*Geotechnical Aspects of Underground Construction in Soft Ground – Ng, Huang & Liu (eds)*


# Physical and numerical modelling

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ABSTRACT: This General Report has been prepared based on a review of twenty papers submitted to the session on "Physical and numerical modelling" related to geotechnical aspects of underground construction. The papers cover a wide range of model feature types in different materials. The problems studied include ground/tunnel face stability, ground/tunnel deformation and earth pressures, ground/tunnel-structure interaction, seismic behaviour, and vehicle fires in a road tunnel. This report highlights and discusses the approaches used in modelling and presents the key findings. Some remarks are given at the end on the objectives of modelling and the work of TC28.

## 1 INTRODUCTION

20 papers have been submitted to this session (Table 1). Three of the papers are joint contributions from authors of two countries.

The papers cover a wide range of model feature types (Table 2). These include tunnels in clay, sand, aluminum rods (modelling a granular mass), layered soils, as well as tunnels in soft or weak rock. There is a paper on modelling of deep excavations with stepped-twin retaining walls, and a paper on vehicle fires in a road tunnel.

## 2 PHYSICAL MODELLING

Eleven papers present results of physical modelling (Table 3). These include six models at 1 g, a photoelastic model and four centrifuge models.

Table 1. Geographic distribution of the papers.

| Country | Papers |
|---|---|
| China | 5 |
| China/France | 1 |
| Denmark | 1 |
| France | 1 |
| Italy | 1 |
| Italy/UK | 1 |
| Japan | 2 |
| Japan/UK | 1 |
| Korea | 4 |
| The Netherlands | 1 |
| UK | 2 |
| | 20 |

For the three models that use aluminum rods, the tests were carried out at 1 g. Numerical modelling was also carried out to compare with the results of the 1 g tests.

The sand model at 1 g was prepared by compaction of the sand using a plate vibrator. The compaction could have created locked-in compaction stresses on the model braced wall and the adjacent tunnel thus influencing the model test results. This was not discussed in the paper.

Two papers present results of modelling of rock tunnel problems using 1 g tests. One used barite powder, sand and plaster mixed with water, and the other used concrete bricks to model the soft rock. There was some discussion on the modelling laws in the papers. While the conclusions on the qualitative behaviour seem reasonable, and are not unexpected, it is not sure if the quantitative results are valid at prototype scale where discontinuities in the rock and the higher stress levels could influence the magnitude of the deformations.

Table 2. Feature types covered in the papers.

| Feature type | Papers |
|---|---|
| Tunnels in: | |
| (a) Clay | 5 |
| (b) Sand | 3 |
| (c) Aluminum rods/crushed glass | 4 |
| (d) Layered soils | 3 |
| (e) Soft/weak rock | 3 |
| Deep excavation (aluminum rods) | 1 |
| Vehicle fires in road tunnel | 1 |
| | 20 |

Table 3. Papers on physical modelling.

| Technique and scale | 2D/3D | Materials used | Papers |
|---|---|---|---|
| Laboratory 1 g model (scales: 1/10, 1/19, 1/80) | 2D | Aluminum rods | 3 |
| | | Sand | 1 |
| | | Barite powder/ sand/ plaster | 1 |
| | | Concrete bricks | 1 |
| Photoelastic models | 2D | Crushed glass | 1 |
| Centrifuge models (75 g, 100 g, 160 g) | 3D | Clay | 1 |
| | | Sand | 2 |
| | | Sand overlying clay | 1 |

There are four papers on centrifuge tests. One of the papers is to study the effects of pile loading on an existing tunnel in an overconsolidated clay, two are on centrifuge tests where dry sand was used to construct the models for studying the interaction mechanisms, and one on tunnelling in an overconsolidated clay overlain by sand under the water table.

## 3 NUMERICAL MODELLING

18 papers present numerical modelling results (Table 4). Different numerical modelling techniques were used.

14 out of these 18 papers used either 2D or 3D codes based on the finite difference method (FDM) or the finite element method (FEM). Some of the codes, e.g. CRISP, FLAC and PLAXIS, are well established codes and the 2D versions are commonly used in current engineering practice. In the analyses, the soil was modelled either as a linear elastic or an elastic-perfectly plastic material with the Mohr-Coulomb or Drucker Prager failure criterion. Where a comparison was made, the elastoplastic model performed better than the elastic model.

In one paper, a slope stability analysis program SLOPE/W based on the limit equilibrium method was used to compute the factors of safety of a clay slope. The results were compared with the results of FLAC and PLAXIS which used the strength reduction method. However, no information is given on the theoretical method used (a few options are available in SLOPE/W such as Janbu, Bishop and Morgenstern & Price) and the choice of slip surfaces, which could affect the computed safety factors. Also, no information is given on what the slope deformation and the soil shear strain were, when the soil strength is reduced for the factor of safety to approach unity.

A visco-elastic model adopting a nonlinear relationship between the normalized shear modulus (and damping ratio) and the shear strain amplitude was used for a 1D ground dynamic shear response analysis. The code EERA was used for the analysis, the objective of this study was to "calibrate" a linear visco-elastic, effective stress based, constitutive model for use in coupled 2D dynamic analyses using the finite element program PLAXIS. The viscous damping was accounted for using the Rayleigh formulation (Woodward & Griffiths, 1996).

The subloading tij finite element model (developed by Nakai & Hinokio (2004)) was used in two cases to provide results for comparing with physical modelling at 1 g which used aluminum rods in the model tests. The tij model takes into account the influence of the intermediate principal stress by introducing a modified stress tij. Also, the subloading concept (proposed by Hashiguchi (1980)) is adopted to model the influence of soil density. Five of the seven parameters in the tij model are the same as those in the Cam-clay model, with one more parameter added to describe the influence of soil density and confining pressure, and another parameter added to characterize the shape of the yield surface. Laboratory biaxial tests were carried out to compare the stress-strain curves obtained from the finite element program FEMtij-2D. In the biaxial tests, shearing of the aluminum rods, which had low friction angles, induced dilatant behaviour. The match between the biaxial tests and the finite element analysis results appears reasonable but this is up to a shear strain level of about 1–2% only (Figure 1).

The Distinct Element code UDEC was used in one case to compare with the results of large-scale model tests carried out using concrete bricks to model rock. However, the paper does not indicate how the discontinuities in the rock were modelled. For the other two papers on tunnels in rock, the numerical simulations were carried out using finite element codes adopting an elastoplastic rock model with the Drucker-Prager failure criterion. It seems that the need for modelling the discontinuities that may be present in the rock was not considered. It is not too clear from the two papers how the rock parameters were determined for the continuum models and the field prototypes.

Results obtained from closed form solutions derived using upper bound limit analysis were presented in two of the papers, for comparison with the results of centrifuge modelling and numerical modelling respectively.

The Fire Dynamics Simulator code incorporating a large eddy simulation model was used to carry out computational fluid dynamics modelling. The objective of this work was to study the heat release rates from vehicle fires in a road tunnel of 15 m in diameter. The computed results were compared with an empirical equation. This indicates that the empirical equation requires improvement for the case of small fires in road tunnels with a large cross section.

Table 4. Papers on numerical modelling.

| Constitutive law | Modelling | Program | Papers |
|---|---|---|---|
| Linear elastic | 2D FEM | PLAXIS | 2 |
| Nonlinear visco-elastic | 1D shear | EERA | 1 |
| Elasto-plastic | 2D FEM | CRISP, Msc.MARC | 2 |
| (Mohr Coulomb) | 2D FDM | FLAC | 1 |
| | 3D FEM | MIDAS-GTS | 1 |
| | 3D FDM | FLAC3D | 3 |
| Elasto-plastic (Drucker Prager) | 3D FEM | MARC | 2 |
| Elasto-plastic (Cam clay + 2 parameters) | 2D FEM | FEMij-2D | 2 |
| Distinct element | 2D DEM | UDEC | 1 |
| Rigid-plastic | Limit analysis | Closed form solution | 2 |
| Large eddy simulation | CFD | Fire Dynamics Simulator | 1 |

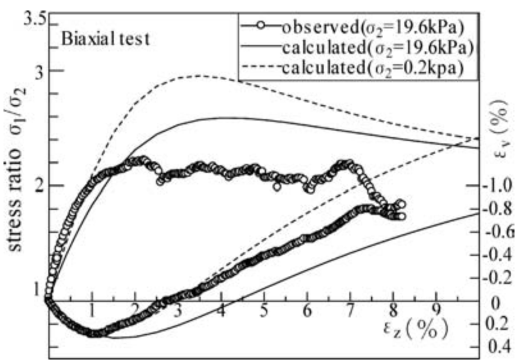



Figure 1. Stress-strain-dilatancy relation.

## 4 PROBLEMS STUDIED

The problems studied as reported in the papers include:

1. ground/tunnel face stability (5 papers),
2. ground/tunnel deformation and earth pressures (8 papers),
3. ground/tunnel-structure interaction (5 papers),
4. seismic behaviour (1 paper), and
5. vehicle fires in a road tunnel (1 paper).

A brief review of selected papers is given below.

### 4.1 *Ground/tunnel face stability*

The subject of face stability is a very important one. If the face pressure applied is too low, there could be a collapse or excessive ground settlement, and if the face pressure is too high, there could be a blow-out failure or excessive ground heave.

A number of researchers have studied this problem (e.g. Anagnostou & Kovári, 1994). The following papers have added to the knowledge base.

Li et al investigated the failure of a large slurry shield-driven tunnel using upper bound limit analysis and numerical modelling. The study is for the 15.43 m diameter Shanghai Yangtze River Tunnel constructed in soft clay. A shallow ground cover section, with a ground cover to tunnel diameter (C/D) ratio of 0.7, was selected for the study. Undrained conditions were assumed in the modelling. A multi-block failure mechanism with a uniform face pressure (suggested by Soubra, 2002) was used for the limit analysis. FLAC3D was used for the numerical modelling (which adopted an elastic-perfectly plastic constitutive model with a Mohr-Coulomb failure criterion). The results of the upper bound limit analysis and the 3D numerical modelling showed that partial blow-out failure of the upper part of the tunnel face occurs when the slurry pressure is large, whereas global collapse of the whole tunnel face occurs when the slurry pressure is small (Figure 2).

The authors noted that the difference between the slurry pressure and earth pressure at the crown and invert for a large diameter slurry TBM tunnel can be large and this could have a significant effect on the failure mechanism and the critical slurry pressure. The failure mechanisms and the critical slurry pressures at the tunnel axis level obtained from the limit analysis and the numerical modelling agree well with each other (Figure 3).

Caporaletti et al reviewed the past research on tunnel stability in undrained conditions (Davis, et al, 1980; Kimura & Mair, 1981; Sloan & Assadi, 1992), in drained conditions (Atkinson & Potts, 1977) and in layered ground (Grant & Taylor, 2000). They conducted centrifuge tests to investigate the stability of a circular tunnel in layered ground, with clay overlain by a medium dense sandy layer, below the water table. The C/D ratio of the tunnel was 2.38. The clay was consolidated from a slurry, to give an overconsolidation ratio ranging between 1.4 and 2.8 with depth. All tests

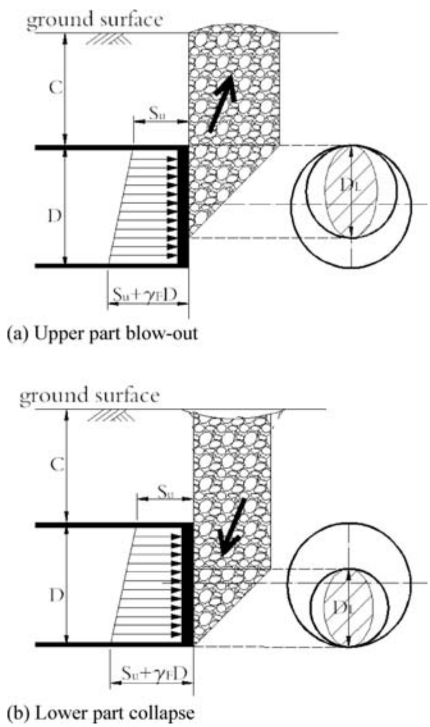



Figure 2. Two kinds of partial failure mechanisms.

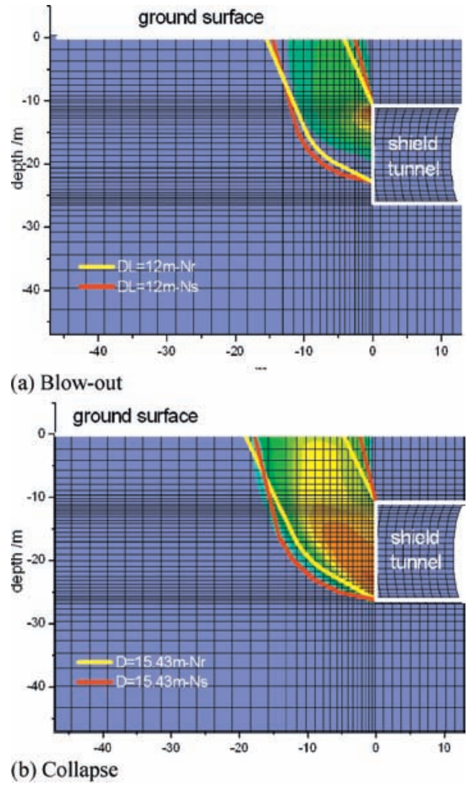



Figure 3. Comparison of failure mechanisms of Case 2 (velocity contour for $FLAC^{3D}$ analysis).

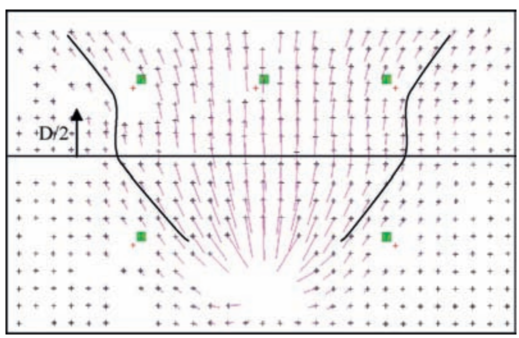



Figure 4. Mechanism of failure from centrifuge tests (VL ≅ 20%).

were carried out at 160 g. The condition of tunnel collapse was taken as volume loss greater than 20%. In the centrifuge tests the mechanism of failure for the layered ground involved a wide area of soil both in sand and in clay, with pseudo-vertical settlements at the sand-clay interface (Figure 4).

It was found that the contribution to stability due to friction acting within the upper sand layer represented a significant contribution. A significant overestimate of the tunnel support pressures to prevent collapse might result if the theoretical solutions obtained for homogenous clays are used with the sand layer treated as a surcharge. The authors proposed a new failure mechanism which provided an upper bound to the experimental data obtained (Figure 5). It would be interesting to examine whether the proposed mechanism is applicable for the case of a loose sand layer.

Date et al carried out a series of centrifuge tests at 75 g to investigate the ground deformation patterns during excavation of tunnels in dry sand. The C/D ratio of the model tunnels was one, and some of the models incorporated reinforcements. The ground deformation was found to be small even when the face pressure was reduced to half the initial pressure of 100 kPa, but once movement started upon further reduction of the face pressure it increased sharply leading to "instantaneous" collapse (i.e. a brittle failure).

The model tests without reinforcement collapsed at a support pressure which agrees with the centrifuge test results of Chambon & Corté (1994). The study found that introduction of face bolts and forepoling yielded different tunnel collapse mechanisms, which depended on the density of the face bolts and forepoling bolts. Surprisingly, the reinforcements contributed

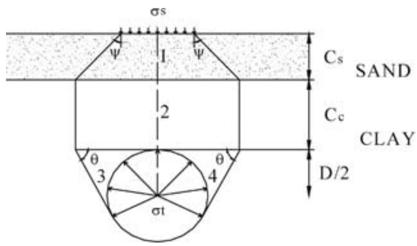



Figure 5. Mechanism of failure for layered ground.

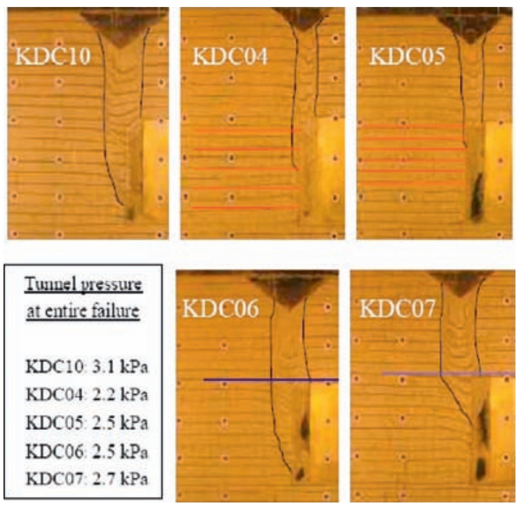



Figure 6. Tunnel failure patterns on the longitudinal section.

to only a slight reduction in the support pressure required to keep the tunnel face stable, compared to the case without reinforcement. The face bolts installed stiffened the ground ahead of the face and were found to be able to reduce the face extrusion. The forepoling divided the ground around the tunnel face into two zones, with the outer zone forming an arch comprising the forepoling bolts. The geometries of the collapse mechanisms are similar to those observed by other researchers for tunnels in sands, e.g. as reported by Chambon & Corté (1994) and Mair & Taylor (1997). They all involve a narrow "chimney", propagating almost vertically from the tunnel up to the ground surface (Figure 6).

FLAC3D analyses were also carried out. The Mohr-Coulomb soil model with strain softening/hardening was found to give a better match to the centrifuge data than the Mohr-Coulomb model without strain softening/hardening. The deformation pattern obtained from the analysis for a model reinforced with face bolts was similar to that of the centrifuge test but the magnitude was smaller. The authors recommended to study further the effect of mesh shape and the effect of changes

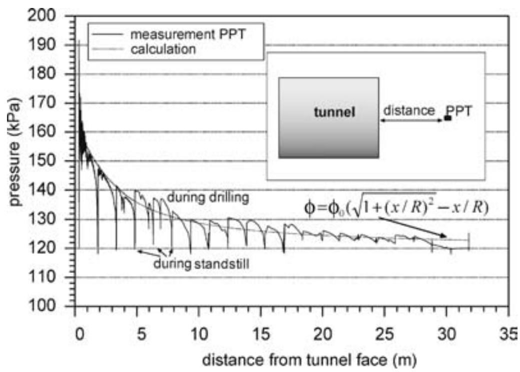



Figure 7. Measured excess pore pressure in front of a slurry shield and approximation.

in soil-bolt interaction properties upon excavation in the numerical analyses.

The information on failure mechanisms presented in the above papers is interesting and useful. There is recent improvement in understanding of the ground-tunnelling interaction processes associated with influence of grouting pressures, removal of the filter cake and the pore pressures generated during the advance of a slurry TBM (Figure 7). This was achieved through field measurements obtained during construction (Bezuijen & Talmon, 2008). Further data and study in this area will no doubt augment the results of existing laboratory and analytical modelling, which have not accounted for such processes. Further understanding of the processes could help to evaluate the need to refine the calculation models and design methods for estimation of face pressures required to prevent collapse and blow-out.

### 4.2 *Ground/tunnel deformation and earth pressures*

A number of papers in this session present results of modelling to study the ground deformation and earth pressures around a tunnel.

Shahin et al developed a new circular tunnel apparatus and conducted 1 g model tests to examine the ground movements induced by tunnelling and the earth pressures around the tunnels. Aluminum rods were used to model a granular soil mass. The surface settlement was measured using a laser type displacement transducer with an accuracy of 0.01 mm, and photographs were taken during the experiments which were later used as input for the assessment of the ground movements using the Particle Image Velocimetry technique (White et al, 2003). To compare with the model test results, numerical simulations were carried out using 2D finite element analyses under plane strain and drained conditions. The computer program FEMtij-2D was used. The initial stresses

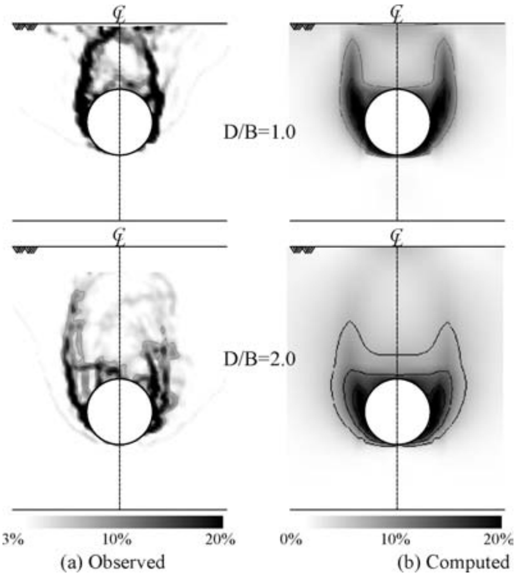



Figure 8. Distribution of shear strain: tunnel invert is fixed.

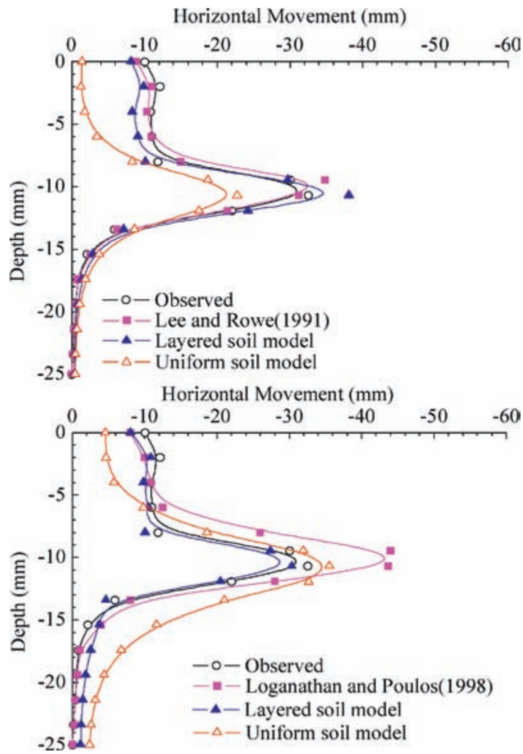



Figure 9. Lateral displacement 15 m behind the tunnel face.

applied correspond to the self-weight condition. Two C/D ratios, viz. 1 and 2, were examined. The effects of full face excavation (with the centre of the excavation kept fixed) and top drift excavation (with the invert of the tunnel kept fixed) were also studied. The surface settlement and earth pressures around the tunnel were found to be significantly influenced by the displacement at the tunnel crown for the same overburden and same volume loss. The volume loss was less significant compared to the crown drift in the case of the shallow tunnel. The full face excavation case produced a wider shear deformation region than that for the case of top drift excavation (Figure 8). The use of an elastoplastic soil model produced better match with the model test surface settlement profile than an elastic soil model. The distribution of earth pressures around the tunnel depended on the excavation pattern. The authors indicated that the numerical simulations were generally in good agreement with the model test results. However, it is no clear whether the tij finite element model is capable of describing the behaviour of tunnels constructed in real soils especially in soils which exhibit contractile behaviour.

Liang et al studied the effects of soil stratification on tunnelling-induced ground movements. 3D analyses were carried out using the computer program FLAC3D. The behaviour of the 2.47 m diameter Thunder Bay sewer tunnel in Canada, constructed using a TBM with segmental concrete lining, in soft to firm clays with silt and sand seams, was simulated. The C/D ratio of the tunnel was 3.8. The soil strata were divided into four sub-layers for the purpose of the analyses. The ground surface settlement, lateral displacement profile at 15 m behind the tunnel face and the subsurface settlement with depth above the tunnel axis from the analyses were compared with the analysis results obtained by Lee & Rowe (1991) using the FEM3D program (also based on an elastoplastic soil model). They were also compared with the field data reported by Belshaw & Palmer (1978). Additional comparisons were carried out with the analytical solution given by Loganathan & Poulos (1998). The study showed that the elastoplastic soil model could simulate the deformation profiles better than those based on the elastic model. The results of the elastoplastic soil model indicated that soil stratification had little effects on the ground surface settlement but significantly influenced the lateral displacement and subsurface settlement profiles (Figure 9). This was different from the elastic soil model which predicted that soil stratification had significant effects in all cases. This is an interesting case history of benchmarking a 3D computer program using data from a past project, illustrating the value of documenting good data and making it available for research.

Song et al studied the time-dependent behaviour of soft ground tunnels constructed using steel reinforcements grouted into the ground ahead of a tunnel

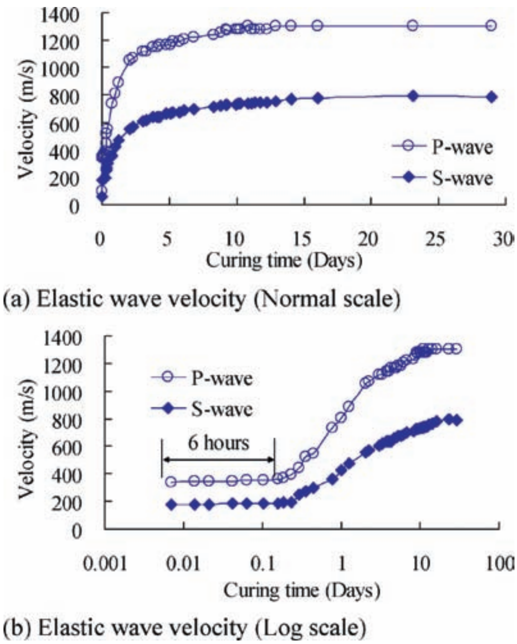



Figure 10. Time-dependent characteristics of elastic wave velocities of a sand-cement mixture.

(a technique which the authors called the "reinforced protective umbrella" method). Laboratory direct shear tests and P and S wave velocity tests (using piezoelectric bender elements) were carried out to determine the strength and stiffness of the sand-cement mixture at different curing times. The test results showed that the sand-cement mixture gained significant increases in stiffness after about 6 hours whereas the apparent cohesion increased to about 2 MPa after 7 days (Figure 10).

3D finite element analyses were carried out using a computer program MIDAS-GTS (2005) to simulate the behaviour of such a tunnel. The tunnel is 18.8 m wide and 10.4 m high, at 15 m below ground. It was constructed in weathered rock, using 12 m long steel pipes as reinforcement. The water table was at ground surface. The analyses incorporated the time-dependent material properties of the sand-cement mixture. The excavation rate was taken as 0.75 m per day. The study concluded that use of the 2–3 days strength and stiffness parameters was adequate for predicting the time-dependent deformation behaviour, for practical design purposes, provided that there is sufficient overlap between the reinforcements. No comparison with any field performance monitoring results was however presented.

Lee et al studied the behavior of a 2-arch rock tunnel using a large-scale test machine (6 m wide × 6.5 m high). The model tests (1/19 scale) were conducted at 1 g. The rock was modelled using concrete bricks. The tests showed that the ground displacements induced by tunnelling were mainly within a zone of 0.25D from the tunnel, where D is the tunnel width. Horizontal displacements of more than 40% and vertical displacements more than 20% of the total displacements occurred during excavation of the pilot tunnel. The authors suggested that the stability of the 2-arch tunnel could be dominated by the stability of the pilot tunnel excavation and that the rock bolt length should be longer than 0.25D. Displacements obtained from UDEC analyses were presented. While these showed the same pattern, details of the analyses were not given. Based on the limited measurements obtained, the authors suggested that the rock load acting on the centre pillar of the 2-arch tunnel may be taken to be 0.15W for preliminary design, where W is the centre-to-centre distance between the left and right tunnels, when the RMR of the rock mass is more than 60. No numerical analyses were carried out. More research was recommended to confirm the proposed empirical relationship. It would be useful to examine the influence of rock discontinuities and the effect of rock block size relative to the tunnel diameter.

### 4.3 *Ground/tunnel-structure interaction*

Broere & Dijkstra investigated the influence of tunnel volume loss on piles using the photoelastic technique. 2D plane strain model tests were conducted to examine the tunnel-pile interaction. Crushed glass (a photoelastic material) was used to model the soil. The effects of volume loss were simulated by making the tunnel diameter contract vertically. From the tests, it was found that significant stress changes occurred close to the pile tips. The tests with a volume loss of 0.6% showed a clear influence of the volume loss on the stresses near the pile tips up to one tunnel diameter away. The study suggested that the influence zone for displacement piles with both end bearing and skin friction, might be slightly larger than for bored piles with end bearing alone. The authors indicated that further field observations, model testing and numerical modelling are required to determine the influence zone.

Lee & Yoo studied the ground shear strain patterns developed around a tunnel and the existing piles nearby due to tunnel construction. Small-scale laboratory model tests at 1 g were conducted. Aluminum rods were used to model the soil mass and the piles embedded in it. A tunnel diameter reduction system capable of achieving a tunnel volume loss of up to 20% was specially developed. The strained controlled tests carried out using this system resulted in ground shear strains which were captured by close range photogrammetry. 3D numerical analyses were also carried out using the finite element program CRISP. Comparison between the physical model tests and the finite element analyses showed good agreement in terms of shear strain patterns. Based on the maximum shear

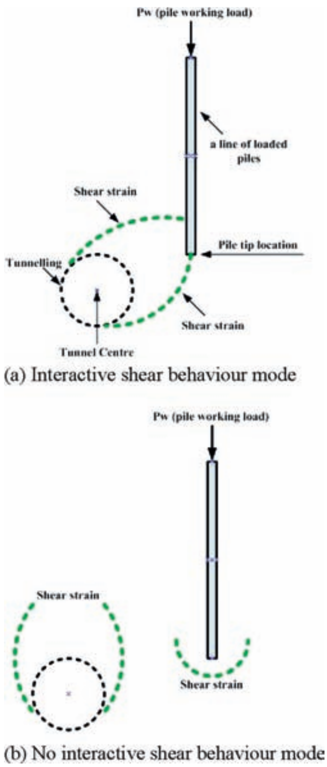


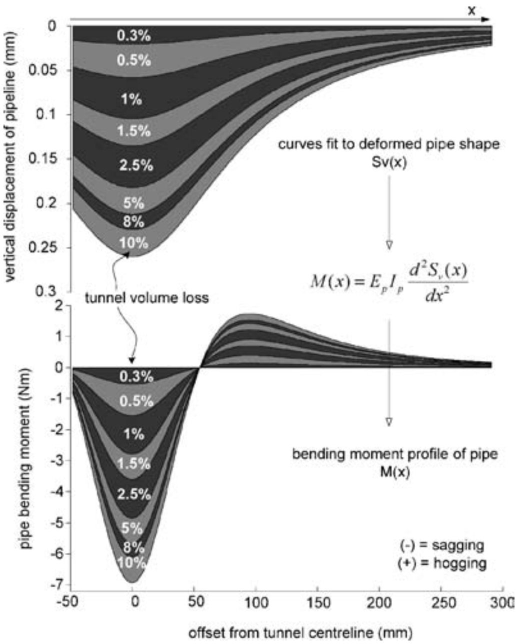

Figure 11. Schematic illustration of shear strain modes for pile-soil-tunnelling interaction.

strain contours, two distinct shear strain patterns were observed, viz. with and without tunnel-pile interaction (Figure 11). The boundary between these two modes of behaviour depended on the location of the pile tip from the tunnel and the magnitude of the tunnel volume loss. The authors suggested that this boundary might serve as a useful guide in the planning the tunnel alignment in areas where piles are present. It may be worthwhile to compare the results given in these papers with the findings of Jacobsz et al (2004; 2005) from centrifuge tests and Selemetas et al (2005) from field tests.

Yao et al studied the effects of loading of bored piles on existing tunnels. Centrifuge model tests were carried out at 100 g. The model tunnel was formed in firm to stiff clay consolidated from a slurry. The tunnel lining deformation, pore pressures in the clay, pile load applied, pile settlements and tunnel face pressures were monitored while the pile loading was being applied. Two C/D ratios, viz. 2 and 3, were studied. The tests examined the behaviour after pile construction. The influence of pile excavation was not considered. In the tests the pile base was set at two different positions: tunnel crown and invert level. The rate of loading was designed to create undrained conditions. Preliminary analysis of the results indicated that the pile settlement had a linear relationship with increase in applied load when the load exceeds half the designed ultimate load. The tunnel centre always moved downwards and away from the pile. Increasing the pile-tunnel clear spacing reduced the deformation of the tunnel lining. The long pile had more effect on the tunnel lining than the short pile regardless of the C/D ratio. The tunnel crown was always subject to significant movement due to pile loading.

Marshall & Mair investigated the soil-structure interaction mechanisms resulting from tunnel construction beneath buried pipelines using centrifuge modelling. The study aimed to validate visually the interaction mechanisms that account for pipeline behaviour. Particle Image Velocimetry was used to measure displacements for characterising the soil-structure interaction. The model tests were carried out at 75 g, using sand prepared to a relative density of 90%. The C/D ratio of the tunnel was 2.4. The study showed that estimation of the tunnel volume loss (defined as change in tunnel volume divided by the original total tunnel volume) using soil displacement data was not simple for sands. This was due to the uncertainty on the extent of the dilation and contractile behaviour of the sand around the tunnel. The soil volume loss (defined as the volume calculated by integrating the soil settlement profile and dividing by the original total tunnel volume) was not always the same as the tunnel volume loss. The magnitude of the former calculated at the ground surface can be greater or less than the latter. The centrifuge pipeline test illustrated that a gap formed below the pipeline at a tunnel volume loss of between 1 and 2%. The gap grew as the tunnel volume loss increased. The bending moments induced in the pipe increased from the onset of tunnel volume loss but did not appear to be sensitive to the growth of the gap height (Figure 12).

Lee & Kim studied the behaviour of a braced excavation in sand adjacent to a tunnel using large-scale (1/10 scale) model tests at 1 g. The braced wall was subjected to preloading to limit the wall deflections during the ground excavation. The tunnel was at a distance of half the tunnel diameter from the braced wall. The sand was prepared to a relative density of 56%. 2D numerical analyses were carried out using the finite element program PLAXIS. It is not clear what constitutive model was used for the sand. The study found that if the wall deflections were significantly reduced by preloading, the stability of the adjacent tunnel would greatly increase. The maximum bending moment and shear force in the tunnel lining decreased due to the preloading. The ground surface settlement also decreased as a result of preloading. The wall deflection profiles from the model tests agreed well with the numerical analysis results. It is noted that the




$$M(x) = E_p I_p \frac{d^2 S_v(x)}{dx^2}$$

Figure 12. Derivation of bending moments from deformed shape of pipeline (from PIV data).

sand was compacted to construct the models. However, it is not clear what initial soil stresses were used in the numerical analyses. Also, no information is given on whether the wall installation and excavation sequence in the analyses matched those in the model tests.

## 5 CONCLUDING REMARKS

In the papers submitted to this session, the modelling objectives are generally not explicitly stated but they probably include one or more of the following objectives:

- To observe/understand collapse mechanisms
- To observe/understand deformation patterns and interactive behaviour
- To assess/verify the usefulness or accuracy of theoretical solutions, software or empirical rules against laboratory (1 g or Ng) model test data
- Same, but against field measurement data
- To benchmark theoretical solutions or software
- To predict field performance

Other than to gain knowledge and to understand the problem, an important goal of the modelling research should be to provide useful and reliable tools or to enhance the existing tools for prediction of field performance, for use in engineering practice. In this regard, some of the papers have contributed to this goal.

TC28 has recently set up two working groups, one on databases on underground works and another on preparing guidelines for comparing field or physical modelling with numerical simulations. The first initiative will be useful for modellers in that good quality data will be archived systematically for easy reference and retrieval, physical modellers could use the information to plan their research and check their model test results against others' work for benchmarking purpose, and numerical modellers could use the data to check the reliability and limitations of the existing theoretical closed form solutions and numerical codes. The second initiative will be useful for those who are carrying out modelling to understand real behaviour or to validate numerical codes.

One of the databases on underground works could include failures observed in model tests and failures in actual projects. Data in the latter category are more difficult to obtain unless there is a forensic investigation and the information is subsequently made publicly available. Information on failures would be invaluable to provide lessons learnt, for calibration of design methods and for providing insights for risk management. If this is to proceed, then there may be merit to collect data on the size of the failure influence zones for different ground and geometrical conditions, and also the time for any cavity created at depth to migrate to the ground surface. Such information is potentially useful for risk management, in particular, for preparing monitoring plans, planning of risk mitigation measures, and preparation of emergency preparedness and contingency plans.

Another database will be on monitoring results. For such databases, the monitoring data should be accompanied by the necessary data on ground and groundwater conditions, the way the soil and rock parameters was measured and interpreted, the method of wall installation, information on ground treatment and the sequence of construction. Such data would allow numerical modellers to check the capabilities and limitations of the existing computer programs. From a practising engineer's point of view, it is often not practical to use overly sophisticated software requiring multiple parameters to characterize the soils for design. This is because of the cost, time and difficulties in obtaining high quality ground investigation data, the uncertainties associated with modelling the ground and the hydrogeological conditions (including the boundary conditions), the effort needed to model the range of design situations and to carry out sensitivity analyses, the need for having relatively simple tools for undertaking design reviews in a timely manner during construction, the difficulties in incorporating effectively the wall installation and ground treatment effects in the analyses, and the lack of competent personnel in the use of sophisticated codes and checking of the computed results from such codes.

There is a lack of systematic comparison on the results obtained from sophisticated software with those from less sophisticated ones. The availability of good quality monitoring data and benchmarking of the existing numerical codes using good quality monitoring data could help to addresses some of these issues.

TC2 on physical modelling in geotechnics has similar initiatives on databases (see http://www.tc2.civil.uwa.edu.au). Cross committee communication will create synergy.

## ACKNOWLEDGEMENTS

The author is grateful to Paul Wu for his assistance in preparing this General Report.

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