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 account for 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 Rod.
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 11 m 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 fibre-glass 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 over lain 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.
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.

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...
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 ($\sigma_0$) while tailskin grouting pressures were between 110 and 150% of $\sigma_0$.

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 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 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 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
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 linings 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 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.)](image)
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, Ohtia, 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, $\xi$ (function of $N$ and bending stiffness $EI$) and relative stiffness, $\zeta$ (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 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 of volume loss, which can be directly linked with tunnel machine performance, thus providing a continuous assessment of the works.

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