 1 were installed to monitor ground and pore pressure responses. The measured field data were used to validate finite element analyses of the tunnelling process and to verify and justify any mitigation measure. The concluded mitigation measure was to increase vertical load at the peat/sand interface by placing backfill at the ground level (1 m high and 24 m wide).

de Queiroz et al. back-analyze K_o statistically using the Bayesian updating technique from measured ground movements. They use a non-linear isotropic hyperbolic model in their finite element analyses.

Table 1. A summary of accepted paper in the session of bored tunnels.

Title of paper	Author	Country
Sub-theme: Case history and back-analysis		
1. Large diameter tunnelling under polders	Autuori & Minec	France
2. Bayesian updating of tunnel performance for K_0 estimate of Santiago gravel	de Queiroz, del Roure & Negro Jr.	Brazil/Chile
3. Control of contaminated groundwater during tunnel excavation	Munfah & Butler	USA
4. Ground and lining responses during tunnelling in water-bearing permeable ground – 3D stress-pore pressure coupled analysis	Yoo & Kim	Korea
5. Geotechnical centrifuge tests to verify the long-term behaviour of a bored tunnel	Pachen, Brassinga & Bezuijen	Netherlands
6. Work and design of "Casting support tunnelling system using TBM" to unconsolidated soil with high groundwater level	Iida, Isogai, Chishiro, Ono, Koyama & Koizumi	Japan
7. Compressed air driving and monitored soft ground and groundwater behaviour	Quick & Meissner	Germany
8. Investigating variations in tunnelling volume loss – a case study	Standing & Burland	UK
Sub-theme: Study of interaction effects		
9. Centrifuge modelling of the effect of tunnelling on buried pipelines: mechanisms observed	Vorster, Mair, Soga & Klar	UK
10. Predicting the settlements above closely spaced triple tunnels constructed in soft ground	Chapman, Rogers & Hunt	UK
Sub-theme: Investigation of face stability		
11. Centrifuge experiments on stability of tunnel face in sandy ground	Oblozinsky & Kuwano	Canada/Japan
12. Effect of slurry clogging phenomena on the face stability of slurry-shield tunnels	Lee, Choi & Reddi	Korea/USA
13. Analytical stability models for tunnels in soil	Sozio	Brazil
Sub-theme: Assessment, measurement and analysis of ground deformations		
14. Settlement assessment of running tunnels – a generic approach	Harris & Franzius	UK
15. Settlement due to tunnelling on the CTRL London tunnels	Bowers & Moss	UK
16. Effect of driving parameter on ground surface movements: Channel Tunnel Rail Link Contract 220	Wongsaroj, Borghi, Soga, Mair, Sugiyama, Hagiwara, Minami & Bowers	UK/Japan
17. Settlements of HSL immersed tunnels	Hakkaart, Mortier & 't Hart	Netherlands
18. Prediction of Ground Settlement by Peck-Fujita and Numerical Methods	Rastbood, Shahriar, Khoshravan Azar & Rastbood	Iran
19. Prediction of shield tunnelling influences on ground deformation based on the construction process	Oota, Nishizawa, Hashimoto & Nagaya	Japan
20. Settlement behaviour of a shield tunnel constructed in subsiding reclaimed area	Komiya, Takiyama & Akagi	Japan
Sub-theme: Influence of grout and muck properties on construction		
21. Grout properties and their influence on back fill grouting	Bezuijen & Talmon	Netherlands
22. Grouting the tail void of bored tunnels: the role of hardening and consolidation of grouts	Talmon & Bezuijen	Netherlands
23. Pressure gradients and muck properties at the face of an EPB	Bezuijen, Talmon, Joustra & Grote	Netherlands
24. Influences of physical grout flow around bored tunnels	Lokhorst, Blom, Slenders & Kwast	Netherlands
25. Influence of hardened grout in shield tail on shield tunnelling performance	Sugimoto, Sramoon & Isasaki	Japan/ Thailand

It is obvious that their numerical predictions for statistical analyses are highly dependent on what constitutive model and parameters are used. It appears that model and parameter uncertainties are not considered in the Bayesian back-analysis.

Munfah and Butler describe control of contaminated groundwater during tunnel excavation for the largest current transportation project in the NY City. A number of tunnels in soft mixed ground were designed to use pressurized face TBM and the cut and cover technique in Queens and Manhattan areas, where soil and ground water were contaminated by inactive hazardous waste and ground water plumes (volatile organic compounds), respectively. To minimize

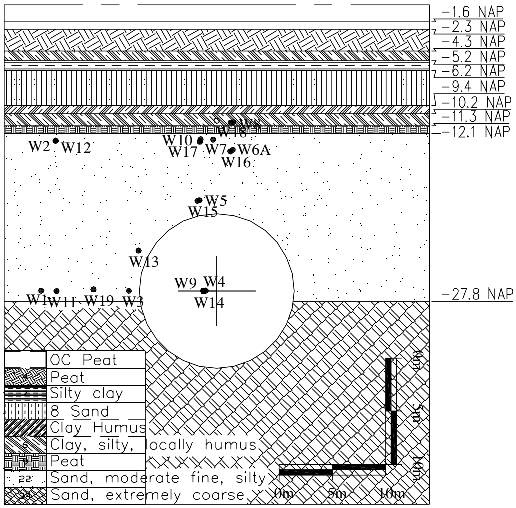

Figure 1. Geotechnical cross section at monitoring site (Autuori & Minec).

environmental impacts due to the construction of the project, the design solution was to minimize drawdown and movement of ground water by using a minimum of 760 mm thick cut-off slurry wall socketed into rock. An extensive hydrological modelling of the site and the ground water regime was developed and field monitoring was planned to provide immediate feedback to designers for verifying design assumptions and devising remedial measures if necessary.

Yoo and Kim present a series of three-dimensional stress-pore pressure fully coupled finite element parametric analyses to investigate the influence of relative permeability of lining to weathered granite (K_L/K_s) on ground and lining responses. Moreover, effects of a pre-grouting scheme were studied. The ground profile and tunnel section analyzed are shown in Figure 2. Decomposed granitic soil and weathered granitic rock were simulated by an elasto-plastic model with an extended Drucker-Prager failure criterion. A non-associated flow rule was adopted. It is not clear what the so-called "coupled" analysis was and why it was needed for the stiff decomposed soil and rock. Moreover, material properties adopted (see Table 2) are not fully justified and explained. No decomposition grades are given in the paper to assist readers to make independent assessments. It appears that some values, e.g., E, listed in the table are somewhat unusual when they are compared with laboratory and field measurements of decomposed granitic soil and rock in Hong Kong (Ng et al. 2000; Ng and Wang, 2001) and in Portugal (Viana da Fonseca et al. 1997).

In order to assist in design of two 6.5 m diameter bored tunnels in soft Holocene clay in Rotterdam, *Pachen et al.* carried out two centrifuge tests to investigate effects of soil consolidation on tunnel lining design in the long run. Figure 3 shows observed deformation of clay at the end of consolidation after applying 55 kPa of surcharge by raining sand in-flight.

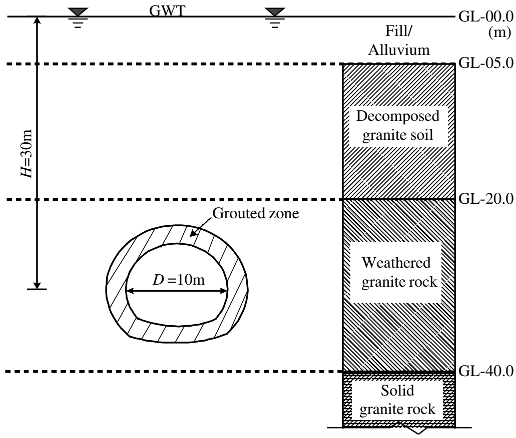

Figure 2. Ground profile for tunnelling condition analyzed (Yoo & Kim).

Table 2. Material properties used (Yoo & Kim).

Material	c' (kPa)	ϕ' (°)	ψ (°)	K (m/sec)	E (MPa)	K_0
Fill	5	30	20	2×10^{-6}	30	0.4
Decomposed granite soil	50	38	15	1×10^{-6}	70	0.5
Weathered granite rock	100	40	15	6×10^{-7}	100	0.5
Granite rock	200	45	15	1×10^{-9}	100	0.7
Grouted zone	200	50	10	6×10^{-9}	500	0.5
Shotcrete	–	–	–	1×10^{-8}	2000	–

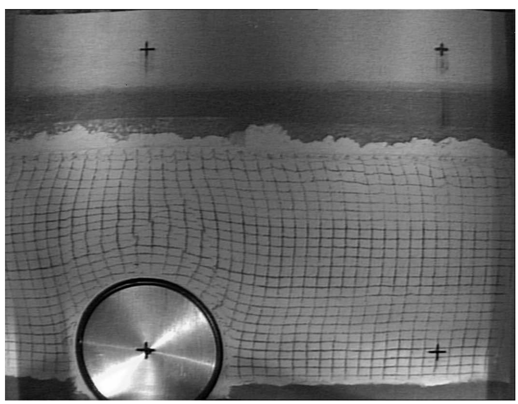

Figure 3. Deformation of clay at the end of consolidation after applying 55 kPa of sand in-flight (Pachen et al.).

Based on the results of centrifuge model tests and numerical simulations using an elasto-plastic hardening soil model, it is concluded that an analytical linear elastic approach using Airy's stress distribution function seems to be conservative for design. The authors also conclude that negative skin frictional force induced around the tunnel due to the consolidation of the soft clay appears to be independent of soil stiffness. For a given soil stratification and tunnel geometry, the negative skin frictional force induced on the tunnel depends on the surface surcharge loading only.

Iida et al. report an interesting case history involving re-design and construction of an 11.44 m diameter Sambongihara tunnel in mixed ground conditions subjected to high hydraulic pressures in Japan. For economical reasons, the initial design of the tunnel construction was to use the New Austrian Tunnelling Method (NATM). However, there were two tunnel face collapses during the construction of the initial 1.2 km long tunnel. A new hybrid construction system using a tunnel boring machine (TBM) to provide face stability first was designed for the remaining 3 km long tunnel. This new system called "casting support tunnelling system using TBM" or SENS in short is described in the paper.

Quick and Meissner introduce design and construction of Tunnel Offenbau for a new high speed railway line. Due to poor ground and unfavourable groundwater conditions, compressed air was used. Many construction photographs are illustrated but with too few explanations. Similarly, limited actual field monitoring data and explanations are provided.

Standing and Burland report a detailed investigation to explain why significant different volume losses of 1.2% and 3.3% were observed at the North and South of St James Park during the construction of the Jubilee Line Extension (JLE) in London. Three primary causes are identified: tunnelling method and control, differences in clay cover from past erosion, markedly differences in geotechnical characteristics – soil permeability in particular. They highlight the importance of understanding engineering geology and the need of close control of construction operations when tunnelling through clays containing water-bearing silt and sand partings.

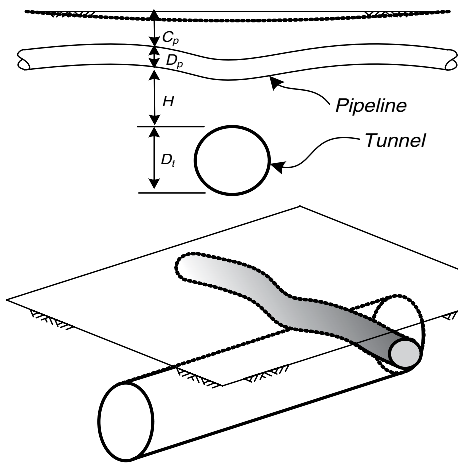

Figure 4. Schematic representation of the problem (Vorster et al.).

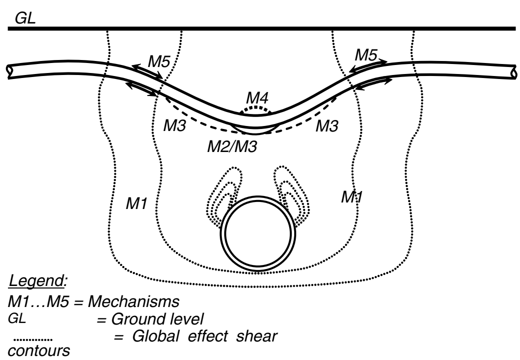

Figure 5. Schematic representation of five global and local mechanisms (Vorster et al.).

3 STUDY OF INTERACTION EFFECTS

Vorster et al. report and discuss nine very interesting and revealing centrifuge model tests, which were designed to investigate interaction mechanisms between buried pipeline and tunnel in sand. To simulate the problem in the field, three model pipelines of different orders of magnitude of bending stiffness were modelled at 75 g. Three configurations of normalised embedment depth, C_p/D_p, and normalized pipe-tunnel separation distance, H/D_t, were investigated, where C_p is the pipe embedment depth measured from the pipe crown, D_p is the pipe diameter, H is the pipe-tunnel separation distance and D_t is the tunnel diameter (refer to Figure 4). The three cases considered in the tests were: $C_p/D_p = 1$ (shallow pipe), $H/D_t = 2$; $C_p/D_p = 2$ (immediate pipe), $H/D_t = 1.5$; $C_p/D_p = 3$ (deep pipe), $H/D_t = 1$. Based on results of the centrifuge model tests, five interaction mechanisms (i.e., M1-M5) due to Global and Local effects are identified and explained. Figure 5 shows schematic representations of global and local interaction mechanisms (i.e., M1-M5) and how they might impact on pipeline behaviour. The authors consider that global effects (denoted by M1) represent shearing caused purely by contraction of tunnel cavity (i.e., as if a green field scenarios is considered, whereas local effects (i.e., M2-M4) result in local shear strains increase over and above those by global effects. These four local

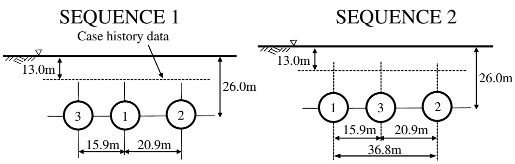

Figure 6. Construction sequence and geometry of triple tunnels (Chapman et al.).

mechanisms include gap formation (M2), decreased stability (M3), negative downdrag failure (M4) and longitudinal interaction (M5) as illustrated in Figure 5. According to the authors, the overall behaviour of pipeline is governed by a combination of global and local effects.

Based on previous work undertaken by the authors, *Chapman et al.* provide details of development of a modification factor to adapt empirical predictions using the Gaussian equation for predicting vertical displacements above twin tunnels. The authors describe, explain and illustrate that this previous method can be extended to incorporate the problem of triple tunnel analyses. Figure 6 shows two different construction sequences and geometry of triple tunnels considered in the paper. The results are compared with those from a two-dimensional non-linear finite element and measurements of sub-surface movements collected during the construction of the Heathrow Express Tunnels in London clay. The comparisons show that a considerable improvement is made when compared to the "unmodified" empirical method. The role of construction sequence and the subsequent settlement profiles are also examined.

Incidentally, Ng et al. (2005) and Tang and Ng (2005) carried out a series of three-dimensional (3D) fully coupled finite element analyses and investigated multiple tunnel interactions between large diameter parallel twin tunnels, hypothetically constructed in stiff London clay using the New Austrian Tunnelling Method. Special attention was paid to the influence of lagging distance between the twin tunnel excavated faces (L_T) and load transfer mechanisms between the two tunnels (see Figure 7). It is found that L_T has a stronger influence on the horizontal movement than the vertical movement of each tunnel and it significantly affects the shortening of the horizontal diameter of the tunnels. The change of pillar width appears to be an approximately linear function of L_T. The location of the maximum settlement is offset from the centreline of the pillar and the offset increases with a range of L_T values. Figure 8 shows the variations of the net offset (ΔL_x) of the maximum ground surface settlements with different L_T values at the plane strain section. It should be noted that a net offset is the offset (L_x) at any L_T value minus the offset at $L_T = 0$. The offset at $L_T = 0$ is taken as the reference since it is very

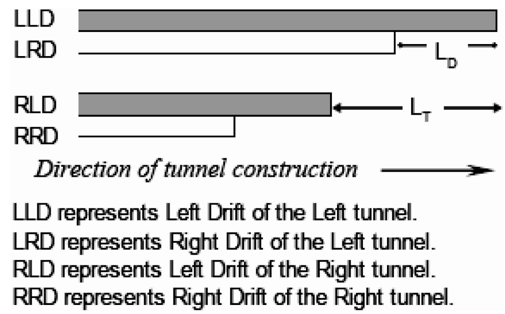

Figure 7. Notations and definitions of lagging length (LD) between the left and right drifts of the same tunnel and lagging distance (LT) between the left drifts of two parallel tunnels (Ng et al. 2004; Tang and Ng, 2005).

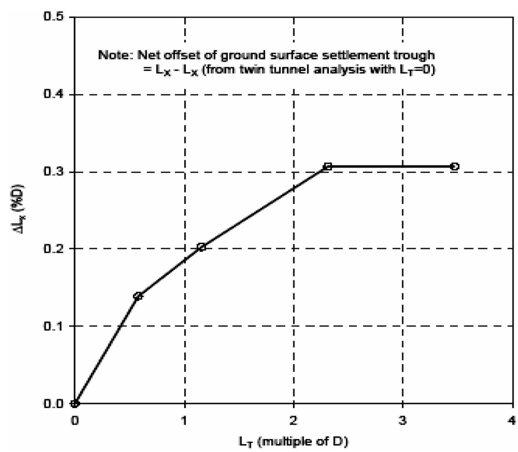

Figure 8. Relationship between net offset of ground surface settlement troughs (ΔLx) at the plane strain conditions and lagging distance LT in terms of multiple of tunnel diameter, D (Ng et al. 2004; Tang and Ng, 2005).

small and negligible. It can be seen that the net offset increases gradually with L_T, but it approaches a constant value (i.e., 0.32%D) at $L_T = 2.5D$ or larger. The amount of offset is an indication of the amount of load transfer associated with the construction of the neighbouring parallel tunnel. The smaller the offset, the more uniform the load shared between the two tunnels (Ng et al. 2004). For instance, in case of $L_T = 0$ when the offset is the smallest and most negligible, the load is shared almost uniformly between the two tunnels. On the other hand, for $L_T = 2.5D$ or larger, more load is taken by the leading tunnel (i.e., the left tunnel here) than the lagging one (the right tunnel here) as illustrated by different distributions of bending moments and axial forces in the tunnel linings for various L_T values (Tang and Ng, 2005).

Other detailed results such as pore water pressure distributions are given by Ng et al. (2004). In addition,

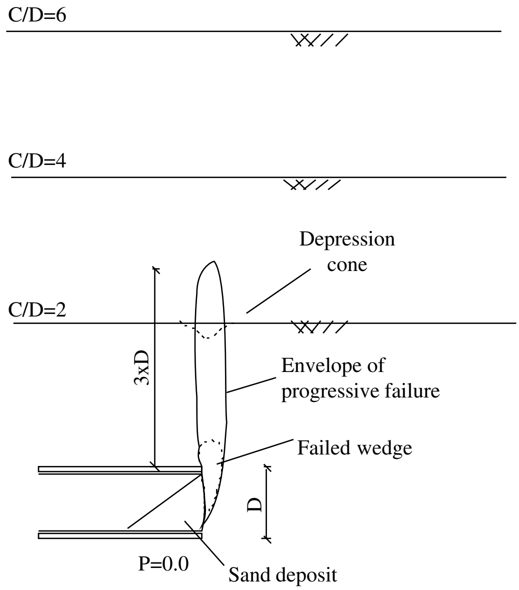

Figure 9. Sketches of tunnel geometry and failure development in PT series (Oblozinsky and Kuwano).

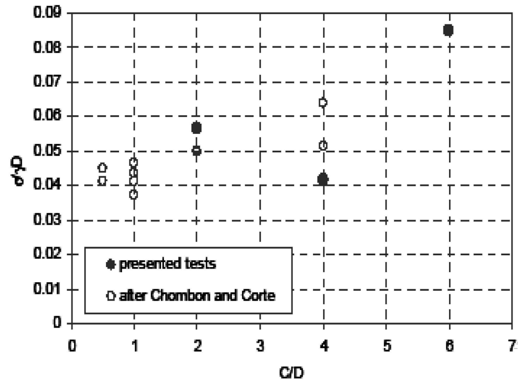

Figure 10. Normalised supporting pressure at failure against C/D ratio (Oblozinsky and Kuwano).

the influence of K_o and stiffness anisotropy on ground deformations and stress transfer mechanisms during an open face excavation are reported and discussed by Ng and Lee (2005).

4 INVESTIGATION OF FACE STABILITY

Oblozinsky and Kuwano report two series of centrifuge model tests to determine the minimum supporting pressure (σ) at a tunnel face, failure mode and extent of a failed zone in dry Toyura sand with relatively density ranging from 79% to 83%. Three C/D cases are considered, where C and D are depth of cover and diameter of tunnel, respectively (see Figure 9). Two different controlled techniques were used in the centrifuge tests to induce failures, i.e., air pressure control (PT series) and volumetric displacement control using water (DT series). Figure 10 shows measured normalised supporting pressure (σ/γD) at failure against C/D ratio. The test results by Chambon and Corté (1994) are also plotted in the figure for comparisons. Although there is scatter in the test results, it may be concluded that normalised supporting pressure increases with C/D ratio.

Lee et al. give details of a series of laboratory experiments for assessing and investigating rheological characteristics of slurry penetration, in light of the soil-filter clogging theory proposed by Reddi and Bonala (1997). Effects of clogging, particle transportation and deposition, slurry concentration and additives are considered in their experiments. By making use of the "membrane model" developed by Anagnostou and Kovari (1994) for assessing tunnel face stability of a slurry shield, the authors calculate face stability with their experimental results. It is found that the critical d_{10} which needs special additive is 0.75 mm. Moreover, a relationship is reported between the tunnel advance rate and the stability of working face due to slurry penetration.

Sozio presents two different three-dimensional analytical models for calculating stability conditions at a tunnel face. The first analytical model is derived from an upper solution, which is based on an integrated block system, formed by an upper block above tunnel crown bearing a lower wedge, which slides towards the tunnel face. Consideration is given for inclusion of water pressures acting on the block and wedge faces. The second model is based on a published lower bound solution, which is then modified by including radial body forces and seepage forces to simulate effects of gravity and flow both in spherical coordinates. The limitations of each analysis are outlined. Critical pressures determined for both models are compared with published analytical solutions and laboratory test results. More test data are called for by the author to verify these solutions.

5 ASSESSMENT, MEASUREMENT AND ANALYSIS OF GROUND DEFORMATIONS

Harris and Franzius describe details of a revised generic method for assessing potential building damage due to tunnelling in urban areas. This revised method is derived from an existing three-phase assessment system proposed by Mair et al. (1996) as shown in Figure 11. The authors consider that the current Phase 2 is inefficient since unnecessary individual reports are required for all buildings. They suggest that it would be better by applying Phase 2 assessments

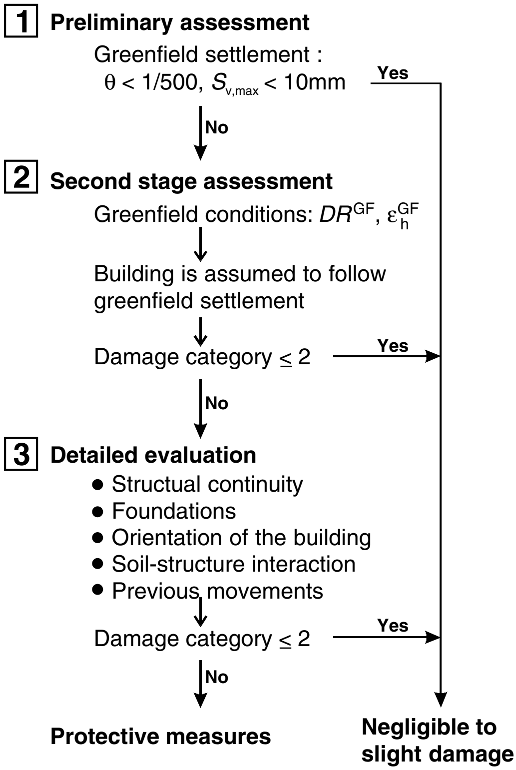

Figure 11. Three-phase assessment system (Mair et al. 1996; Harris and Franzius).

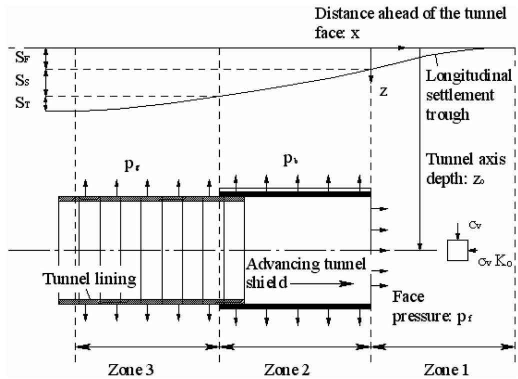

Figure 12. Definition of parameters (Wongsaroj et al.).

to representative sections taken through settlement contours from Phase 1. Along each section, a high number of different building geometries are analysed to determine the worst case, i.e., the maximum tensile strain. In this way, unnecessary reports can be avoided. Moreover, insight along the route alignment can be improved and potential problematic areas can be identified. The newly revised method is illustrated by a case study of a 2.7 km long section of running tunnels on the Crossrail project.

Bowers and Moss summarise observations of ground movements made during tunnelling for the Channel Tunnel Rail Link (CTRL) in London. The CTRL consists of two 8.15 m diameter and 18 km long bored tunnels which carry the underground railway from the Channel Tunnel to St Pancras in London. The tunnels were bored in mixed ground conditions including stiff clays, sands and gravels of the Lambeth Group, London clay, the Thanet Sand, Thames river terrace gravels, the Chalk and the recent estuarine deposits. Due to presence of many sensitive buildings and infrastructures in the urban areas, 2% volume loss was assumed initially for assessing buildings damage. Subsequently, less than 1% volume loss was specified for six EPB TBMs used for the boring. Based on the field measurements collected and analysed, it is found that the measured average volume loss is only 0.5%. The authors suggest that the use of 2% volume loss may be too conservative and it is more reasonable to use 1% volume loss for initial assessments in similar ground conditions.

Oota et al. present a summary of horizontal and vertical ground movements measured during tunnel construction using the EPB and SPB methods from nine construction sites in Japan. Key factors affecting ground deformations are identified and discussed. By carrying out a series of simplified 2D finite element analyses, the authors claim that they can simulate field observations closely. Obviously, the real challenge for any numerical simulation is to get Class-A predictions right, not Class-C predictions.

Wongsaroj et al. analyse the causes of ground movements induced by bored tunnelling using an earth pressure balance (EPB) machine for the Contract 220 of the Channel Tunnel Rail Link in London. They summarise five major causes of ground movements due to shield tunnelling: (i) stress relief at the tunnel face; (ii) over-cutting edge around the shield; (iii) closure of tail void behind the shield; (iv) deflection of lining and (v) consolidation of ground around the tunnel. The authors make an excellent attempt to link the observed ground movements to the key tunnel driving parameters of an EPB machine, such as face pressure as well as shield and tail void grouting pressure. As shown in Figure 12, zones of influence are identified and illustrated.

Dimensionless groups of TBM parameters are identified and expressed in Table 3, where D_e, D_s and D_{tv} are the diameters of the excavation, the shield and the lining, respectively, and where V_b, V_g, p_f, p_b and p_g have been averaged over the zones of influence defined in Figure 12. Although it is found difficult to establish simple correlations linking the settlement data to their causes, i.e., the driving parameters of the

Table 3. Dimensionless groups of TBM parameters (Wongsaroj et al.).

Machine parameter	Dimensionless group	Zone of influence
Face pressure: p_f	Face Pressure Ratio $FPR = p_f/\cdot z_o$	Zone 1
Bentonite injection pressure: p_b	Bentonite Pressure Ratio $BPR = p_b/\cdot z_o$	Zone 2
Bentonite injection volume: V_b	Overcut Filling Ratio $OFR = 4\cdot V_b/(D_e^2 - D_s^2)$	Zone 2
Grouting pressure: p_g	Grouting pressure ratio $GPR = p_b/\cdot z_o$	Zone 3
Grout injection volume: V_g	Tail void Filling Ratio $TFR = 4\cdot V_g/(D_{tv}^2 - D^2)$	Zone 3

Table 4. Some engineering properties of soil layers in borehole No. 9, Golestan park (Rastbood et al.).

Soil type	Permeability (m/day)	Young modulus (kN/m^2)	Poisson's ratio
CL	0.02	8000	0.4
SC-SM	0.6	15,000	0.25
CL	0.0815	8000	0.40
SP	0.727	26,000	0.35
CL	0.075	9000	0.35
SC-SM	3.46	7500	0.25
SP-SM	1.14	34,000	0.4
CL	0.065	14,000	0.4
SM	0.14	49,000	0.4

tunnelling machine, meaningful trends are sometimes observed between face, grouting pressures and the corresponding component of volume loss. Certainly this dimensionless analysis is an important attempt to characterise ground movements in a more scientific and systematic manner. With more similar work is carried out in future, a significant improvement in our Class-A predictions can be expected.

Hakkaart et al. provide details and explain predictions of settlement of two immersed tunnels in soft soils for the Dutch High Speed railway Line (HSL) linking Amsterdam and the Belgian border. Useful measured data are provided in the paper.

Rastbood et al. analyse and compare settlement predictions due to twin parallel tunnel constructions in cohessionless soils below the ground water table in Tabriz city. The diameter of the tunnels was 6.6 m and the shield tunnelling method was considered. Ground surface settlement troughs caused by the construction of these twin tunnels were predicted by two different methods: (i) the Peck-Fujita empirical method for single tunnel and by use of the principle of superposition and (ii) 2D Finite Element (FE) drained analysis

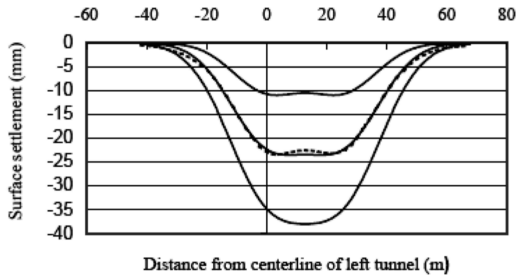

Figure 13. Comparisons of ground surface settlements for pillar width = 19 m, dashed and solid lines from numerical and empirical method, respectively (Rastbood et al.).

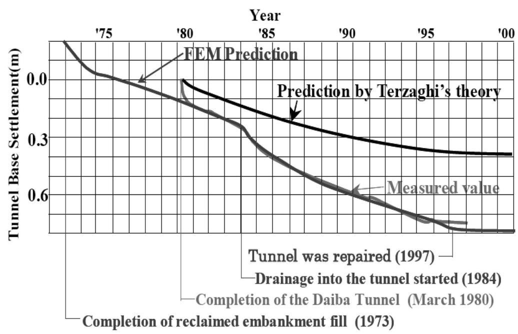

Figure 14. Comparison between the measured and computed values at the Daiba Eastbound tunnel (Komiya et al.).

using an elasto-plastic model with Mohr-Coulomb criterion. Some engineering properties of soil layers in borehole No. 9, Golestan park are given in Table 4. No explanation and justification is given for some unusual parameters chosen. Moreover, an important parameter, i.e., the coefficient of lateral earth pressure at rest, K_o, used in the FE analyses is not reported. Incredibly good agreement in computed values by these two methods is presented for six different pillar widths. Figure 13 show the comparisons of ground surface settlements for pillar width = 19 m. Considering the relatively simple predictive methods used and crude assumptions adopted, this excellent agreement between the predictions by the two methods is somewhat difficult to be understood.

Komiya et al. back-analyse settlement behaviour of the twin Daiba shield tunnels, which were constructed in consolidating soft alluvial clay strata beneath a reclaimed land. Three-dimensional coupled finite element analysis using the Cam clay model was carried out to compute settlements of the twin tunnels and to compare the computed values and field measurements over the last 20 years. Figure 14 shows the comparison between the measured and computed values at the Daiba Eastbound tunnel. It is obvious that an incredibly good agreement is achieved between the measured and back-analysed results.

6 INFLUENCE OF GROUT AND MUCK PROPERTIES ON CONSTRUCTION

Following on detailed explanations of six major functions of grout injected in tail void between soil and lining (Sharlaw et al. 2004) shown in Figure 15, *Bezuijen and Talmon* suggest that the grout can also provide sufficient resistance to overcome buoyancy force that occur in the first rings after a tunnelling operation using a TBM. The buoyancy force arises because the average density of lining and air that forms the tunnel is less than the density of the grout. This imbalance force can be eliminated by reducing grout density as well as by an increase of its yield stress. A simple but very important analytical model is derived by the authors to calculate loading on a lining along the axis of a tunnel as a function of mortar properties and possible movement of the lining. It is found that loading on lining depends on vertical pressure gradient of the grout and density of the tunnel (i.e., lining and air). By making use of measured results as shown in Figure 16, a simplified and useful calculation model is developed to estimate any induced shear force and bending moment of a lining theoretically. Some calculation examples are given.

Talmon and Bezuijen describe and report the development of a new and vital theoretical iterative finite-difference model for calculating grout pressure, deformation of grout-soil boundary, buoyancy force, fluid loss, thickness of grout cake and shear stress components in the grout behind a TBM. Account is given for rheology, hardening and consolidation of a grout, mechanical properties of soil and movement of tunnel lining behind the TBM. The development of this theoretical model appears to be somewhat based on a detailed study of field measurements of grout pressures by 14 pressure gauges at the Sophia Rail Tunnel shown in Figure 17. By taking into account of the rheology, hardening and consolidation of the grout, mechanical properties of the soil and movement of the tunnel lining behind the TBM at the Sophia Rail Tunnel, the calculated grout pressure distributions behind the TBM are shown in Figure 18. Other theoretical predictions by this newly developed model are also illustrated in the paper.

Sugimoto et al. describe key features and explain functions of an articulated shield as illustrated in Figure 19. Based on an articulated shield calculation model developed by Sugimoto and Sramoon (2002), the authors report an investigation of the influence of hardened grout in the shield tail on tunnel performance and they compare their Class-C predictions of directional deviations and shield velocity with field

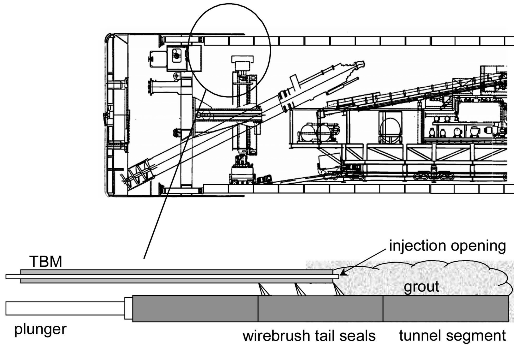

Figure 15. Sketch of TBM and details of injection system (Bezuijen and Talmon).

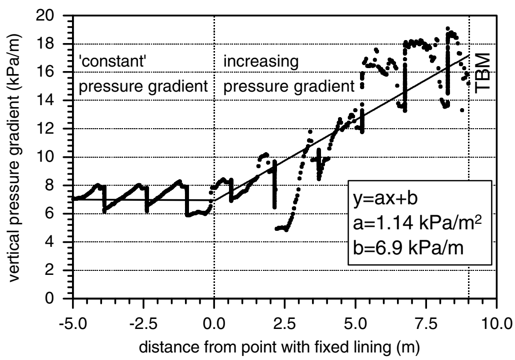

Figure 16. Example of gradient in the grout pressure as a function form the distance (0 on the X-axis represents the point where the lining is more or less fixed. The TBM is at 9 m). Results measured at Sophia Rail Tunnel (Bezuijen et al. 2004).

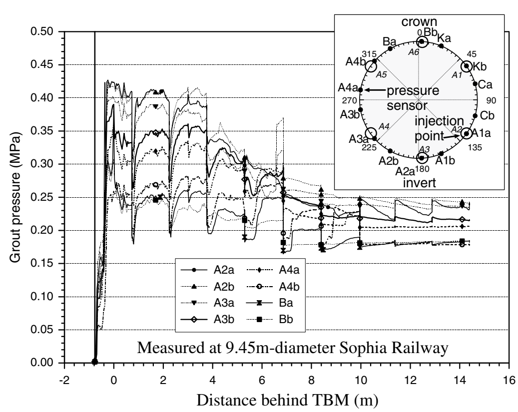

Figure 17. Results of field measurement on the grout pressure at Sophia Rail Tunnel (Bezuijen et al. 2004).

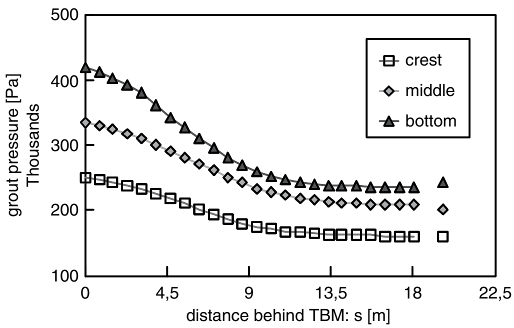

Figure 18. Calculated grout pressure for Sophia Rail Tunnel as a function of the distance behind the TBM (Talmon and Bezuijen).

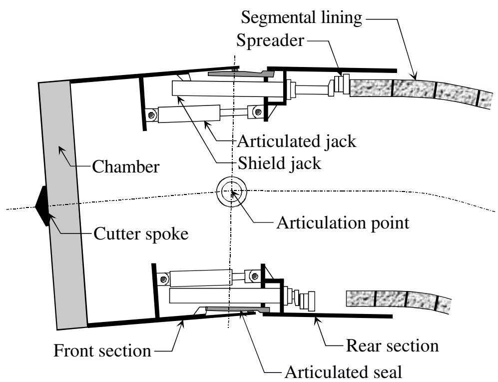

Figure 19. General feature of articulated shield (Sugimoto et al.).

measurements of an 8 m diameter articulated shield tunnel operation from a test site. The authors seem to be satisfied with their calculation model and predictions. It is concluded that the articulated angle of the shield is a predominant factor influencing shield behaviour. Moreover, the hardening of grout in the shield tail is one of key factors affecting shield behaviour, especially at rotation of the shield.

Bezuijen et al. describe, discuss and explain measurements of pressures, densities and degrees of saturation of mixture samples taken, pressure drops in the muck of an EPB-shield at the transition between the pressure chamber and screw conveyor of the EPB TBM during the boring of the Botlek Rail tunnel. The EPB TBM was operated in saturated sand with high ground water table. Foam was applied for soil conditioning. Figure 20 shows the locations of instruments in the TBM, where E and W denote total and pore water pressure gauges, respectively. Based on the field measurements at the EPB TBM, it is found that different from a slurry shield, there is no direct relation between measured vertical pressure gradient and the

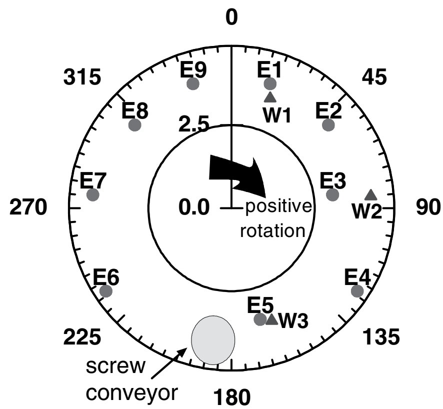

Figure 20. Position of instruments in the TBM looking from the tunnel to the TBM and definition of rotation (Bezuijen et al.).

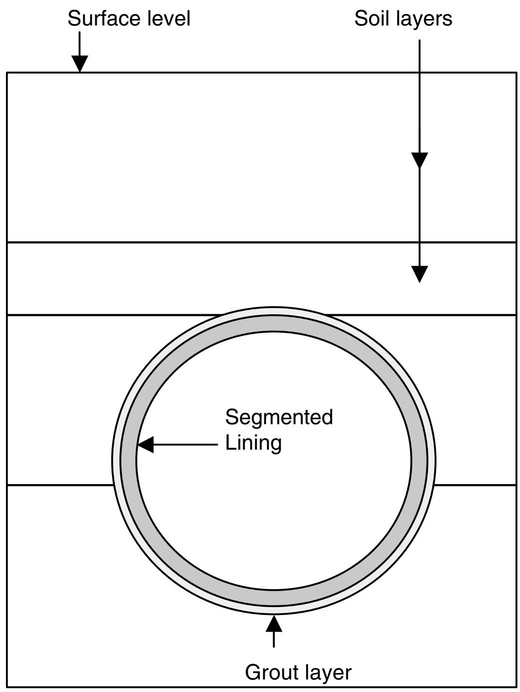

Figure 21. Schematic representation of the tunnel section modelled (Lokhorst et al.).

density of the muck in the pressure chamber. The pressures measured differ considerably at different rings. Densities of the mixture found indicate a porosity just higher than the maximum porosity of the sand. A pressure drop of approximately 100 kPa is found when the muck passes from the pressure chamber to the screw conveyor. While boring in sand with an EPB shield, an important function of the foam is to increase the porosity of the sand to such a value that deformation is possible, without or with only limited grain stresses. This is different from boring in clay where the lubricating is more important (Mair et al. 2003).

Lokhorst et al. report a series of 2D FEM analyses to study soil-grout-lining interaction of a TBM (see Figure 21). They used a commercial software called ANSYS in their analyses. Soil and grout were simulated by the Drucker-Prager and an isochoric visco-plastic models, respectively. At the soil-grout interface and grout-lining interface, cohesion contact and sliding capabilities were specified in the analyses. It is claimed that good agreement is obtained between back-analysed and measured radial grout pressures at the 14.5 m diameter Green Heart Tunnel in the Netherlands. However, no input parameters are justified and given in the paper. The authors are strongly encouraged to calibrate their numerical predictions with computed results by the theoretical model developed by Talmon and Bezuijen presented in this Symposium.

7 CONCLUDING REMARKS

A total of twenty five papers have been reviewed in this report. Among the reviewed papers, seven are from Asia, fourteen are from Europe and four are from the South and North America. However, none of these papers is from Australia and Africa. Some effort seems to be necessary to encourage contributions and active participations from these two continents in the future.

Some papers presented in this session provide invaluable and excellent case histories involving bored tunnel design, construction and field measurements. There is no doubt that readers can benefit from them significantly.

It is evident that there has been an increasing interest in the study of interaction effects and an improved understanding of pipeline-tunnel and tunnel-tunnel interaction has been gained.

A new and excellent theoretical model has been developed for predicting grout pressure, deformation of grout-soil boundary, buoyancy force, fluid loss, thickness of grout cake and shear stress components in the grout behind a TBM. The rheology, hardening and consolidation of a grout, mechanical properties of soil and movement of a tunnel lining behind a TBM can be considered. Significant improvement has been made in understanding of grout and muck behaviour. Very encouraging results have been obtained between predictions and field measurements.

There is an urgent need to call for Class-A predictions and explanations of any discrepancy found between Class-A predictions and field measurements. Well-tuned Class-C predictions provide limited values only. Caution should be given to any geotechnical parameter such as K_o deduced from Class-C predictions that can be obtained from many possible combinations of input parameters.

ACKNOWLEDGEMENTS

The author would like to thank his research students, Messrs Zhou Zheng Bing, Robin and Chen Rui, for their assistance in formatting the paper.

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Geotechnical Aspects of Underground Construction in Soft Ground – Bakker et al (eds)

Mitigating measures for underground construction in soft ground

General Report for the 3rd session: Mitigating measures

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ABSTRACT: This general report reviews and summarises 15 papers submitted to the session on Mitigating measures. An asset management system for underground structures is introduced to help infrastructure owners or clients make decisions about construction and repair. The significance of reducing tunnelling- or excavation-induced displacement and instability within soft ground is described.

1 INTRODUCTION

In my previous General Report on TBM and Shield Tunnelling session during IS-Tokyo 99, the submitted papers were reviewed from the standpoint of risk management system (Akagi, 1999). The risk management cycle consists of the following four stages, as shown in Fig.1:

(1) Listing of risk factors
(2) Risk analyses
(3) Countermeasures against risk factors
(4) Assessment of the countermeasures.

Mitigation is the process of making something milder or less intense or severe. Therefore, mitigating measures are equivalent to "countermeasures" in the articulated risk management system.

The assessment of the countermeasures adopted shall be performed from both technical and economical points of view, the latter assessing the costs needed to conduct the mitigating measures, including those prior to, during and after the underground construction. Underground structures become aged during their service and require repairs; therefore, the cost of mitigating risks during the life cycle of the underground structure shall be taken into account in the engineering construction project. The economic assessment may be done by the cost versus benefit basis. Unless the countermeasures are appropriate, other alternative mitigating measures are examined and assessed. That is the risk management cycle.

However, the risk management system provides insufficient information for the engineering project client or its owner to make the decision to go ahead with construction, particularly in the case of repairing an existing underground structure. The project clients need a reason to conduct the repairing construction. This reason must be provided by an asset management system.

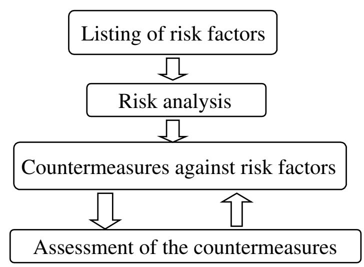

Figure 1. Risk management cycle.

2 ASSET MANAGEMENT SYSTEM FOR UNDERGROUND STRUCTURE

Asset management is a term originally used in the management of property and stock. The purpose of an asset management system is to evaluate existing property and stock on a monetary basis, and it is employed by property owners to make decisions regarding their investments.

An asset management system can be used to manage infrastructure, such as transportation systems, water and energy supply systems and communication network systems, including underground structures. The value of the existing aged underground structure is evaluated by the price of its initial construction or its reconstruction cost reduced by the lost value during its usage, as indicated in Eq. (1).

$$V = V_0(1 - \frac{t}{T}) \tag{1}$$

in which V is the current value of the existing aged structure, V_0 is its initial construction or reconstruction cost, T is its life span, and t is the time elapsed since its initial construction or reconstruction.

The infrastructure owner is able to make a decision about repairs based on a comparison between the cost of repairing and the value of the existing infrastructure. In other words, if geotechnical engineers have conducted an asset management analyses, they will be able to reasonably persuade the financial management section of the client or the owner to invest in countermeasures for avoiding geotechnical risk in an underground structure.

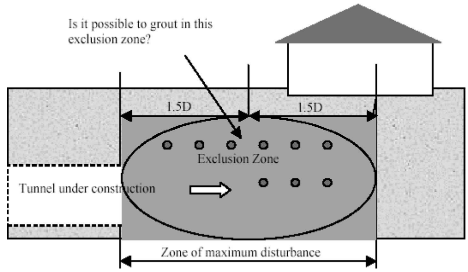

Figure 2. Grouting in exclusion zone in compensation grouting.

3 PAPER REVIEW AND REPORTER'S COMMENTS

The paper review and the reporter's comments are described below in the order of the initial paper number.

Cheong and Soga (No.23) present results on their investigation, via a small-scale experiment, of the effects of underground excavation on compensation grouting during tunnelling, as shown in Fig.2. One risk issue is the compensation efficiency in view of the volume change of the clay around the tunnel excavation. Another is the tunnel face instability due to the acting compensation grouting pressure close to the tunnel face. It seems to be difficult to control the fracture direction developed within the clay due to the compensation grouting in the field.

Chapman, Chan and Ahn (No.43) propose a method of predicting ground displacement due to compensation grouting during the construction of the Heathrow Airport Express tunnels, as indicated in Fig.3, using a Gaussian curve fitting procedure. This method can be employed in the risk analysis. The central risk issues include settlement, rotation and distortion of the existing tunnel structure. How can the rotation and distortion of the tunnel be predicted using the proposed semi-empirical method?

Agral, Ashkhmen, Korolev and Pronia (No.46) show a simple case history of a trial injection of a fine-disperse binding material suspension to improve an underground area, where a highway tunnel crosses a subway tunnel in Moscow. The target of the soil improvement was not indicated in the paper, and the cost of injection should be taken into account.

Simic (No.49) conducts a systematic approach to the evaluation of the interaction between super-structures and tunnelling-induced settlement. The procedure provides a useful tool for risk analysis of the damage to super-structures from tunnelling-induced subsidence, as shown in Fig. 4.

Bilotta, Russo and Viggiani (No.57) perform plane strain finite element analyses on the effects of a diaphragm wall embedment between a shallow tunnel and an existing structure, as shown in Fig.5. The barrier wall is shown to be deepened at least at the tunnel invert level. Useful information for risk assessment of the countermeasures is provided. Although the countermeasures to protect the existing structure are important, the control of the tunnel face stability and the reduction of the tunnelling induced settlement are considered to be much more significant.

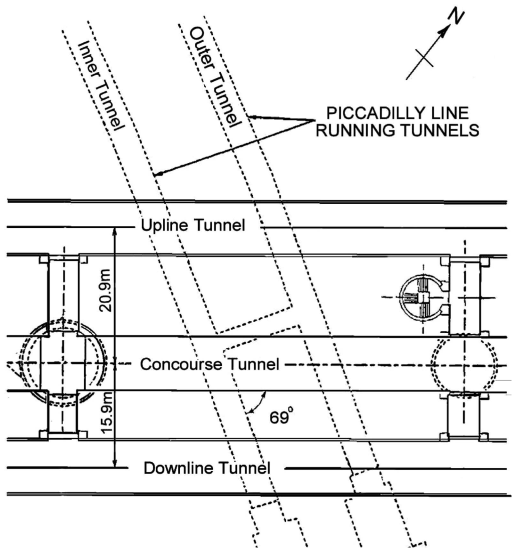

Figure 3. Layouts of the piccadilly line tunnels and the Heathrow Airport Express tunnels.

Stadelmann (No. 60) details the selection process for the construction method of the Zurich motorway tunnel in soft ground with a high water table, as demonstrated in Fig.6. The work includes a list of the geotechnical hazards involved in the construction and the numerical simulation of the mechanical behaviour of the soft ground due to the tunnelling. Comparison among the possible construction methods with appropriate auxiliary measures was carried out from the viewpoint of risk and construction costs. This is a good

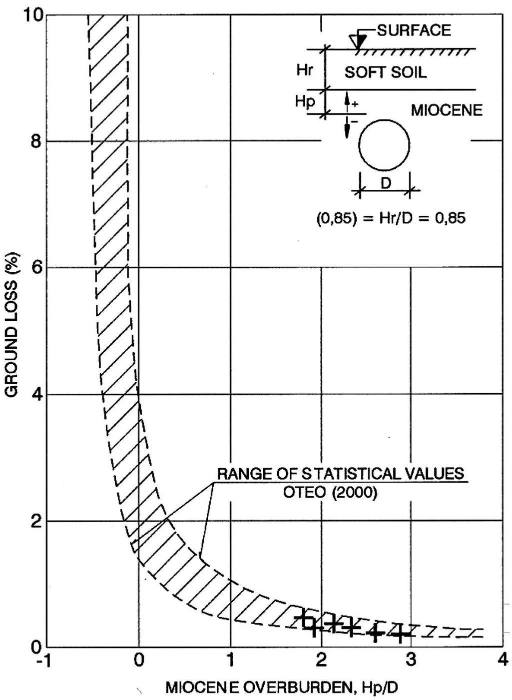

Figure 4. Ground loss measured in green filed trough.

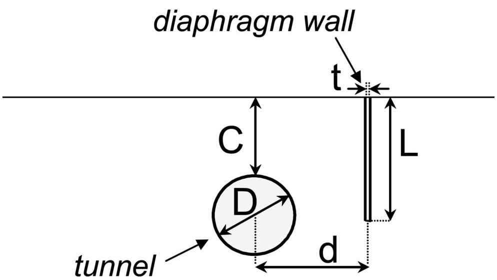

Figure 5. Sketch of the model used in the simulation.

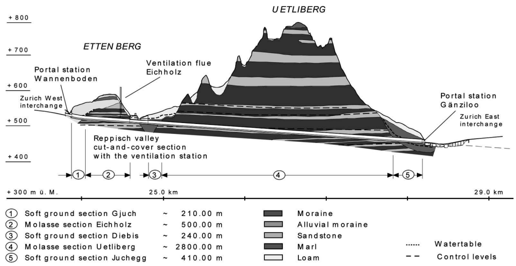

Figure 6. Longitudinal profile along the tunnel.

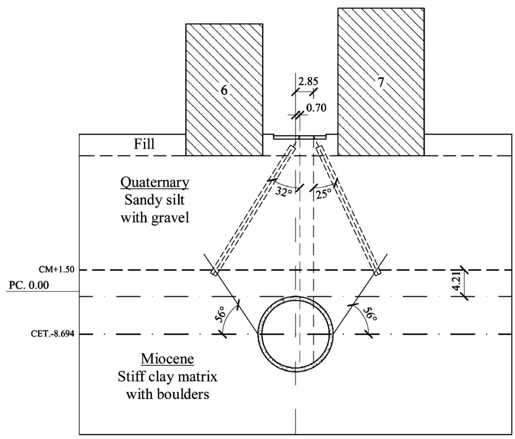

Figure 7. Cross section of jet grout columns for ground movement control.

example of risk management-based selection of tunnelling construction methods. An actual case history of this tunnel construction is expected to be reported.

Gens, Di Mariano, Gesto and Schwarz (No.66) report case histories of ground movement control in the construction of a new metro line in Barcelona using three types of methods: (i) the installation of a jet grouted column screen (Fig. 7), (ii) compensation grouting, and (iii) structural jacking. The measurement results of the vertical displacement during the tunnelling seemed to be determined by the tunnel face stability and the EPB (Earth Pressure Balanced)-type TBM (Tunnel Boring Machine) tunnelling-induced ground movement. Although the various auxiliary construction methods during under-ground excavation are sure to be effective with a proper application, the appropriate tunnelling techniques using EPB-type TBM within alluvial sand and gravel or clay with boulders are crucial. The tunnelling technique controls the earth pressure acting on the cutting face of the TBM, the excavated soil volume, and the tail void grouting.

Pachen, de Groot and Meijers (No.69) indicate the geotechnical risk during slurry-type TBM tunnelling just below the existing old railway in Rotterdam, as shown in Fig. 8. The potential of the liquefaction of the saturated loose sand beneath the existing embankment was demonstrated from undrained triaxial tests. Countermeasures against the liquefaction were chosen from the viewpoints of risk reduction, interruption of rail traffic and cost. An actual case history using the adopted countermeasures shall be reported.

Haß and Schäfers (No.73) focus on the artificial ground freezing method as a countermeasure in underground construction, specifically against collapse and excessive ground movement, as demonstrated in Fig. 9. Good case histories of artificial ground freezing technology development and its successful applications in Europe, 3 German cases and 1 case in Netherlands are presented. Special attention is needed for the application of the ground freezing method to soft clay.

Christiaens, Hemerijckx and Vereerstraeten (No.88) present a successful case history of slurry-type TBM tunnelling in dense sand under the old Antwerp city centre, as shown in Fig.10. The countermeasures adopted were a pipe-jacked roofing method, a jacking method with sheeted trench and a grout injection method. Detailed information about slurry-type TBM driving processes shall be provided, i.e. the magnitude of slurry pressure, the established excavation volume and the tail void grouting.

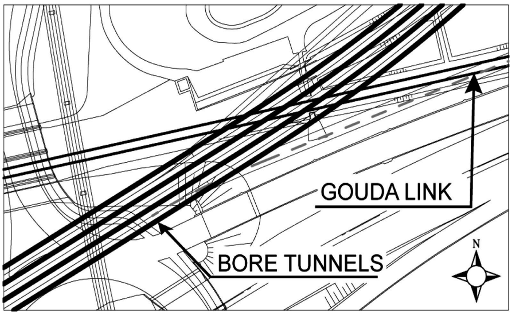

Figure 8. Overview railway crossing gouda link.

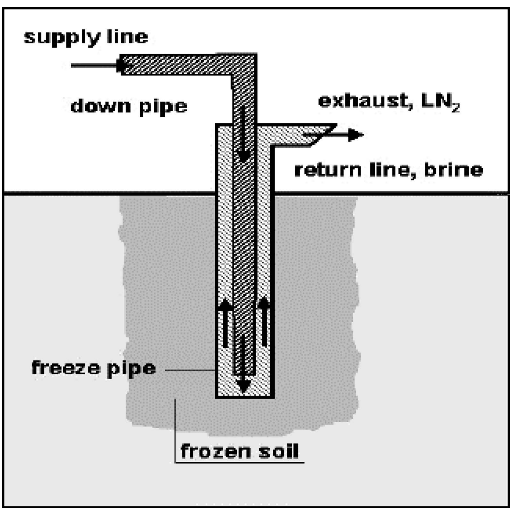

Figure 9. Ground freezing principle.

Spasojevic, Mair and Gumbel (No.96) introduce centrifuge model test results on the interaction between an old deteriorating pipe lined with close-fitting polymeric liners and the surrounding soil, as indicated in Fig. 11. A gap in knowledge exists between the development of the trenchless renovation technique and design practice. The information gained from their series of centrifuge models of soil load transfer should bring progress to rational design approaches. The actual event is expected to be more complicated than in the centrifuge modelling, i.e. the effects of underground water and the variations of backfill soil properties will come into play. The cost aspect of the rehabilitation will also be reported.

Chiriotti, Avagnina, Grasso and Tripoli (No.106) report a successful case history of EPB-type TBM tunnelling in a weathered rock mass and granular residual soil just below historically sensitive buildings, in Porto. The countermeasures adopted included grout injection prior to the tunnelling and compensation grouting during and after the TBM driving,

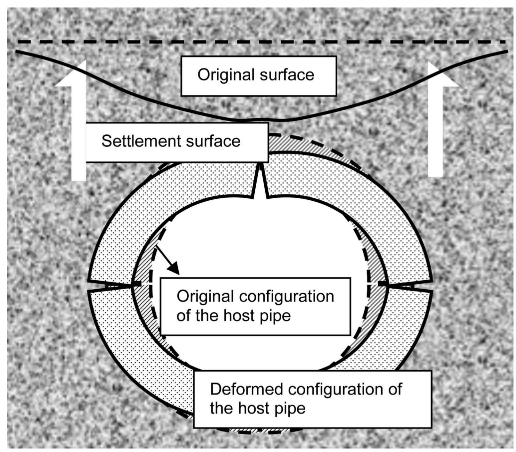

Figure 11. Stress distribution accompanying deterioration of the host pipe.

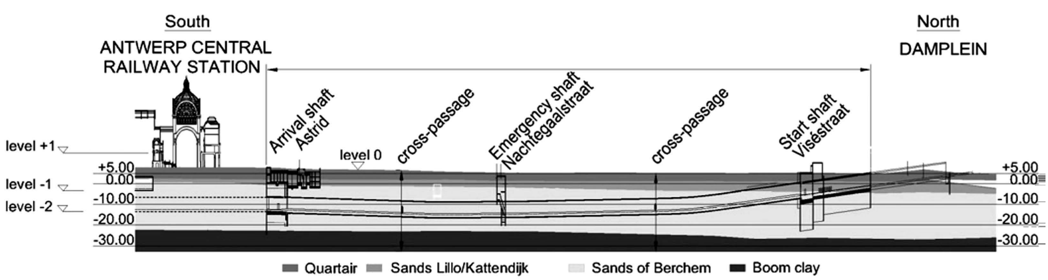

Figure 10. Longitudinal profile of the tunnel.

as demonstrated in Fig.12. Sufficient investigation of the EPB-type TBM driving technique within poor soil conditions was conducted to maintain tunnel face stability and reduce the ground settlement. Information on the costs required to conduct the countermeasures, including the monitoring and the GIS system, shall be indicated.

Petrukhin, Shuljatjev and Mozgacheva (No.121) demonstrate a vertical geotechnical barrier erected by compensation grouting, in the case of diaphragm wall excavation in soft ground, as shown in Fig.13. Although the authors claim the effectiveness of the vertical geotechnical barrier, the reduction of the ground displacement due to a diaphragm wall excavation seems to be much more crucial.

Shirlaw, Boone, Sugden and Peach (No.123) clearly point out the importance of managing risk through the control and the specification of EPB-type TBM tunnelling, including the shape of the disc cutter (Fig. 14). The proposed risk management system covers site

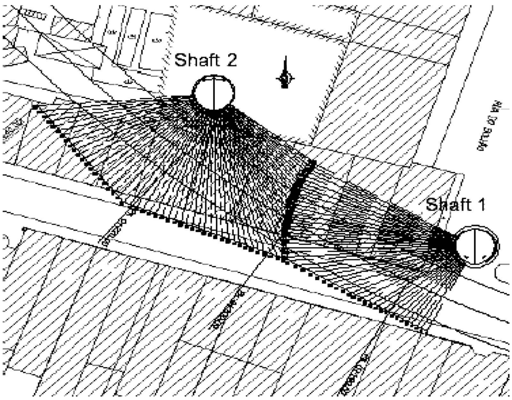

Figure 12. Plan of the sub-horizontal grout injection.

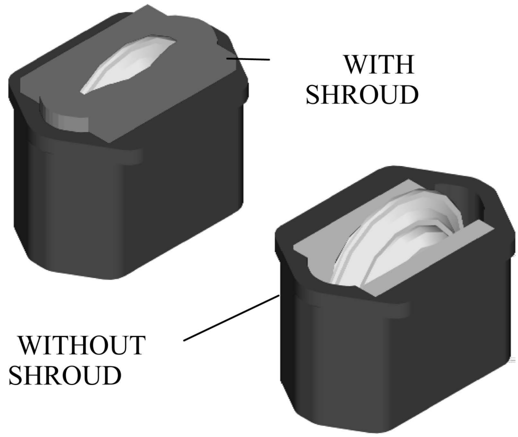

Figure 14. Shape of the disc cutter.

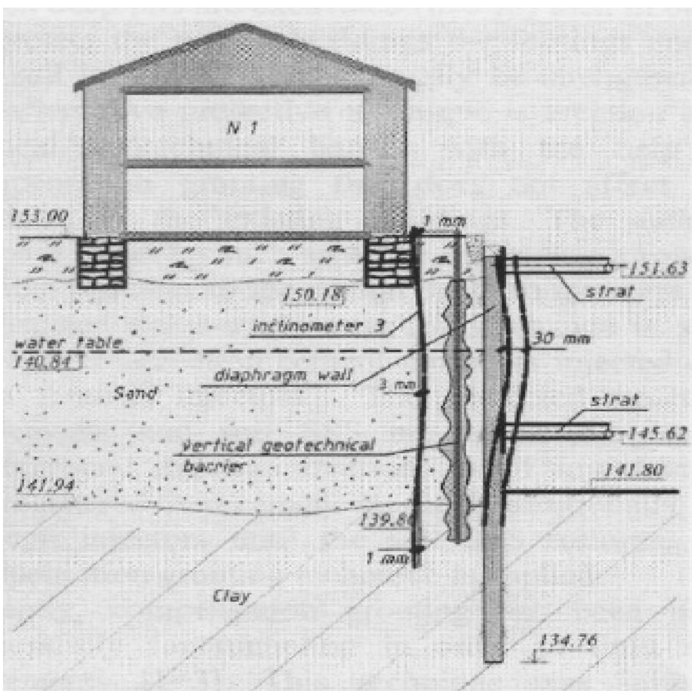

Figure 13. Excavation profile near building N1.

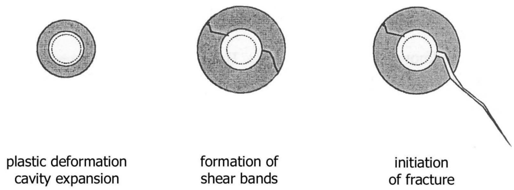

Figure 15. Hydraulic fracture initiation in sand.

investigation, specification of TBM, planning for tunnelling, and expert supervision. Although the countermeasures to protect the existing structure from damage using compensation grouting are important, the reduction of the risk source through successful control of the TBM driving and the excavation themselves is more crucial.

te Grotenhuis, van Tol, Haasnoot and Bezuijen (No. 128) demonstrate the modelling of fracture grouting in sand, as indicated in Fig.15. Their fracture model reproduces the fracture propagation process, the yield and friction characteristics of the grout and the bleeding of the injected grout. The validity of the modelling shall be demonstrated from experimental evidence.

4 CONCLUDING REMARKS

This general report reviews and summarises 15 papers submitted to the session on mitigation measures.

First, an asset management system for underground structures was introduced for infrastructure owners or clients making the decision whether to conduct construction and repair. This kind of approach is crucial under the recent tightening of budgets for public infrastructure. More information on the cost of the mitigating measures for underground construction should be provided.

Another issue is the significance of reducing tunnelling and excavation-induced displacement and instability within soft ground. Various mitigating measures for underground construction were reported in this session, i.e. (i) the installation of a jet-grouted column screen, (ii) compensation grouting and (iii) structural jacking, as indicated in *Gens et al*. However, the effects of the countermeasures on the reduction of damage due to ground displacement and instability are completely dependent on the tunnelling and excavation techniques. Unless the tunnel face stability is maintained and the tunnelling-induced settlement is properly controlled, as described by *Shirlaw et al.*, the countermeasures lose their objective.

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Chapman, D.N., A.H.C. Chan and S.K. Ahn. *Using a semi-empirical method to estimate the ground displacements caused by grouting operations*.

Cheong, M.T. and K.Soga. *Influence of underground excavation on compensation grouting in clays; small-scale laboratory experiments*.

Chiriotti, E., N. Avagnina and P. Grasso. *Compensation grouting for TBM tunnelling beneath shallow-cover*.

Christiaens, M., E. Hemerijckx and J.-C. Vereerstraeten. *Tunnelling under the city centre of Antwerp. A new underground railway link for the HSL Paris-Brussels-Amsterdam*.

Gens, A., A. Di Mariano, J.M. Gesto and H.Schwarz. *Ground movement control in the construction of a new metro line in Barcelona*.

Haß, H. and P. Schäfers. *Application of ground freezing for underground construction in soft ground*.

Pachen, M.A., M.B. de Groot and P. Meijers. *Crossing a railway embankment of loose packed sand with a shield tunnel*.

Petrukhin, V.P., O.A. Shuljatjev and O.A. Mozgacheva. *Vertical geotechnical barrier erected by compensation grouting*.

Shirlaw, J.N., S. Boone, N.B. Sugden and A. Peach. *Controlling the risk of sinkholes over EPB driven tunnels – a client perspective*.

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te Grotenhuis, R., A.F. van Tol, J.K. Haasnoot and A. Bezuijen. *Fracture grouting; improving the grouting efficiency in sand*.

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Numerical analysis of tunnels and deep excavations

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ABSTRACT: This report covers 33 papers submitted to the session Numerical Analysis of Tunnels and Deep Excavations. The majority of papers (26) addresses aspects related to tunnelling, both NATM and bored tunnels. As compared to previous conferences analysis of interaction between tunnel construction and surrounding structures such as buildings and pile foundations seems to have become more important. Only about 30% of the papers include actual field data but some additional papers cover practical aspects of tunnel construction. In this report the key points of each contribution are summarized. Finally some concluding remarks on the role of numerical modelling in underground construction in soft ground are added from a personal point of view.

1 INTRODUCTION

Numerical modelling by means of finite element, finite difference, boundary element and discrete element methods have become a standard tool for assessing the serviceability limit state (SLS) and, at least to some extent, the ultimate limit state (ULS) of geotechnical structures in soft ground. However, a gap between research and practice can still be observed. In the former large 3D models with sophisticated constitutive equations are frequently employed whereas in the latter simple elasto-perfectly plastic constitutive models are applied in (2D) analyses in most cases. That this trend has only slightly changed in recent years towards more advanced analysis in practice is confirmed by the papers presented to this symposium.

The question arising from this is obviously whether results obtained from relatively simple analysis are meaningful at all or whether sophisticated models concentrate on issues which are not of great importance in practice. Probably the answer is somewhere in between and depends strongly on the goal of the analysis in each particular case. However, one should always keep in mind that numerical models (and in fact all models in civil engineering but in particular in geotechnical engineering) are *models* and do not represent *reality* whatever complexity is introduced.

2 CLASSIFICATION OF SUBMITTED PAPERS

33 papers have been submitted to this session, 26 dealing with problems relating to tunnelling and 7 addressing deep excavations.

Out of the 26 tunnel papers 8 have investigated the influence of tunnel construction on existing structures such as buildings and piles, 6 papers provide (limited) comparison with in situ performance. Only 1 paper involves model tests. Some papers specifically deal with problems related to bored tunnels highlighting the increasing importance of numerical analysis in this field.

3 of the deep excavation papers include field measurements; again only 1 contribution makes use of model test data. 1 paper presents data from a comprehensive data base where measured and calculated displacements are compiled for a large number of deep excavation problems.

In the following section the most important aspect raised in each of the papers will be briefly summarized. An attempt is made to group them according to the main topic of the papers but of course there is some overlap between the different categories chosen.

3 KEY POINTS OF PAPERS – TUNNELLING

3.1 *Practical aspects dominating*

Stability analysis of the marly formation around a tunnel in Iran by Fahimifar & Soroush: describes the analysis of a tunnel excavation for a highway in the south part of Iran. Analytical calculations based on ground-support interaction curves as proposed by Hoek & Brown (1980) are used as well as numerical analysis employing the discontinuum code UDEC. The preliminary findings from the simple analytical analysis were confirmed by the numerical analysis, namely that a 200 mm shotcrete lining and rock bolts are not sufficient for stabilizing the tunnel but additional lattice girders are required.

Calculating GRC for tunnels supported by grouted rock bolts by Palassi & Qoreishi: ground reaction curves based on numerical analysis using the finite difference code FLAC are presented. Results are shown for a simplified example and comparison is made with analytical approaches suggested by Indraranta & Kaiser (1990) and Zakariaee (2003) whereas better agreement was achieved with the latter.

Design of initial support system of Isfahan twin tunnels in soft ground by Shahriar, Rastbood & Azar: the influence of the thickness of the shotcrete lining on the factors of safety of tunnels is investigated but unfortunately no details are given on how this factor of safety is obtained from the numerical analysis.

Soil-structure interaction and its influence on displacements induced by tunnel excavations by Chissolucombe, Assis & Farias: a real situation encountered during construction of the Brasilia Metro is analysed where a fuel station founded on piles suffered some damage due to tunnel excavation. By means of 3D finite element analyses the influence of existing structures on the displacement pattern around the tunnel is discussed.

Numerical analysis of foundation for underground bridge project in Moscow by Kolybin, Razvodovsky, Skorikov & Starshinov: this paper presents an interesting study of a very complex practical problem by means of a combination of 2D finite element and 3D spring models highlighting the importance of interaction between structural and geotechnical engineers in solving such problems.

Rigid-plastic analysis of tunnel face stability affected by ground water by Konishi, Kawashima, Tamura, Kitagawa, Lida & Matsunaga: the assessment of tunnel face stability by means of a rigid finite element analysis including water pressure as additional force is suggested. The proposed method has been applied to railway tunnels under construction but no field data from construction are available as yet. A concept for a design chart separating stable and unstable conditions is included.

A new stress-strain approach for tunnel face stability by Guilloux, Kurdts, Bernhardt & Wong: a simplified, but practical, approach for assessing the safety factor of tunnel faces, including face reinforcement in open face tunnelling or stabilizing pressures for bored tunnels, has been developed. The method is conceptually similar to the convergence-confinement method frequently applied for analysing tunnel sections well behind the face. Validation is provided by means of 2D axisymmetric and 3D finite element analyses. A simple design example is also given.

3.2 *Theoretical aspects dominating*

Numerical analysis for provision of tunneling – induced ground deformation in granular soil by Castelli & Motta: verification of settlement troughs based on a Gaussian distribution curve by means of a discrete element analysis (PFC) is attempted. Although a good agreement could be achieved a slight question mark remains on the validity of the results because a scaled model has been used in order to reduce the computational effort. In addition, some of the input parameters for the PFC model would need further justification.

Numerical analysis of settlements related to tunnelling: the role of stress-induced anisotropy and structure degradation in fine-grained soils by Amorosi & Boldini: the results of undrained analyses of tunnel excavation considering stress-induced anisotropy and structure degradation for different volume losses are discussed. The authors conclude that anisotropy has little effect on the settlement trough for the case considered, which however is in contradiction to other data published in the literature. It is acknowledged that generalization might be difficult and the negligible influence of anisotropy could be due to the formulation used in this study. As expected destructuration has some influence on tunnel stability but for moderate volume losses the effect on settlements is again not significant.

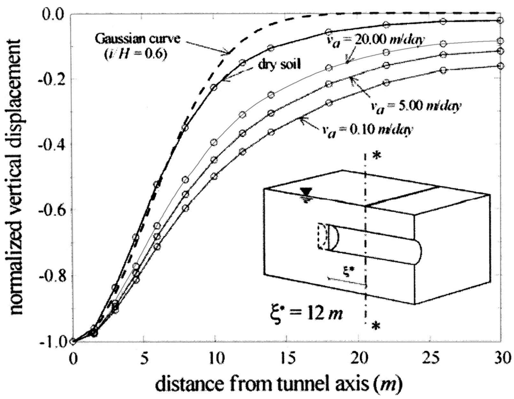

Figure 1. Influence of advance rate on normalised settlement trough (Callari & Casini).

Three-dimensional analysis of shallow tunnels in saturated soft ground by Callari & Casini: fully coupled analyses in medium to low permeability soils are presented. It is shown that the settlement trough in saturated soil is wider than in dry soil, depending on the advance rate, and is not consistent with a Gaussian distribution (Figure 1). A slow advance rate leads to higher settlements. The importance of taking lining permeability into account is pointed out.

Influence of lagging distance on the interaction of two open face parallel tunnels by Tang & Ng: describes results from a theoretical study by means of a 3D coupled consolidation analysis on the effect of the lagging distance between two parallel tunnel excavations. The conclusions drawn are that horizontal movements are stronger influenced by the lagging distance than vertical movements and that there is a shift of maximum surface settlement, the amount of which depends on the lagging distance. The maximum settlement does not occur at the centre line of the pillar between the two tunnels. Distribution of bending moments in linings is also influenced by the lagging distance.

3.3 *Theoretical aspects – field data*

Numerical Simulation of a strain softening behavior of a shallow tunnel for a bullet train by Akutagawa, Lee, Doba, Kitagawa, Konishi & Matsunaga: it is shown that even with a simple strain softening constitutive model good agreement between numerical analysis and in situ measurements can be achieved in a back analysis. However, no details on the numerical implementation are provided and thus no information is given how mesh dependence of results, as would be expected in this type of problems, is overcome.

Numerical analyses of a tunnel in London Clay using different constitutive models by Masin & Herle: data from the Heathrow Express trial tunnel are used to study the performance of different constitutive models, namely Mohr-Coulomb, Non-linear elastic Mohr-Coulomb, Non-linear elastic with cross-anisotropy, Modified Cam-Clay, Advanced kinematic hardening (3SKH) and Hypoplastic including intergranular strains. An attempt was made to model the full stress history and the results clearly show, not surprisingly however, that more advanced models show better agreement with measured data than simple models. But even with the advanced models the calculated settlement trough tends to be slightly too wide as compared with measurements (Figure 2). It is pointed out that non-linear elasticity plays an important role which confirms data published in the literature. It is further emphasized that none of the models predicted horizontal displacements very well.

Prediction of settlements and structural forces in linings due to tunnelling by Möller & Vermeer: a very useful comparison of 2D and 3D analyses of tunnel excavation employing the so-called ß-method (load reduction method, convergence-confinement method respectively) is presented. As expected, it is concluded that ß-values differ for settlements, bending forces and normal forces and therefore care has to be taken when choosing these values in engineering practice for design. The influence of the round length and soil strength is also investigated.

3.4 *Bored tunnels – general*

4D grouting pressure model PLAXIS by Hoefsloot & Verweij: it is reported that no clear connection between grouting pressure and observed settlements, which however have been very small in the particular case considered, could be established. A 3D FE-model was developed and the influence of various parameters such as grouting pressure, face pressure, tunnel bending stiffness and soil stiffness on calculated settlements has been studied.

Parametric studies for shield tunnelling in soft soils by Kasper & Meschke: results from a comprehensive numerical study investigating the influence of the filter cake, the slurry pressure, the bending stiffness of jacks, the weight of the TBM, the conicity and the friction angle of ground by means of a fully coupled analysis are presented. The influence of each of these factors on the development of settlements with time is shown in diagrams (e.g. Figure 3).

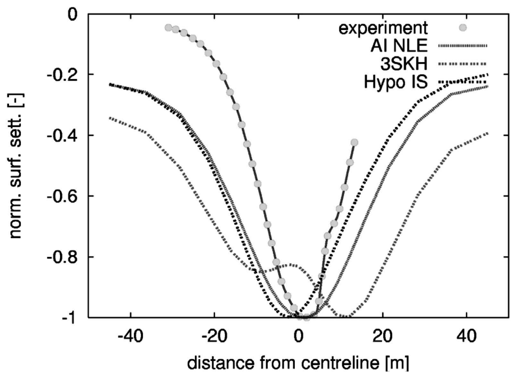

Figure 2. Normalised settlement trough for advanced models and measurement (Masin & Herle).

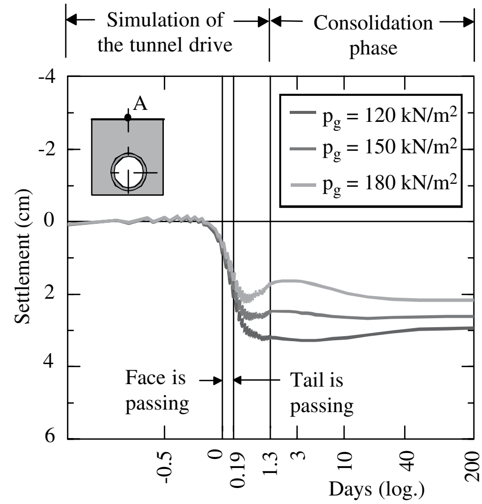

Figure 3. Influence of grouting pressure on calculated settlements (Kasper & Meschke).

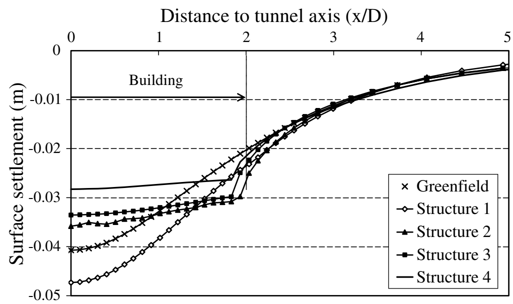

Figure 4. Calculated settlement trough for different stiffness of structure (Jenck & Dias).

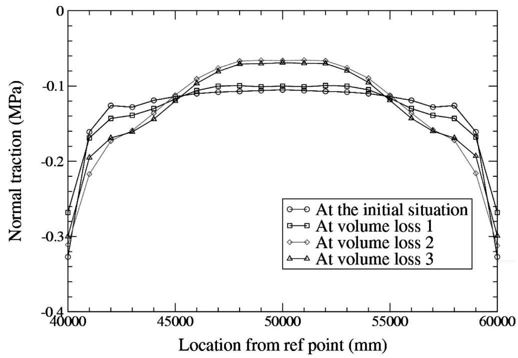

Figure 5. Normal traction on rough foundation for different values of volume loss (Boonpichetvong, Netzel & Rots).

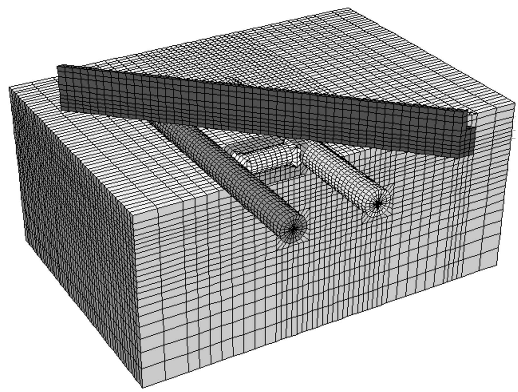

Figure 6. 3D numerical model used by Viggiani, Soccodato & Burghignoli.

3.5 *Bored tunnels – lining*

Axial pre-stresses in the lining of a bored tunnel by Koek, Bakker & Blom: the importance of longitudinal forces in tunnel linings and the effects of a sudden in- or decrease of jack forces, leading to an exponential change in the axial force, is addressed. It is emphasized that using plywood as joint material may cause axial forces to vanish in case of deterioration, with consequences for possible ring sliding and leakage.

Numerical modelling of the behavior of shield tunnel lining during assembly of a tunnel ring by Mashimo & Ishimura: in situ measurements and analysis based on a subgrade reaction method investigating the loading on linings during assembly are compared. It is concluded that these stages should not be ignored in the analysis because of the thrust force required to install keystone elements and the pressure caused by tail seals and tail grease.

3.6 *Tunnel-structure interaction*

Numerical analysis of soil-structure interaction during TBM tunnelling under a structure by Jenck & Dias: the importance of 3D analysis when assessing the influence of tunnelling on existing buildings is highlighted. The main conclusions from this study are that the Mohr-Coulomb constitutive model predicts wider settlement troughs than observed and the settlement trough is only influenced if a certain stiffness of the building is exceeded (Figure 4). Applying greenfield results to the building would be too conservative. It is acknowledged that higher order models for soil and structures (linear elastic in this study) would improve the accuracy of the results.

Numerical analysis of soil-foundation-building interaction due to tunnelling by Boonpichetvong, Netzel & Rots: again the influence of tunnelling on shallow foundations is studied whereas in this contribution emphasis is put on the interface behaviour between structure and ground (Figure 5). It was found that the roughness of the foundation has a significant influence on the calculated displacements and thus on the strains transferred to the foundation of a building. A fixed smeared crack model was applied for representing the nonlinear behaviour of masonry buildings.

A study of the interaction between the new C Line of Roma Underground and the Aurelian Wall by Viggiani, Soccodato & Burghignoli: a detailed 3D numerical analysis (Figure 6) employing a Mohr-Coulomb model is described. It was found that the crucial construction phase depends on the stiffness of the wall. For stiff walls it is at the end of construction whereas for flexible walls it is when the tunnel is passing.

A study of the response of monumental and historical structures to tunnelling by Burghignoli, Lacarbonara, Soccodato, Vestroni & Viggiani: various models with different levels of complexity have been applied to calculate the effect of tunnelling on monumental structures, in this particular case the Basilica of Massenzio. The basilica was discretized by means of a fully three-dimensional finite element mesh employing a nonlinear material model for

Roman concrete which was subjected to the settlement profiles obtained from semi-empirical relationships and 2D finite element analyses. The results show that computed effects of tunnelling on the structure are less pronounced when using more refined models as compared to simplified analysis.

3.7 *Tunnel-pile interaction*

Three-dimensional analyses of piled raft foundation subjected to ground movements induced by tunnelling by Matsumoto, Kitiyodom & Kawaguchi and *Analyses of pile foundations subjected to ground movements induced by tunnelling by Kitiyodom, Matsumoto & Kawaguchi*: these companion papers discuss the performance of single piles, a pile group and a piled raft subjected to the influence of tunnelling employing different models (3D FDM, Beam-Spring) treating soil and structures as elastic materials. Good agreement with respect to lateral pile deflection and bending moments in the piles was achieved with all approaches employed.

Analysis of effects of tunnelling on single piles by Surjadinata, Carter, Hull & Poulos: a method based on a combination of finite elements and boundary elements is proposed. A 3D finite element analysis for the free-field situation (no pile present) is performed and the resulting displacement field is imposed on a 3D boundary element model of the pile. Thus the computational effort is significantly reduced because only one 3D finite element analysis is required when investigating different pile arrangements. The applicability of the model is demonstrated by comparing calculated and measured horizontal displacements for 3 case histories and one benchmark case.

Simulations of tunnel excavation in 2D and 3D conditions considering building loads by Nakai, Sung, Shahin & Yamamoto: compares 2D and 3D finite element analyses and model tests of a comprehensive study investigating the deformation mechanisms and earth pressure distributions during tunnel excavation when a group of piles or a piled raft is situated in the vicinity of the tunnel. As in other contributions to these proceedings it is highlighted that the presence of structures leads to significantly different settlement troughs as compared to the greenfield situation.

4 KEY POINTS OF PAPERS – DEEP EXCAVATIONS

4.1 *General aspects*

A 3D FE model for excavation analysis by Bakker: a very general paper, highlighting the importance of 3D analyses for modelling excavation problems and the influence of soil stiffness on the extension of the calculated settlement trough. Technical standards codes have to meet from a practical point of view are discussed.

Study of soil-retaining wall interaction by the Contact Finite Element Method by Ras & Bekkouche: a short contribution presenting some results from a numerical study on the influence of wall friction on the earth pressure distribution of concrete retaining walls.

4.2 *Practical aspects dominating*

Soil improvement during excavation in soft ground and development of analysis method for earth retaining structure by Motoi: presents an extension of the subgrade reaction method to account for soil improvement on the passive side during excavation. The developed model has been successfully applied to analyse a practical project.

Numerical analysis of displacements of a diaphragm wall by Mitew-Czajewska: compares subgrade reaction models and numerical analysis employing a Mohr Coulomb constitutive model with in situ measurements (back analysis). Only results for maximum horizontal displacement are presented and generally a good match could be achieved, numerical results being closest to measured values. The difficulties in obtaining realistic spring constants for subgrade reaction models and their influence on the results are addressed.

4.3 *Theoretical aspects dominating*

Passive resistance of soft clay in deep excavations by Tamano, Nguyen, Kanaoka, Matsuzawa & Mizutani: numerical analyses and physical model tests are employed to evaluate time effects on the passive resistance of retaining walls. It is advocated that the decrease of passive resistance with time should be taken into account when analysing practical projects. However, it was found that Rankine's formula provides a reasonable lower bound for the steady state passive resistance.

4.4 *Interaction with structure*

Settlement of single foundations due to diaphragm wall construction in soft clayey ground by Schäfer & Triantafyllidis: 3D FEM analysis of diaphragm wall construction are presented and its influence on the settlement of an adjacent footing using a hypoplastic constitutive model, varying the panel length and the position of the footing is investigated.

4.5 *Data base*

New developments of the MOMIS database applied to the performance of numerical modelling of underground excavations by Mestat & Riou: presents interesting data from the comprehensive MOMIS database

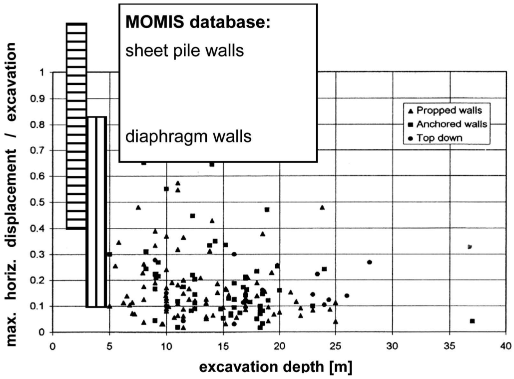

Figure 7. Range of values for ratio of maximum horizontal wall displacement vs excavation depth as suggested from MOMIS data base (reproduced from Long, 2001).

including 168 cases of retaining structures. It contains comparison of numerical analysis and in situ measurements with most of the analyses being back analyses. The main conclusions drawn in the paper are that errors in horizontal wall displacements are much smaller than for settlements and that errors in bending moments are much higher than for strut loads.

If horizontal displacements are related to the depth of the excavation (as compiled e.g. by Clough et al., 1989 and Long, 2001) the MOMIS database suggests 0.1 to 0.8% for diaphragm walls and 0.4 to 2% for sheet pile walls, which is rather on the upper end and above the data presented by Long, 2001 (Figure 7). However, further information, in particular on the stiffness of the retaining measures, not provided in the paper, would be required before conclusions on the discrepancies between the two compilations can be drawn.

5 CONCLUDING REMARKS

The papers submitted to this session cover a wide range of topics important in the analysis of tunnels and deep excavations. Tunnel – structure interaction analyses seem to have gained significant importance. It is observed that analyses for practical projects are still using "simple" constitutive models in many cases but, certainly depending on the purpose of the analysis and the expertise of the user, valuable results can be achieved using these models. It would not be appropriate to dismiss them as some of the more theoretical papers may suggest.

Sophisticated theoretical investigations provide valuable insight into a certain problem, but despite their high level of modelling, generalization is often difficult suggesting the conclusion that each project is a prototype and requires separate analysis.

Summarizing the topics of the papers in wider terms, such as whether real practical problems have been analysed or particular aspects, some of them in great theoretical detail, have been investigated only a slight overhead of "theoretical" papers is observed. This clearly indicates that numerical methods are established both in practice and research, with different levels of complexity though. It has to be pointed out that only numerical models are capable of taking into account the influence of the stiffness ratio of ground and interacting structures such as piles or buildings with reasonable accuracy. The importance of this, also from a practical point of view, has been addressed in a number of papers submitted to this session.

Despite the advances made, further research in numerical modelling is essential, given e.g. the fact that it is still a challenge to predict a settlement trough following a Gaussian distribution curve. From a practical point of view some rough guidelines on how sophisticated models have to be for serving a given purpose of an analysis would be helpful for geotechnical engineers in practice.

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Monitoring ground and structural response to underground construction works

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ABSTRACT: This General Report covers 21 papers that are included in Session 5 of the Symposium, relating to the monitoring of ground movements caused by tunnelling and deep excavations. The papers have been divided into three groups: tunnelling, excavations and new monitoring methods. The group concerning tunnelling is the largest and has been sub-divided into monitoring case studies involving greenfield sites; soil-structure interaction (including three papers on the effects of tunnelling on piled foundations); and tunnel linings.

Overall it is noted that, in several of the case studies, the magnitude of displacements is very small, e.g. less than 5 mm, reflecting improved construction practice, technology and better control. Control is frequently achieved with data from monitoring ground, construction work and structural response. As displacements are often very small, this necessitates increased resolution and accuracy of the instrumentation and surveying techniques. This is particularly the case if the results are to be meaningfully interpreted.

1 INTRODUCTION

This session contains 21 papers from nine countries. The content of the papers can be broadly divided into tunnelling and excavations. There are a greater number of papers concerning tunnelling, covering subjects ranging from decisions about the alignment and type of tunnelling, to details of the tunnel lining. The main emphasis of this session concerns the monitoring data, which help us to understand better ground and structural response to tunnelling and deep excavations. The quality of some of the monitoring data reflects improvements in the instrumentation available. This is in fact becoming increasingly necessary because the ground and structural movements observed are often very small, being less than about 5 mm at the most. This equally reflects how tunnel boring machine technology has advanced, particularly with the sophisticated closed-face machines, another topic covered by some of the papers.

In addition to the improvements in the quality of monitoring new techniques are being developed. Two of the papers describe such developments.

Previous Aspects of Underground Construction conferences have provided a forum for discussing ongoing international projects. In many instances our technology has to keep up with the increasing demands from society who on one hand want modern efficient transport infrastructure but with minimum impact on the environment and existing structures and monuments. These conferences give us the opportunity to learn collectively about new developments, approaches and problems. Within this session there are many papers that exemplify such exchanges.

2 MONITORING PHILOSOPHIES AND TECHNIQUES

Assessing the influence of underground construction necessitates monitoring of the ground and structures with varying levels of complexity. The primary quantity usually measured is displacement. Vertical surface displacements are the easiest and most reliable to measure. Horizontal displacements are more difficult and less reliable but are important for establishing strains, which are frequently linked with damage. Other quantities such as subsurface displacement, forces and stresses in structural elements and crack monitoring are usually secondary.

Another quantity that is being increasingly used is volume loss, almost always expressed as volume of the surface settlement trough divided by the nominal tunnel volume (usually given per unit length of tunnel). Several of the papers within this session have taken this approach. It has been suggested that volume loss is an appropriate quantity to use for assessing the control of tunnelling operations as it is probably the most significant factor in estimating tunnelling-induced settlements (see discussion in Jardine, 2003 and Mair, 2003). There is also a better understanding of which aspect of the tunnelling causes the volume loss and again this is mentioned by some authors in this session (see also Burland *et al*., 2004).

It seems appropriate to mention first the paper by ***de Vries and Duijvestijn***, discussing the new North-South metro line currently underway in Amsterdam. They provide a background to the reasons for the new line and focus on the works associated with the Central Railway Station. Because of the sensitivity of the railway lines and station buildings, which have to remain fully functional during the construction works, and the fact that new construction methods are being used, monitoring is essential. The scheme adopted has primary and secondary systems with a layout dictated by the sensitivity of the structures as identified through a risk analysis approach. The primary system involved automated total stations and liquid levels, the choice depending on whether clear lines of sight were available. The secondary system comprised existing crack monitoring and detailed inspection and recording of architectural features. It is interesting to note that at the Toulouse conference in 2002 the use of automated total stations was a fairly new development, while now they are considered as a key component of most large-scale monitoring systems (as is evident from several of the papers in this session).

As the railway structures have very low tolerances to movement, corrections to the monitoring data are made to take into account seasonal movements. This necessitated monitoring in advance to establish their magnitude and form and relate them to temperature (the authors recognise that other quantities, such as solar radiation and humidity, are likely to have an influence, but these are not measured). The authors report that measurement points on the front façade of the station building fluctuated by ± 2.0 mm, representing the combined effect of seasonal temperature changes and instrument accuracy. A system is in place for making the corrections, based on measurement results from a longitudinal joint in the brickwork. Other examples of data from advance monitoring on the North-South line project were given by Netzel and Kaalberg (2002) at the Toulouse conference.

More general information about the monitoring approach and methodology adopted on the North-South line are given by ***Van der Poel, Gastine and Kaalberg***. They describe the very comprehensive monitoring system that is in place with thousands of optical targets mounted on surface structures read by a network of 74 robotic total stations and several thousand manual precise levelling points, allowing both ground and building movements to be determined. Additionally several hundred subsurface boreholes with extensometers and inclinometer devices (sometimes combined) have been installed. Most of the monitoring takes place within a zone where settlements have been predicted to be greater than 1 mm. There is also a buffer zone extending about 35 m beyond that where only precise levelling is performed. The vast data sets are relayed by a radio network about every 20 minutes to engineers for processing and analysing. Checks are made between the different monitoring methods, which in turn are related to datum points outside the zones of influence, thus providing relative and absolute measurements.

An observational method approach is used for control of the construction works using the monitoring data. The data are stored, presented and updated on a GIS database, which also includes trigger levels to give staged warnings (alarms) when movements reach predicted values and to alert site staff when precautions need to be implemented to control construction in order to avoid damage.

Another major tunnelling project currently underway in Barcelona is described by ***Schwarz***, ***Boté and Gens***. The total length of the tunnel route is 38.5 km through very mixed geology which has been roughly grouped into igneous rock, soft rock and soils, requiring different tunnelling techniques. Discussion is given of three options that were considered for the tunnel section: two 6 m diameter tunnels with a single line in each; a single 9.5 m diameter tunnel with two lines within it running side by side and; a single 12 m diameter tunnel with the lines one on top of the other. The advantages and disadvantages of each are described in the context of issues like the geology, tunnelling operations and station connections. Combinations of the latter single tunnel sections were adopted according to the local conditions.

The reasons for monitoring on the project are listed as (i) an indicator and analysis tool for building response to the works; (ii) control of excavation parameters; (iii) a check on the influence of tunnelling on the groundwater regime (in some areas the effects of tunnelling on hydrological conditions is considerable) and; (iv) a means of providing data for research and development tasks. Ranges of computed volume losses are given for the different tunnelling methods and ground conditions. The maximum value quoted is 0.9%, resulting from the EPB machine operating in river alluvium, which is still quite a small value. The rates of tunnelling advance are also related to the geological conditions and associated tunnelling method. It was necessary to implement protective measures where intensely weathered granite was encountered. This took the form of groundwater lowering and grouting with micro-cement above the tunnel.

Two papers provide details of new techniques for monitoring. ***Take***, ***White***, ***Bowers and Moss*** describe a method of real-time image analysis involving remote digital photography and a Particle Image Velocimetry (PIV) processing technique in conjunction with data transfer and automated web-based update systems. The image analysis method is explained. A digital image from a camera comprises a matrix containing the intensity recorded at each pixel. The two-dimensional movement of a point on the image can be determined by comparing images, i.e. intensity matrices, from different times using the PIV technique. The precision of the PIV measurements depends on the processing software which use different correlation algorithms and sub-pixel interpolation methods. A precision of better than 1/100th of a pixel was achieved for the project described here. A considerable advantage of the system is that it is not necessary to place survey targets on the monitored structure, although doing so helps establish the image scale (mm/pixel).

The system was tested on a retaining wall influenced by tunnelling for the Channel Tunnel Rail Link Project (CTRL) in North London. Scatter of calculated settlement data prior to the tunnel affecting the monitored points was between 0.1 and 0.25 mm (see Figure 1), the larger values being associated with greater distances from the camera (the datum for measurements was about 120 m away). Excellent correlation was observed between targets. The profiles shown in Figure 1 are relative to the settlement of the camera itself and so the closer the target to the camera the smaller the movement and eventually the camera settled the same as the targets, as indicated by the fact that the profiles roughly return to zero displacement. The maximum differential settlement between the camera and the targets was less than 6 mm.

Details are also given of a validation exercise performed to assess the accuracy of the system. It was found to be less than 0.2 mm over a distance of about 20 m, i.e. comparable with other surveying techniques. The system shows considerable promise for future projects because of its accuracy, versatility and real-time acquisition capability. A limitation might be the need for adequate lighting of the targets.

Another new, but quite different, monitoring technique described by ***Metje***, ***Chapman***, ***Rogers***, ***Miao***, ***Kukureka***, ***Henderson and Beth*** involves the use of optical fibre sensors. At intervals along the length of the optical fibres Fibre Bragg Gratings (FBGs) are 'written' within them, resulting in a change of the refractive index of the material and hence making it possible to monitor relative movements between FBGs. Four optical fibres are installed (glued) within grooves formed on the four inner faces of a square-section pultruded fibre-glass Smart Rod. Initial tests were performed on the Smart Rod with incorporated optical fibres to assess its mechanical properties under different environments. This revealed that the process of stripping the coating of the fibre to form the FBGs was significant. Two laboratory tests were then performed, involving (i) a crane beam and (ii) a bench test procedure. It was established that the Smart Rods are very temperature dependent but that this can be effectively corrected providing the temperature is carefully measured. Excellent agreement was found between measured and theoretically calculated strains as shown in Figure 2.

The authors also discuss practical issues such as the importance of the manner of clamping the Smart

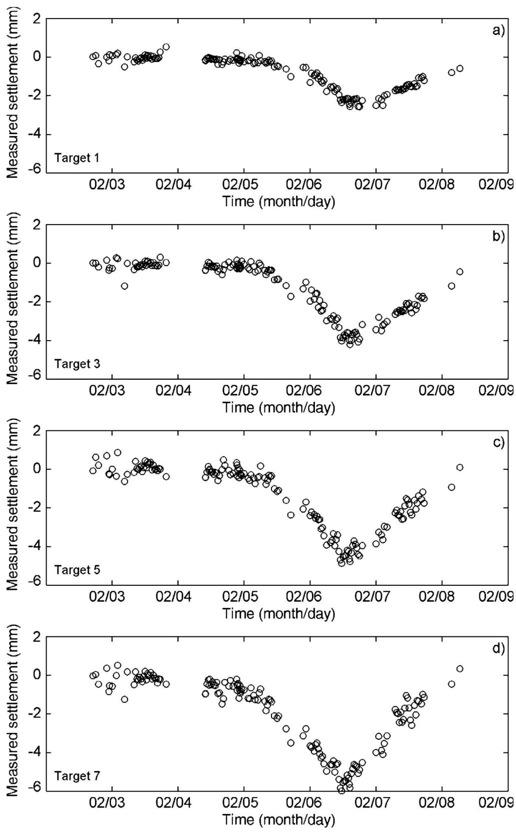

Figure 1. Results from real-time monitoring using image analysis techniques described by Take *et al.* (note that the targets are at increasing distances from camera: 1 the nearest and 7 the furthest away).

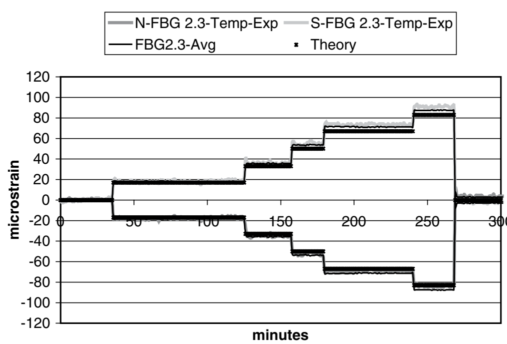

Figure 2. Comparisons between measured and theoretically calculated strains at one FBG location resulting from vertically-induced movement of the Smart Rod in 0.5 mm increments. (Metje *et al.*).

Rods and the position of the optical fibres within the grooves. This system is intended for use in inaccessible environments, e.g. within existing tunnels affected by nearby construction activity. Once perfected, this system would offer many advantages over conventional strain measurement systems, for example using tape extensometers which require direct access and which have their own inherent problems (Standing *et al.*, 2001). The direct measurement of strain along lengths of buildings and other structures using a system like this, capable of such small resolutions, would also be of great interest for control and research monitoring.

3 GROUND MOVEMENTS AND TUNNELLING PERFORMANCE

The papers in this section give details of ground movements, in particular with reference to the effect of protective measures and tunnelling machine performance. In the 2001 ISSMGE conference in Istanbul, Professor Mair, as discussion leader of the session concerning underground construction in urban areas, proposed these two subjects for discussion (Mair and Standing, 2002). The papers here help address the lack of case study data relating to both issues.

Russo and Modoni present an interesting case study concerning a short 50 m length of tunnel forming part of the 'high velocity' railway line close to Florence. The tunnel was 11m high and 15 m wide, passing through mixed face conditions with loose gravely soil above the crown, underlain by silty soil. As the crown was very shallow and in order to protect overlying structures, the tunnel was constructed using a jet-grouted canopy extending around its extrados almost to invert level as shown in Figure 3. Additionally fibreglass tube spiles were installed into the face. The inclusions forming both of these protective measures were linear elements with overlapping lengths. A staged sequence was used for excavation and installation of the inclusions to provide support at all times. Ribs and shotcrete were also placed immediately as excavation proceeded followed by a final cast-in-place lining.

The rate of construction/excavation increased as experience was gained and equally the magnitude of ground movements reduced. Installation of the jet-grouted elements for the canopy tended to cause surface heave above the centre-line of the tunnel, diminishing laterally, while the elements forming the side walls (pillars) resulted in long-term settlement. Excavation of the face had a smaller influence on settlements. The results shown in Figure 4 illustrate these effects and show that the maximum settlements experienced were less than 30 mm. The authors draw attention to the benefit of the reinforcing measures and point out that particular care is needed where long-term consolidation effects might occur in low permeability soils.

Details and monitoring data from another tunnelling project in Italy, the Torino Metro, are given by ***Barla***, ***Barla***, ***Bonini and Crova***. In this case tunnelling was by an EPBM of 7.8 m diameter, passing through cemented sands and gravels which are overlain by 8 to 10 m of sand and gravel deposits with the water table well below invert level. The protective measure in this case, implemented to avoid any localised instability, was by a consolidated slab formed above the crown prior to tunnelling. This slab was roughly 5 m wide by 2.5 m deep and was made using consolidation injections from the ground surface.

Numerous monitoring cross-sections were installed along the route for assessing vertical displacements which, from the data presented, seem to be minute, with maximum values of about 2.5 mm above tunnel centre-line but generally being about 1 mm. These values increase by about 50% in the longer term. TBM parameters are also given in the form of thrusts and these have been related to different geological units.

The settlement data have been modelled using standard approaches, i.e. assuming a Gaussian distribution for the transverse trough and also using numerical analyses. The latter have been used to assess TBM parameters such as face pressure.

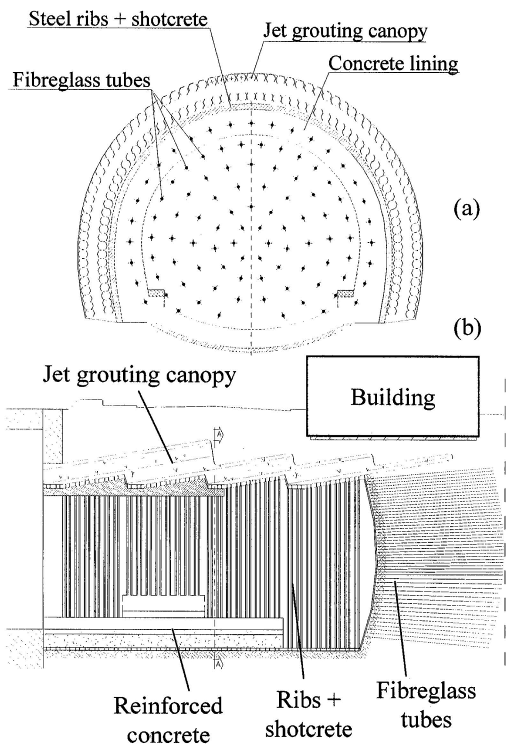

Figure 3. Construction details of jet-grouted canopy and face reinforcement (a) in cross-section and (b) longitudinally for the Florence tunnel (Russo and Modoni).

Bowers and Moss give details of the 18 km of twin tunnelling carried out for the Channel Tunnel Rail Link (CTRL) project. The 8.15 m tunnels were to pass through a variety of ground conditions, including Thames river deposits, London Clay, Lambeth Group, Thanet Sands and Chalk (i.e. most of the geological formations encountered in London). For this reason the tunnelling specification was for closed-face EPB machines that could also operate in an open-mode when passing through competent strata such as the London Clay. In the event, following ground movement and building damage assessments using a 2% volume loss, it was decided to tunnel most of the route in closed-face mode. One of the main reasons for this was the poor condition of the structures established during inspections.

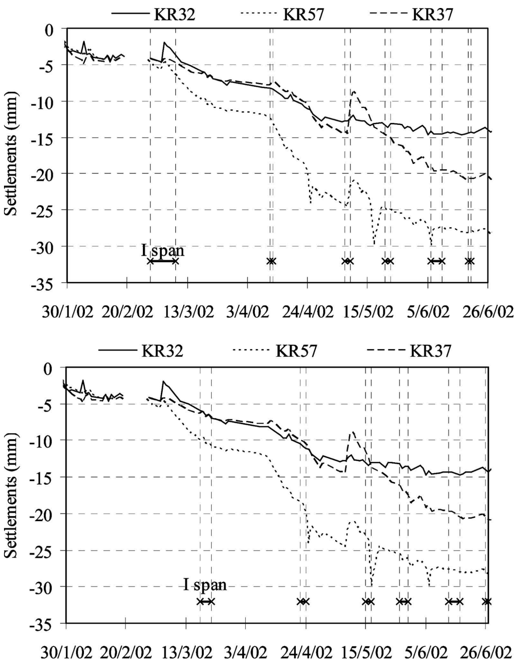

Figure 4. Settlements along a longitudinal section of the tunnel (left hand side) relating (a) to jet grouting operations and (b) excavation sequences (Russo and Modoni).

Monitoring was primarily by precise levelling with automated total stations being used in cases of difficult access. Results were stored on a central database and assessed both to refine the tunnelling process and also to reconsider the necessity for advance mitigation works. Two categories of ground movement are described associated with: (i) formation of a typical settlement trough, which can be reasonably characterised and (ii) much larger, erratic and highly localised movements resulting from causes such as the interception of geological features (e.g. peat and alluvium with pockets of water). Generally the volume loss values were in a range of 0.25 to 0.75% for most of the geological formations encountered, being much smaller than those used for the settlement assessments. Volume losses from the drives on one of the contracts are shown in Figure 5. Occasional instances where the second type of ground movement occurred are also given and explained. Careful monitoring also led to a better understanding of the EPBM operation, identifying that maintaining a constant fluid pressure around the shield and better control of the tailskin grouting results in reduced ground movements.

Vanoudheusden**, **Petit**, **Robert**, **Emeriault**, **Kastner**, **de Lamballerie and Reynaud also provide valuable data from the construction of part of the Toulouse Metro using a 7.8 m diameter EPBM through hard sandy clay with very dense sand inclusions (Toulouse molasses). The monitoring data were collected as part of a research project METROTUOL. The section considered comprised surface and subsurface instruments for measuring horizontal and vertical displacements around the tunnel. Additionally the tunnel lining was instrumented with strain gauges and TBM parameters were also carefully recorded.

Vertical surface displacements are, similar to the case of the Torino Metro tunnel, incredibly small, with heave above the tunnel centre-line of about 1 mm and settlement to the sides of less than 0.5 mm. The fact

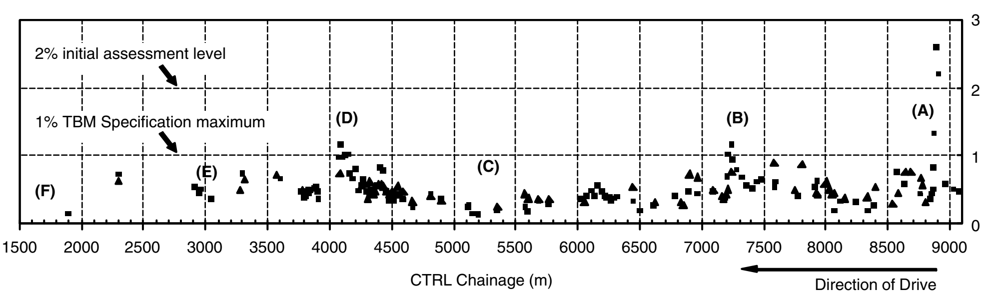

Figure 5. Volume losses for the upline (square symbols) and downline (triangles) tunnel drive on Contract 220 of the CTRL project (Bowers and Moss).

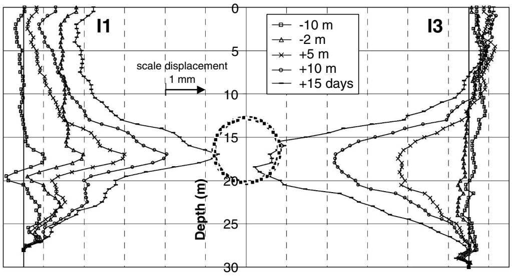

Figure 6. Transverse horizontal movements either side of tunnel from Toulouse Metro project (Vanoudheusden *et al.*).

that the profile is well defined is a testament of the accuracy of the precise levelling. A similar observation can be made regarding the very small subsurface movements measured with inclinometers and extensometers. A clear pattern of horizontal displacement with tunnel advance is presented both longitudinally and transversely. Movements were again very small with a maximum of 5.3 mm. An example showing the quality of the data is given in Figure 6. Vertical displacements were much smaller, being within ± 0.5 mm. This difference is attributed partly to the K_0 value of the molasses. It would be very interesting to construct a plot showing the resultant vectors of displacement to understand better the mechanisms of ground movement taking place and to compare them with those observed for open-face shield tunnelling (e.g. as given by Nyren, 1998).

Average strains measured within the tunnel linings indicated compressive stresses which correlate very well with the range of over-burden stresses between the crown and the invert. TBM parameters are also presented and show that the face pressures were typically about 60% of the total vertical stress at crown level (σ_0) while tailskin grouting pressures were between 110 and 150% of σ_0.

The papers in this section have provided excellent evidence of how well the sophisticated EPBMs can operate with very small displacements and low volume losses. High quality surveying instruments and dedicated staff are required to capture meaningful monitoring data in such cases. Examples of how precautions such as forepoling, spiling and grouting can help minimise movements in poor ground conditions are also given.

4 TUNNELLING CASE STUDIES INVOLVING SOIL-STRUCTURE INTERACTION

In the previous section the papers covered ground displacements and TBM parameters generally considering tunnels in greenfield conditions, no specific details of overlying structures being given. In this section the papers present case studies giving various structural responses to tunnelling operations. These inevitably involve soil-structure interaction and the order of the papers reflects increasing complexity, starting with surface buildings, progressing to the effect on existing tunnels and finally structures with piled foundations. This latter scenario was another area recognised at the previous Toulouse conference as being one where few case studies were available.

Because of the complexity of predicting structural response from tunnelling, an observational method approach is commonly adopted. The first instance where this has been reported as such was for the construction of the Chicago Subway. Monitoring techniques and technology have improved greatly since then, allowing much tighter control of tunnelling works using feedback from surveying and instrumentation.

Kontogianni, ***Psimoulis***, ***Pytharouli and Stiros*** reflect on this in their paper and give some recent examples from underground construction projects undertaken in Greece where an inductive (i.e. observational method) rather than a deterministic approach has been adopted. This has been necessary because of the difficulty in predicting rock mass characteristics or where shallow tunnels were to be constructed beneath historic structures. As noted in this paper and as has been mentioned in several of the papers within this session, the observational method is particularly viable with the improvements in real-time, high accuracy survey systems and data handling and processing capabilities.

A number of case studies are mentioned in the paper under three headings: metro tunnels in the urban environment; road tunnels and mining works. In the first class, tunnelling-induced deformations are of primary concern, particularly where historic buildings might be affected, examples from Athens and London are cited, where the observational method was used to safeguard such structures. The road tunnel case studies are more concerned with stability as evidenced from rapid convergence of newly constructed tunnel sections. Examples are given where monitoring data enabled causes of large displacements to be identified, e.g. weak fault zones. It is suggested that the data might be used to predict the rock quality in advance of the tunnel. In mining projects, intensive monitoring on a real-time basis can be used for safety issues (i.e. for stability checks), particularly as frequently excavations for mining purposes are only temporary. Examples are given where causes of large deformations were identified from geodetic data.

Moss and Bowers describe one of the challenging aspects of the CTRL project where the new tunnels were to be constructed beneath existing railway tunnels, with settlements limited so that they could remain operational during the works. The three-stage assessment implemented is described, the third stage being necessary for extraordinary structures such as existing tunnels. The existing linings, which were of various forms, were identified as being the critical element to protect, following detailed inspections and analyses. The capacity of the metro tunnel lining system was defined in terms of bending curvature. However, as this is not an easy quantity to measure, it was related to volume loss and hence settlement which could then be readily monitored. Damage mitigation measures were implemented in the form of loosening the circle bolts of the existing linings. Three trigger levels were defined to allow an incremental planned response to movements to be implemented.

Details are given of three pairs of operating tunnels that the CTRL had to pass beneath. This was successfully achieved by close teamwork, careful tunnelling control and the mitigation measure described above (this was only applied to one pair of tunnels). Various monitoring techniques were used with both automatic and manual measurements (the latter being restricted to 'engineering hours', to observe the response of the ground, the existing tunnels and the linings. For each pair of existing tunnels, two CTRL tunnels were constructed beneath them, with separations varying from 4 to 14 m. Measured volume losses were less than 0.6% (a value of 1% was used during the engineering assessments) with flexible deformed profiles following a Guassian form (similar observations on existing tunnels were observed during construction of the smaller diameter Jubilee Line tunnels, see Standing and Selman, 2001). An example of the data from tunnelling beneath one of the tunnels is shown in Figure 7, where the influence of the two CTRL tunnels and also the use of superposition can be assessed. This case study provides important evidence, with its associated data, of how tunnelling beneath existing tunnels can be achieved with minimum disruption.

The next three papers considered here are case studies involving the response of piled foundations to tunnelling works. The expansion of underground infrastructure means that it is becoming increasingly common to have to consider cases where new tunnels affect piled foundations. So it is exciting to have the information from these papers to help with our understanding and knowledge in tackling this complex soil-structure interaction problem.

Further interesting case studies and data from the CTRL project, this time relating to piled foundations, are given by ***Jacobsz***, ***Bowers***, ***Moss and Zanardo***. Three piled bridge pier foundations are described, one with end-bearing piles and the other two friction piles. The methods by which the structures were assessed are explained. Recourse was made to research by Jacobsz *et al*. (2004), looking at zones of influence around

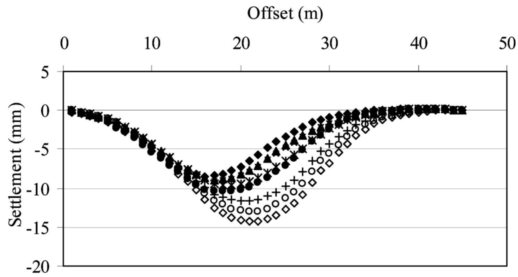

Figure 7. Data from the CTRL project showing settlements of an existing tunnel from tunnelling beneath it. The initial settlement profile is from the first Up-line tunnel, followed by developing profiles from Down-line tunnel (Moss and Bowers).

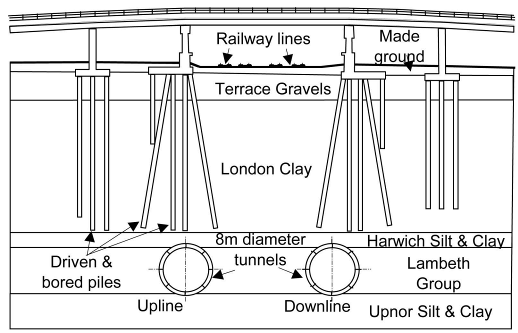

Figure 8. One of the piled bridge pier foundation case studies assessed during the CTRL project (Jacobsz *et al*.).

tunnels, expected pile settlements and load redistributions. In the case of the end-bearing piles settlement of the superstructure was judged to be the same as the soil (Terrace Gravels) at pile toe level. These were estimated and the bridge structure deemed safe for the level of movement anticipated. Monitoring data indicated this to be the case and no damage was sustained. In all three cases it was emphasised that great care was taken with the tunnelling operations at these locations to minimise volume losses.

Figure 8 shows a section of the first of the friction pile case studies where the pile toes were very close to the tunnels. The Terrace Gravels were grouted as a mitigation measure in this case, both to increase shaft capacity at that horizon and to create a pseudo-slab beneath the pile caps. Total surface settlements of 8 to 10 mm were observed (volume loss of 0.6%) with no detrimental effects on the bridge.

In the third case the strains along the length of the pile, both vertically and laterally (to obtain bending strains) were estimated from ground movements with depth assuming full friction at the soil-pile interface. The results indicated that the piles would not be over-stressed and that assuming that the pile movement is the same as that for the greenfield surface settlement is conservative. No mitigation measures were implemented and no damage was sustained to the bridge.

The authors recommend that pile capacities should be re-evaluated for such assessments as frequently there are large factors of safety allowing potential redistribution of loads in the piles.

A research project carried out in close collaboration with the parties on the CTRL project is described by ***Selemetas***, ***Standing and Mair***. Four full-scale instrumented piles were installed above and to an offset of one of the 8.15 m diameter tunnels. The driven cast in-situ piles (about 0.48 m in diameter) had load cells at their base and were instrumented along their length with sets of strain gauges and inclinometer electrolevels. Two pile lengths were investigated: one end-bearing in the Terrace Gravels (8.5 m long) and the other more of a friction pile with its toe in London Clay (13 m long). During tunnelling works the piles were loaded with kentledge to about half of their bearing capacities. Comprehensive surface and subsurface instrumentation was also installed in the ground close to the piles. The research was to investigate the zones of influence mentioned above where piles are subjected to different degrees of settlement relative to ground settlements and to examine changing load distributions along the pile lengths as the tunnels approach and pass beneath them.

Volume losses during the two tunnel drives were 0.2% and 0.5%, with the TBMs operating in closed EPBM mode. The zones of influence proposed by Kaalberg *et al*., 1999 and Jacobsz *et al*., 2004 were essentially confirmed as shown in Figure 9. This verifies that the Gaussian curve for modelling ground settlements can be used as a reference frame for assessing pile settlements. Preliminary data from the base load cells are presented showing, for the piles above the tunnel centre-line, increases as the EPBM approached, from the applied face pressure, followed by a reduction as the volume loss occurred with the pile settling more than the ground. Increases in base load were observed in piles when at the greatest offset from the tunnel (e.g. within zone C), caused by negative shaft friction. This is one instance where larger volume losses would have been beneficial to provide a more definitive response corresponding to open-face TBM works.

Pang, ***Yong***, ***Chow and Wang*** present and discuss data from part of the MRT North East Line contract in Singapore, where forward-thinking enabled instrumentation to be installed in bridge pier piles so that the influence of future planned tunnels, running parallel to the bridge, could be assessed.

The data from one pair of piles forming part of a four-pile group supporting one of the bridge piers are presented. The piles are 62 m long and 1.2 m

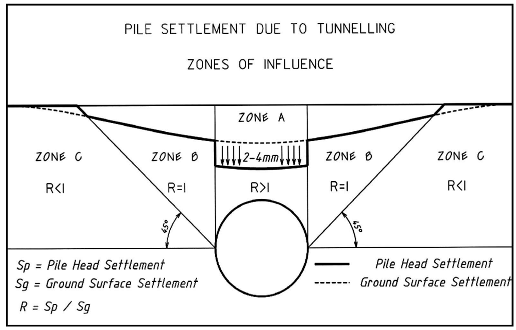

Figure 9. Zones of influence of piled settlement due to EPB shield tunnelling in London Clay (Selemetas *et al*.).

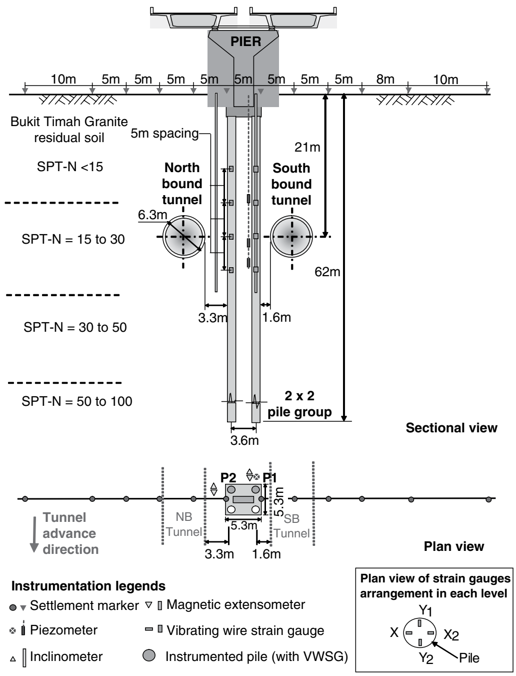

Figure 10. Typical section and instrumentation layout for pile-tunnel interaction study reported by Pang *et al.*

in diameter with four sets of strain gauges installed orthogonally, in pairs, to enable average axial loads and bending moments in transverse and longitudinal directions to be determined (see Figure 10). The data presented relate to one of the 6.3 m diameter EPBM tunnels, constructed in residual soils, 1.6 m from the nearest piles at a depth of 21 m (to its axis). The surface settlement profile from tunnelling had a Gaussian form with a maximum value of about 18 mm. Correlating the developing settlements with TBM position has enabled the volume losses relating to the different phases of the tunnel process to be identified. It is

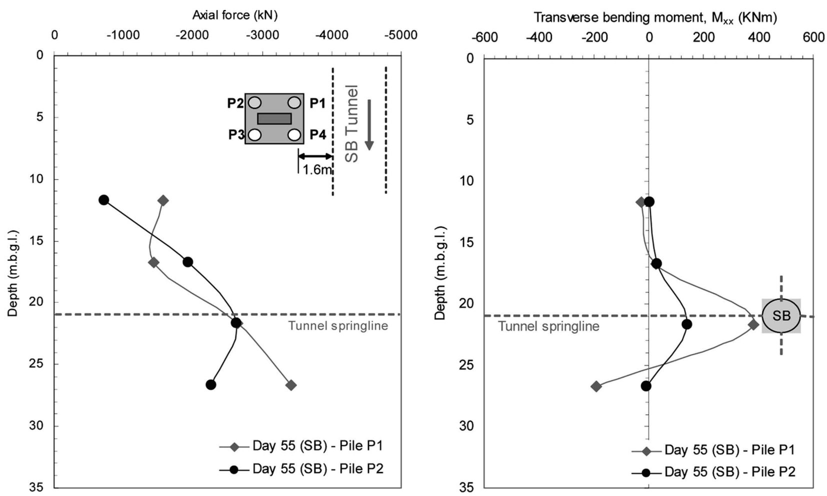

Figure 11. Response of pile foundation in terms of axial force and bending moment (Pang *et al.*).

reported that the range of volume losses was between 0.32 and 1.45%.

Information from the strain gauges within the piles reveals that the piles experience down-drag, registered as increasing axial force, with greater force developing in the pile nearer the tunnel as might be expected. Calculations indicate that down-drag loads were between 9 and 43% of the structural capacity of the piles (just from the first tunnel) with peak values occurring when the face of the TBM was in line with the piles. Clear trends in bending moment distributions along the length of the pile are also shown, with maximum values, although small, occurring in the close vicinity of the tunnel (see Figure 11). Also evident is shielding of the outer pile by the inner pile between it and the tunnel (see Figure 10). Some interesting relations between volume loss and axial force and bending moment are also presented, showing increases in both quantities with volume loss. The authors conclude that volume losses up to 1.5% do not seem to have a significant effect on the piles.

5 STUDIES INVOLVING TUNNEL LININGS

Two papers specifically discussing tunnel linings are described in this section. The data presented by Vanoudheusden *et al.*, covered in Section 3 of this report also include results from the monitoring of tunnel linings.

The first sentence of the paper by ***Bilotta***, ***Russo and Viggiani*** reads 'prediction of forces acting on tunnel lings is a rather complex task'. This is a very good introduction to the subject! They briefly describe difficulties in understanding the behaviour of linings and mention some proposed methods (by others) for modelling the forces imposed on them, particularly taking into account the presence of the joints. The uncertainty in tunnel lining design is evident from records from different projects, which often indicate significant variations in tunnel lining thickness for similar ground conditions. It is perhaps useful to note that a new guide to tunnel lining design has recently been produced by the British Tunnelling Society (BTS, 2004).

The research described involved instrumenting twelve segments (i.e. for two rings) with embedded vibrating wire strain gauges: five per segment, to allow circumferential forces and bending moments to be determined (with one dummy gauge in each). The transducers were monitored from the time of initial casting and were calibrated by loading the segments under laboratory conditions prior to installation within the tunnel. At the location where the ring was installed surface and subsurface ground displacements were also measured. The data from some of the segments are presented which show the increase in circumferential force at different positions around the tunnel, resulting from the gradually increasing ground stresses. It is noted that the measured bending moments are low, as might be expected with a segmental lining. The results presented are preliminary as the tunnelling project was underway at the time of writing.

Spasojevic, ***Mair and Gumbel*** describe analyses and sophisticated centrifuge model tests used to simulate the conditions of a deteriorating lining (sewer tunnel) rehabilitated with an internal cured in-place liner. Means of maintaining and renovating existing tunnel infrastructure and understanding the complex interactions between the ground, deteriorated lining and new liner are becoming increasingly necessary with aging systems and greater demands on them. The research project described in this paper sets out to investigate some of the main governing factors.

The manner in which the buckling mechanism of the problem (i.e. the collapsing sewer lining) is dealt with analytically (semi-empirically) is described and this then leads on to the design of the testing apparatus. An ingenious articulated tunnel was used to simulate the deteriorating sewer tunnel, it being possible to activate the hinges during the course of the test. The new liner was installed from the outset with a small gap between it and the outer lining, as encountered in practice. The effect of voids at different locations around the tunnel (formed in practice from soil being washed into the damaged lining with water inflow) was investigated using water-filled membranes placed at strategic positions that could be deflated during the test. Traffic loading was also imposed and varied at the ground surface. The whole system was heavily instrumented to measure deformations etc.

Experimental data from two tests are presented showing for these cases that voids present at the springing of the extrados have greater impact on distortions of the liner than void collapse at the invert. Overall the paper reports that the test programme has shown that the response of the flexible new liner is governed by interactions between the liner, host pipe and surrounding soil, resulting in non-circular distortions. The results are being used to formulate an improved practical design methodology for such liners.

6 EXCAVATIONS

Five papers are included in this section relating to the construction of excavation works. The first two papers concern specific aspects of wall construction and the others ground and structural response to excavations.

Kondo, ***Nakayama***, ***Naoe and Akagi*** describe a technique of wall excavation involving air foam rather than conventional bentonite slurry. In this new technique a foaming agent is diluted with water, stirred (whisked?) with air, increasing its volume 25 times and then mixed with soil. The methodology for assessing a *bentonite slurry* is explained, the key parameters being specific gravity and funnel viscosity, showing how variations of these quantities outside a certain range lead to performance problems. Countermeasures to solve these problems are also listed. A similar exercise is then performed for the new *air-foam-soil* medium, which because of its very different nature (to bentonite slurry) has to be quantified/characterised using alternative parameters. These are the unit weight of the air foam (c.f. specific gravity for slurry) and 'table flow value' (c.f. funnel viscosity): values of this latter quantity increase with decreasing viscosity. The air-soil-foam mix is also quantified in terms of mixing ratio and water content. Relationships between these quantities and potential performance problems have been investigated using a series of model tests in order to develop a management chart similar to that for a bentonite slurry. A range of values has been identified, outside of which there may be potential performance problems. A detailed cost analysis between the two methods is made, indicating that using the new medium could save about 30% in terms of the combined cost of stabilisation and soil disposal. Mention is made of a successful field test that was performed using the new management chart. The new method sounds as though it has much promise both in terms of saving resources and helping to minimise waste. Further full-scale field trials would undoubtedly help confirm this.

Another wall construction development, this time concerning tie-back anchors, is described by ***Tamano***, ***Nguyen***, ***Kanaoka***, ***Fuseya and Tonosaki***. The anchors have an enlarged (under-reamed) fixed length which is constructed using a drilling bit that can 'do the splits' (something usually associated with ballet dancers!), in this case opening out from a diameter of 135 mm (used to drill the free length) to 800 mm, hence the name 'splits anchors'. The sequence of construction is shown in Figure 12.

These anchors have components of resistance from: end-bearing at the front; shaft friction and suction. The authors point out that care is needed to isolate suction (which is generally not relied upon) and to understand the effects of relaxation and creep, the latter occurring because of creep of steel tendons, tendon-grout bond and most significantly at the soil-grout bond. Results are presented from two field tests on splits anchors, of two lengths (1.5 and 3 m), installed vertically within a stiff ($S_u = 126$ kPa) slightly over-consolidated structured clay. The tests on the anchors investigated

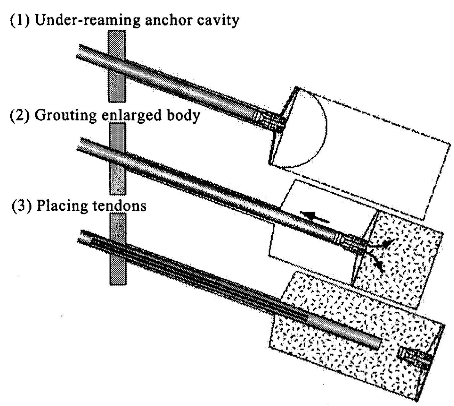

Figure 12. Construction sequence for splits anchors (Tamano *et al.*).

performance, through loading cycles, relaxation and creep. An example of the data from a relaxation test, where the load at the head of the anchor was locked off at 400 kN are shown in Figure 13. The effects of atmospheric temperature are clear.

The field test data were compared with results from a finite element analysis, the latter also being used to help understand longer-term effects and to assess the different components of the resisting forces. The combined field tests and numerical analyses indicate that the larger diameter of the splits anchors makes them more susceptible to load relaxation from consolidation. The end–bearing component makes a significant contribution to capacity but is over-estimated when using super-position to assess their overall resistance. Suction forces should be isolated from the results from field tests as it should not be relied on in the long-term. The tests also indicated that construction of the enlarged anchor using the splits bit caused minimal disturbance to the surrounding soil.

A simplified method for calculating (i) the maximum settlement behind a retaining wall and (ii) its distance from the wall is proposed in the paper by ***Kojima***, ***Ohta***, ***Iizuka and Tateyama***. The method is based on the results from a 2-D finite element parametric study in conjunction with field case study monitoring data. In the numerical analyses the type of element modelling the wall, wall stiffness, excavation width and depth and penetration length and position of support were varied. The intention of these analyses was to identify the critical 'major influence factor' governing movements of the soil behind the wall. The manner in which the wall is modelled was found to be more critical to settlements than varying soil properties. Three types of wall element were investigated as part of this exercise.

Field data from 42 case studies were compiled and checked by plotting position of maximum settlement against maximum settlement both normalised with excavation depth on the diagram originally given by Peck (1969). The data fall well into the regions originally mapped out by Peck and have been correlated in terms of an 'average' blow count $\bar{N}$ value given as a function of excavation and test depth (presumably based on the N-value from the Standard Penetration Test). Two further indices are then introduced, based on the 'major influence factor' identified in the numerical analyses: an equivalent stiffness, ξ (function of N and bending stiffness EI) and relative stiffness, ζ (function of equivalent stiffness, excavation depth and wall penetration depth).

Two diagrams relating (i) maximum settlement with relative stiffness and (ii) position of maximum settlement with equivalent stiffness are given which enable predictions to be made. The authors point out that the two indices (equivalent and relative stiffness) can be readily determined during the design stage of the excavation and it seems that the only other quantities required are basic knowledge about the soil types and their consistency and a profile of SPT N-value over the depth of wall penetration.

Skorikov, ***Razvodovsky***, ***Kolybin and Starshinov*** describe a construction project in Moscow involving several high-rise towers with two storeys of excavation beneath them. Russian codes specify that a mean building settlement should not exceed 12–15 cm with tilts less than 0.002 to 0.0024. Additionally if mean contact pressures are greater than 500 kPa the foundations should be piled. The paper considers the tallest building of the complex, for which the foundation pressure was 550 kPa but the use of a plate (raft) foundation was preferred for economy. It was therefore necessary to perform detailed analyses to assess whether total and differential settlements could be tolerated under the given loading. The intention was that the ground and settlements would be closely monitored during construction to confirm the results of the analysis.

The ground conditions comprise broadly loams and sandy clays of moderate strength. Two sets of calculations were performed: preliminary calculations of settlement using elastic theory followed by more complex analysis using a finite element code. In modelling the soil-structure interaction, elastic and elastic-plastic models were used, producing settlements that differed by 30% and tilts in opposite directions. Following these analyses the outline of the raft foundation was changed, the sequence of construction was altered to avoid outward tilting and to apply a surcharge to help with stability. Further analyses were performed with structural software to analyse the mechanical behaviour of the buildings, using springs to model the soil, whose stiffness values were determined from the earlier analyses.

Very good correlation was achieved between the monitoring data and the results from the numerical

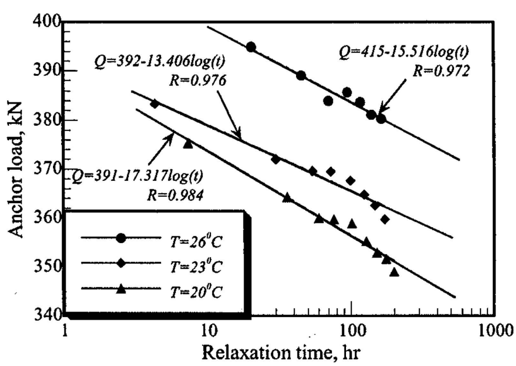

Figure 13. Results from a relaxation load test, with head load locked off at 400 kN, for the 3 m long anchor (Tamano *et al.*).

analyses, the latter slightly over-predicting values. The maximum magnitude of settlements from construction of the 32-storey tower was about 10 cm with a maximum tilt of 0.0005.

Heave of the base slab was also monitored and was found to be greater than expected from the unloading of the soil. This was attributed to freezing of water, from a sand lens which seeped out during prop construction, between soil and the underside of the raft, amplifying heave movements. Nearby buildings were also monitored: their maximum displacements were less than 2 mm and no damage was reported.

The final paper in this session is another very interesting case study from the Toulouse METROTOUL research project (c.f. the paper by Vanoudheusen *et al.*), this time by ***Emeriault***, ***Bonnet-Eymard***, ***Kastner***, ***Vanoudheusden and de Lamballerie***, concerning the response of the strutted diaphragm walls, ground and buildings behind the excavation for St-Agne station. Two monitoring sections perpendicular to the excavation were set up: one in essentially greenfield conditions and the other along the side of low-rise brick masonry structures adjacent to the station works. The excavation was within the Toulouse molasses, discussed earlier, to a depth of 17.2 m with the walls 20.7 m deep. Two inclinometer tubes were installed within the wall, extending below its base, roughly at the end of the sections. Struts within the excavation were strain gauged so that forces could be deduced. Vertical displacements were measured by precise levelling along both sections and horizontal strains along the buildings were monitored.

Profiles of settlement behind the walls are very small for both sections, the maximum being less than 4 mm in both cases, but the shapes of the profiles are slightly different (see Figure 14). The greenfield profile appears to exhibit more curvature and extends back much further (estimated by the authors to be about 60 m), than the profile measured on the buildings which is more rigid. In both cases the magnitude of movement is much smaller than would be expected using relations given by Clough and O'Rourke (1990) based on numerous case studies. Care needs to be taken in interpreting such small movements. The small magnitude is attributed to the improvement in construction techniques, the number of struts used and the good mechanical properties of the Toulouse molasses. There seems to be little long-term displacement.

Maximum horizontal displacements within the diaphragm walls were between 9 and 10 mm for both sections, occurring just above the base of the excavation. As wall embedment was quite small, being 3.4 m, some rotation is evident. It is also suggested that there might have been rigid-body lateral translation of the wall as its movement at the top (∼3 mm) does not correlate with the horizontal displacements along the buildings (∼9 mm). These were larger than

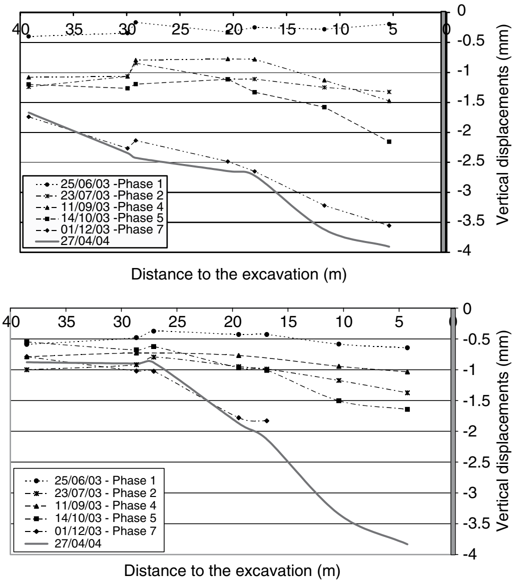

Figure 14. Settlement profiles along monitoring sections perpendicular to the excavation (a) greenfield case and (b) along the instrumented buildings (Emeriault *et al.*).

expected and it is suggested that this may be due to the high K_0 value of the molasses. The paper also presents and discusses measured strut loads. Most of the different monitoring methods, e.g. for walls, struts, buildings and the ground, produced results that are consistent. This is an important case study as it has detailed measurements allowing a better understanding of soil-structure interaction resulting from excavation-induced deformations.

7 CONCLUDING REMARKS

The papers in this session provide some excellent case studies to advance our knowledge of ground and structural response to tunnelling and excavation works. This is particularly important for cases where there is complex soil-structure interaction, for instance involving existing tunnels and piled foundations where careful thought is required with the assessments and where case studies can provide additional confidence and insight.

It is evident that improvements in construction technology and control have in many cases resulted in much smaller displacements, often amounting to no more than several millimetres, than would have been expected a few years back. The use of sophisticated protective measures such as fore-poling, spiling and grouting have also helped limit movements and in some cases enabled tunnels to be constructed in very sensitive locations, which hitherto would not have been possible without the risk of damage.

In order to assess and understand these smaller displacements the resolution and accuracy of the measuring systems has had to improve. There have been considerable advances in this respect, particularly with automated devices that provide regular, real-time readings. Automated total stations are a good example, where, through initial background monitoring, the thermal and seasonal response of the ground and structures can be understood and isolated from the construction –induced displacements. New techniques continue to be developed to advance our range of monitoring systems.

Finally the methods of managing, processing and interpreting the vast data sets resulting from automatic monitoring are continually advancing. Several of the papers mention this, particularly those involving the control of live construction projects. In tunnelling projects the settlement data are often expressed in terms on volume loss, which can be directly linked with tunnel machine performance, thus providing a continuous assessment of the works.

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Barla G., Barla, M., Bonini M. and Crova R. Back-analysis of monitoring data for Metro Torino.

Bilotta G., Russo G. and Viggiani C. Ground movements and strains in the lining of a tunnel in cohesionless soil.

Bowers K.H. and Moss N.A. Settlement due to tunnelling on the CTRL London tunnels.

De Vries J.H. and Duijvestijn A.M.W. Monitoring Amsterdam Central Station during construction of a new metro tunnel.

Emeriault F., Bonnet-Eymard T., Kastner R., Vanoudheusden E. and de Lamballerie J.Y. Movements induced on existing masonry buildings by the excavation of a station of Toulouse subway line B.

Kojima K., Ohta H., Iizuka A. and Tateyama M. A simple evaluation method of adjacent ground settlement due to excavation work.

Kondo Y., Nakayama T., Naoe H. and Akagi H. Cost reduction of diaphragm wall excavation usaing air foam.

Kontogianni V.A., Psimoulis P.A., Pytharouli S.I. and Stiros S.C. Geodetic monitoring of underground excavations: 70 years after Terzaghi's innovative techniques at the Chicago Subway tunnels.

Metje N., Chapman D.N., Rogers C.D.F., Miao P., Kukureka S.N., Henderson P. and Beth M. Optical fibre sensors for remote tunnel displacement monitoring.

Moss N.A. and Bowers K.H. The effect of new tunnel construction under existing metro tunnels.

Pang C.H., Yong K.Y., Chow Y.K. and Wang J. The response of pile foundations subjected to shield tunnelling.

Russo G. and Modoni G. Monitoring results of a tunnel excavation in an urban area.

Schwarz H., Boté R. and Gens A. Construction of a new Metro line in Barcelona: design criteria, excavation and monitoring system.

Selemetas D., Standing J.R. and Mair R.J. The response of full-scale piles to tunnelling.

Skorikov A.V., Razvodovsky D.E., Kolybin I.V. and Starshinov A.A. Behaviour of plate foundation in deep excavation beneath 32-storey building in Moscow.

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Take W.A., White D.J., Bowers K.H. and Moss N.A. Remote real-time monitoring of tunnelling-induced settlement using image analysis.
Tamano T., Nguyen H.Q., Kanaoka M., Fuseya Y. and Tonosaki W. Field tests and numerical investigation of spilt anchors in structured clay.
Van der Poel J.T., Gastine E. and Kaalberg F.J. Monitoring for construction of the North/South Metro line in Amsterdam, the Netherlands.
Vanoudheusden E., Petit G., Robert J., Emeriault F., Kastner R., de Lamballerie J.Y. and Reynaud B. Analysis of settlements caused by tunnelling with an earth-pressure balance machine and correlation with excavation parameters.

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Deep excavations

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ABSTRACT: This General Report reviews a selected group of papers related to "Deep Excavations" as part of the 5th International Symposium on Geotechnical Aspects of Underground Construction in Soft Ground. The themes of the reviewed papers include theory, design, performance prediction, performance measurement, and performance analyses. Several unusual case histories and failure cases are also discussed and presented in brief as they relate to the overall theme of the session. The papers were reviewed in the context of past conferences in this series, the goals of Technical Committee 28, and important aspects for design, construction, and performance of deep excavations. Concluding recommendations are made with respect to engineering principles and implications related to publication of case histories for future conferences and evolution of the state of the art and practice.

1 INTRODUCTION

A total of twenty-one papers have been included under the theme of "Deep Excavations." Another six papers, included under different themes, are also related to issues associated with deep excavations. These papers covered a range of familiar topics including:

– investigations and testing
– physical experimentation and theory
– design and performance prediction
– field performance
– unusual problems and solutions.

This General Report is not intended to summarize all of the papers within this theme. A selected group of the papers are reviewed to provide the context for drawing general conclusions and recommendations related to the subject of deep excavations.

2 REVIEW OF SELECTED PAPERS

In preparation of this General Report on Deep Excavations, a group of papers included in the proceedings of this conference was selected for review. The papers were chosen since they highlighted important aspects of investigations and testing, theory, experimentation, design, or performance related to support of deep excavations in urban environments. Many papers have been included in this conference and within Session 6. Attempting to summarize all the session papers was considered to neither add value to this conference nor result in a coherent and meaningful General Report. By their omission in this report does not suggest that other papers are not valuable contributions to this conference and the topic of deep excavations. Rather, each paper is valuable in its own right and the reader is encouraged to examine them all.

2.1 *Investigations and testing*

Herbschleb et al. (2005) summarize the work undertaken to define the ground conditions for the station excavations associated with the new Amsterdam North/Southline. A large number of in situ and laboratory tests were completed to identify the stressstrain characteristics of multiple soil types for different stress paths. These test results were then used to form the basis of sophisticated numerical modeling to predict the response of the excavations and nearby displacement-sensitivity historical structures.

Following significant research, the authors selected a non-linear "hardening" constitutive model with consideration of *both* loading and unloading stiffness stress-paths to represent soil behavior. Field and laboratory testing was completed using a variety of methods to obtain parameters consistent with the selected constitutive model. Importantly, sufficient testing was completed so that geotechnical parameter *variations* could also be adequately characterized. Statistical analyses of test results were also used to subdivide geologic units.

Rodriguez (2005) presents a case history in which geophysical testing was initially completed to support a seismic site response study for a new commercial development. Down-hole and cross-hole seismic geophysics were used to determine shear wave velocity (V_s) profiles from which maximum shear modulus,