Underground constructions in soft ground – general report of the TC204

Construction souterraine en sols mous - rapport général du TC204

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ABSTRACT: This paper presents the general report of the TC204 session "Underground Constructions on Soft Ground" of the 19th International Conference on Soil Mechanics and Geotechnical Engineering. A total of 32 papers were assigned for this session, they were divided in five general topics and reviewed briefly in this report. Apart from the regular topics of the TC204, this session also presents papers on rock tunneling. This report is intended to provide a general overview of the papers of the TC204 session. However, for a better understanding of the presented contents, the readers are encouraged to look for the papers themselves in the proceedings.

RÉSUMÉ : Ce document présente le rapport général de la session TC204 "Travaux souterrains en sol meuble" du 19e congrès International de Mécanique des Sols et de Géotechnique. Au total, cette session comporte 32 communications, qui ont été réparties en cinq thèmes généraux et sont brièvement analysées dans le présent rapport. Outre les thèmes du comité TC204, cette session présente également communications concernant les tunnels au rocher. Ce rapport est destiné à fournir une vue d'ensemble des communications de la session TC204. Pour une meilleure compréhension des contenus présentés, les lecteurs sont encouragés à consulter les articles complets dans les comptes-rendus.

KEYWORDS: TC204, underground construction.

1 INTRODUCTION

The papers for the TC204 session "Underground Constructions in Soft Ground" on the 19th ISSMGE conference show the on-going development of this broad field, covering several important topics, proposing solutions for different problems and tracing the way for future research. 32 papers from 19 nations have been allocated in the TC204 session (Figure 1).

In this report, the papers have been divided for their connection to two types of underground structures: tunnels (22) and deep excavations (10). The tunneling papers are organized for their stage in the construction lifecycle: Design & Modelling, Construction & Monitoring and Operation & Maintenance. The papers dealing with deep excavations are organized into Design & Modelling and Case Studies.

2 TUNNELLING

This topic gathered almost 70% of the papers in the session, and are all connected to bored tunnels. Both immersed and cut-and-cover tunnels are not discussed in this event.

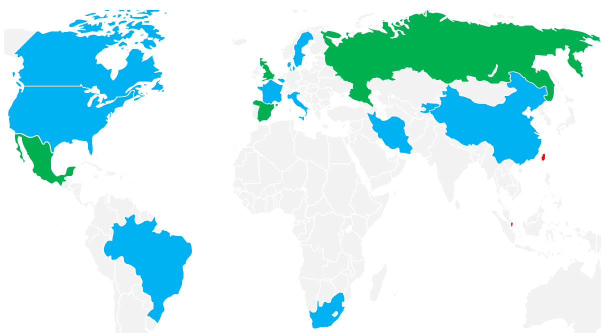

Figure 1. Map of contributing nations (**6** - Singapore, **5** - Chinese Taipei; **2** - Mexico, Russia, Spain, UK; **1** - Belgium, Brazil, Canada, China, Denmark, France, Hong Kong, Iran, Italy, Kyrgyzstan, South Africa, Sweden, USA)

2.1 Design and Modelling

To plan all the design stages of a bored tunnel, one must consider a wide range of scenarios and technical challenges to complete the project. The paper from **Asanbekovnam et al.** entitled "Design features of transport tunnels in difficult physical-geographical conditions of Kyrgyzstan" exemplifies this planning stage, specifically for mountain territories where seismic conditions must be considered in the design. Possible obstacles of these regions are areas of landslides, rockslides, avalanches and snow drifts watercourses. The authors report on experience from the Fergana Ridge, where the measured stresses in the rock were highly dependent on the extension and opening of faults, and the nature and kinematics of the tectonic zone. Empirical experience has set a limit of 10^{-4} m/year of horizontal displacement along the faults for the location to be considered suitable for a tunnel project.

The paper "Geotechnical engineering considerations for the analytical design of an adequate tunnel support system" from **Ongodia et al.** investigates which factors influence the design of the support system. The study focuses on the effective rock-support interaction, with special regard for the rock bolt. The parameters influencing this interaction are discussed through a gradual scheme of studies: Geology, material properties, geotechnical parameters, construction process. Figure 2 presents some typical unstable wedges described in the study. The paper also describes how the tunnel depth can influence the predominant types of failure. The authors report that shallower tunnels have a higher potential of tensile and shear failure, while deeper tunnels generally fail in compression. This is probably connected to the relative magnitude of stress anisotropy at different depths.

The third paper on broad design aspects was written by **Svensson and Friberg**: "GeoBIM - a tool for optimal geotechnical design". The BIM process, which involves the generation and management of digital models of physical and functional characteristics of projects, is rarely used in geotechnical projects, in contrast to building projects. This study explores why that is the case, how to change this scenario, and what would be the benefits of doing so. Figure 3 shows an example of the visual feedback of this process. The uses of the geotechnical data are divided in five stages: Storing during project, modelling, design, visualization, long-term management.

Emphasis was given to the fact that the BIM database should enable the user to handle all the available data, which requires it to be generic yet able to handle very detailed information regarding method specific parameters. Communication between the geotechnical modelling software and the database requires data formats that are compatible both ways, which is not always the case nowadays. The authors then present the GeoBIM database, developed in Sweden for underground infrastructure projects, and applied in four major projects in the country. The database is implemented in a cloud based PostgreSQL format, and can handle information from geotechnical sounding and sampling, rock cores, surface and borehole geophysics, laboratory tests, groundwater and contaminated soil investigations.

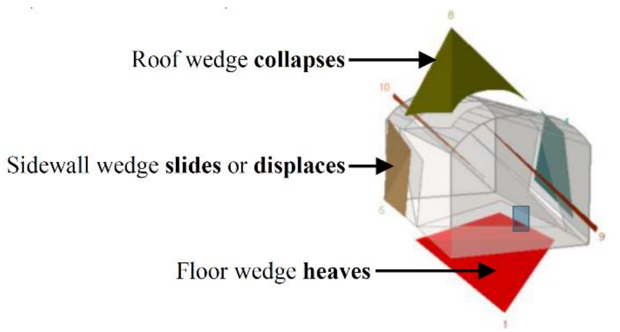

Figure 2. Typical unstable wedges in rock tunnels (from Ongodia et al.)

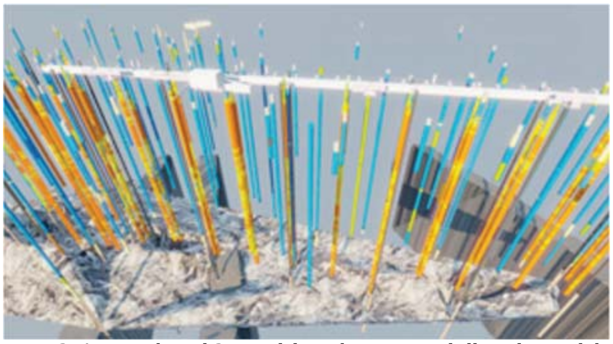

Figure 3. A geotechnical 3D model combining core drilling data and the structure geometry (from Svensson and Friberg).

From general design to specific analytical and numerical models, the following papers present attempts to simulate a tunnel construction. The paper "Numerical analysis of ground level settlement due to tunneling for intersecting tunnels" from Rashidi and Hamidi presents a 3D finite element model (FEM) to calculated the ground settlements above non–leveled intersecting tunnels. The upper tunnel was always excavated first. The software Plaxis3D was used to run the simulation, but it is not clear how the EPB excavation and support system were considered. Nevertheless, the authors report some key results: the settlements were inversely proportional to the distance between the two tunnels, and to the intersecting angle (from 45 to 90º), but the diameter of both tunnel had the highest impact on the magnitude of settlements.

Lee and Choi discuss, in their paper "Numerical Analysis of Cross Passage Opening for TBM Tunnels", the structural reactions of precast concrete segmental lining systems when cross passages are created. Segmental linings have several joints, which enables different modes of relative motion and failure. The open ring of a cross passage normally transfers part of its load to adjacent closed rings. In the case of openings spanning for more than one ring, it is normally necessary to use special segments and or a temporary steel bracing structure. The authors list several possibilities for the bracing structure, which are shown in Figure 4. Another tool is the bicone circumferential joint system, which helps to transfer the interface shear force between the opened and the adjacent rings.

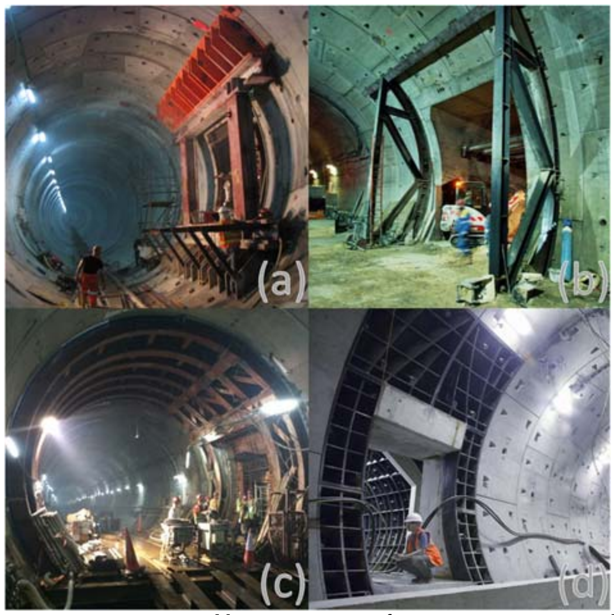

Figure 4. Temporary steel bracing structures for cross passages: a - steel lintel and sill beams; b - half-moon frame, c - full-moon frame, d - rectangular frame of steel segments (modified from Lee and Choi)

The authors present a case study of a railway project in Hong Kong with a 6+1 segmental lining system with bicones, 6.5 m internal diameter, 30 cm thick and 1.5 m wide. The required cross passage was 6.3 x 4.5 m, which required intervention in five consecutive rings that should be supported by a temporary steel hamster cage. The case was analyzed by a 2D FEM to determine the soil pressures, which are then applied to a 3D bedded beam spring model to obtain the reaction of the structural elements. The field measurements indicate that the lining ovalization was slightly overestimated, but the general profile shows good agreement.

The paper "A simple model for to introduce the rock deformability at hydraulic tunnels", from Oteo et al., takes the discussion to a different perspective on how to decide the appropriate values for an important constitutive parameter in hydraulic tunnels: the rock deformability. The author evaluates how the construction process, through the decompression zone it creates, alters the effective deformability of the rock. Based on the tunnel width (W), the authors report values for the thickness of the decompressed zone from 0 to 0.8.W, depending on the quality of the rock mass. They also point out that the traditional recommendations for the field rock deformability (as a ratio of the deformability obtained in the lab) are highly variable for low values of RMR (Rock Mass Rating), which is the case in several hydraulic tunnels. To tackle this problem, the authors review a range of cases where the field modulus could be back analyzed for low RMR values, and based on this the authors proposed new recommendations for the field deformability of the overall rock mass and of the decompressed zone around the excavation, which is presented in Figure 5.

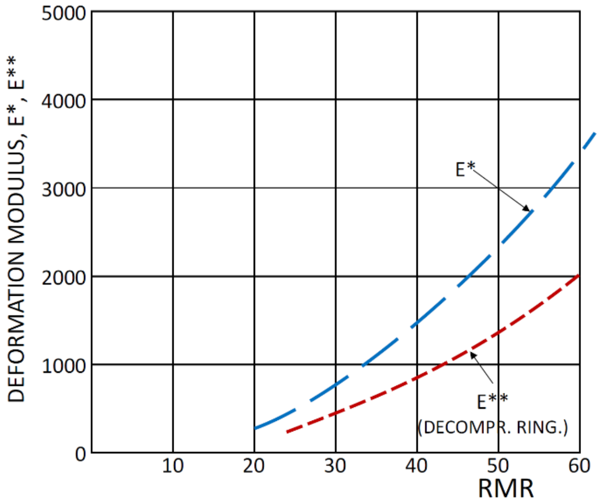

Figure 5. Field deformation modulus (MPa) for the rock mass E* and the decompressed ring around the tunnel E** (from Oteo et al.)

Diverging from the computationally demanding numerical methods, the paper "Analytical method to perform ground-lining interaction analysis for circular tunnels" from Angeles and Guichard presents the case that analytical solutions can also be used to achieve a robust tunnel design. The authors propose a series of steps, which can be briefly described as: 1. Decompression: estimates the stress release as the ratio between the ground loss parameter and the maximum excavation convergence; 2. Soil-structure interaction: From the stress state of the previous stage, the lining compression is calculated; 3. Consolidation: Calculates the reduction in pore pressure, change in lateral confinement, and negative friction on the lining. For long term behavior, a plastic flow factor is applied to the elastic modulus of the concrete. The first two stages can be calculated for both drained and undrained conditions. For each stage, a series of references to the analytical methods and equations are presented, but the reader is referred to the original paper. An example calculation shows how several aspects can be simulated in this framework. It is worth considering if the simplicity of the formulas and underlying assumptions balances out with the straightforward use and flexibility of the method.

Traditionally, the component of tunnel design that most relies on analytical solutions is the assessment of face stability, which is the focus on two papers from this session. "Tunnel face stability in heterogeneous soils considering partial collapse: the case of Madrid's Metro system" from Senent and Jimenez applies the idea of a rigid block mechanism to layered ground conditions, implementing variable properties and the possibility of partial collapse. The mechanism is defined incrementally from the contour of the tunnel face, based on a rotation axis. The proposed methodology was validated against the results of a FLAC^{3D} numerical model. The computed critical pressures didn't differ more than 2.5 kPa, and the shapes of the failure surfaces were also quite similar (Figure 6). The second validation step successfully reproduced an empirical relation, developed for the extensions of the Metro Madrid system (Spain), which relates the minimum thickness of a strong layer above the tunnel crown (assuming a soft soil deposit above it), and the content of fines in the soil (indirectly related to the soil strength).

The other paper "Prediction of critical distance for tunneling with water-filled cavity ahead of tunnel face" from Qin and Chian, presents an upper-bound limit analysis where the kinematically admissible mechanism reaches a water filled cavity. Through a parametric analysis the authors concluded that the supporting pressure was directly proportional to the tunnel diameter, water pressure and soil weight, but inversely proportional to the soil resistance and the distance between the tunnel face and the water cavity.

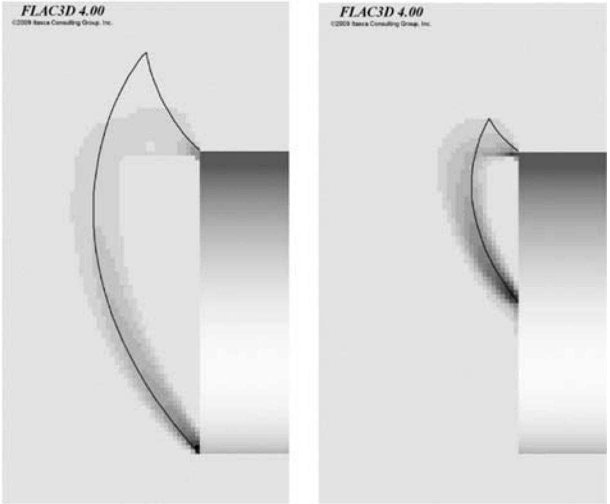

Figure 6. Comparison of failure mechanisms for two scenarios, highlighting the possibility of partial face collapse (from Senent and Jimenez)

Tunneling in soft ground often requires several soil reinforcement and improvement methods, which require specific design methodologies. The paper from Le and Taylor entitled "The reinforcing effects of Forepoling Umbrella System in soft soil tunnelling" presents a thorough evaluation of the relative effects of different parameters in a forepoling system. A series of eight centrifuge tests (125g) in clay were performed varying the tunnel depth, poles embedded length, center-center spacing, and filling angle. The tunnel cavity was supported by compressed air in a latex membrane lining the tunnel, and a 3D printed guide was used to insert the model poles into the sample at 1g (Figure 7).

The results indicate that the forepoling system reduces the surface settlement by up to 85%, and increases the tunnel stability ratio (considering the clay's undrained shear strength in triaxial extension) by up to 21%. Overall, it was also noted that the effect of the forepoling system became more significant when the tunnel support pressure was reduced. In terms of the relative effects, they report that an 80% increase in the bending stiffness of the steel pipe only yielded an improvement in settlement reduction of about 25%, indicating that the soil strength to support the poles is also very important. Overall, the embedded length and pole stiffness always improved the efficiency of the support system.

Regarding the mechanism, the most effective filling angle is directly connected to the relative tunnel depth. For shallow tunnels, most of the soil mobilization takes place at the roof, so a smaller angle is sufficient to provide support. On the other hand, deeper tunnels mobilize a larger region, reaching the springline, which require a larger filling angle.

Figure 7. High precision, 3D printed, insertion guide to simulate forepoling in small scale physical models (from Le and Taylor)

Tyagi and Lee (20) in their paper "Factors affecting the stability of large diameter tunnel in improved soil surround" focus on cement-threated soils and how their reduced permeability may define the failure mechanism. A series of plane-strain FEM calculations were performed using the Cam-Clay model to represent the soil and the Mohr-Coulomb model to represent the improved soil ring around the tunnel. The numerical results match previous centrifuge tests in terms of the location of tension zones (Figure 8), validating the methodology. From there, a parametric study revealed that a lower permeability in the treated soil causes a smaller rate of water pressure dissipation, which increases the occurrence of tensile failure. To quantify this process a dimensionless time variable is devised with the excavation time, consolidation coefficient and ring thickness. The authors show that the minor effective stress correlated well with this time variable, and can be used to predict when tensile failure is more likely. Based on these steps, the authors review a new stability ratio formula considering these consolidation effects. The results align in a clear correlation between the stability ratio and the relative thickness of the improved soil.

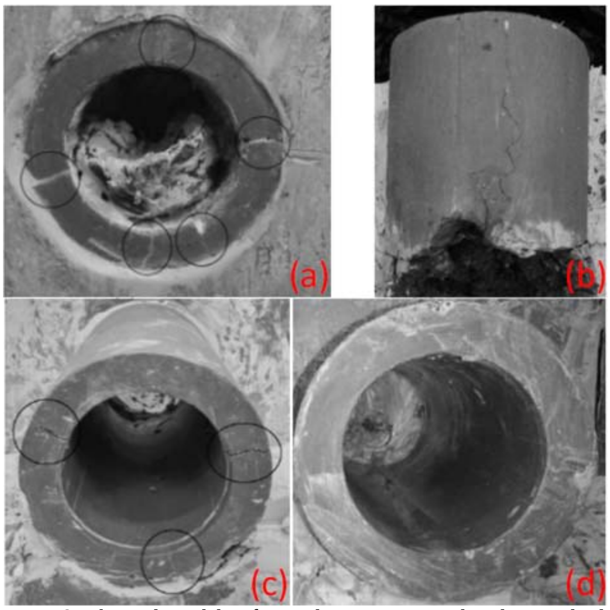

Figure 8. Physical models of tunneling in improved soils - Multiple cracks or rupture with roof collapse (a, b), Tension cracks near shoulders (c), No cracks (d) (modified from Tyagi and Lee)

While most studies are concerned with the ground settlements due to tunneling, these settlements tend to be a problem only if they disturb the surrounding structures. The paper from Dias and Bezuijen entitled "A design tool for pile tunnel interaction" presents a simple methodology to evaluate the settlements and stresses caused by tunneling near pile foundations. The study relies on a modified version of the load transfer method, where the pile reaction can be computed for both loading and unloading and the settlement variable is replaced by the relative pile-soil settlement. This enables the traditional method to consider the ground settlements along the pile, and the consequent stress redistribution along the pile shaft and at the pile toe. The authors present a simple example of a linear profile of settlements, highlighting how the method captures the interaction mechanisms while holding the equilibrium and continuity conditions. Then, the effects of typical settlement profiles from analytical solutions are assessed. The typical mechanisms of pile tunnel interaction that are reported in the literature could be reproduced quite well.

The last paper from this section offers new insights into the natural processes of excavation. The authors Frost et al., in their paper "Biologically-inspired insights into soil arching and tunnel stability from the topology of ant nests", explore how ants excavate soils in a very efficient way, avoiding obstacles and preserving particles that are critical for the stability of the nest. Discrete element modelling was used to simulate the removal of particles to create underground cavities, relating the cavity width to the maximum particle size. The authors analyze how a layout with two overlying rectangular cavities can be more stable, compared to a single cavity, as it reduces the vertical stress on the ceiling of the lower cavity (Figure 9). This effect was not observed with circular cavities, which might explain the elongated shape of chambers in ant nests.

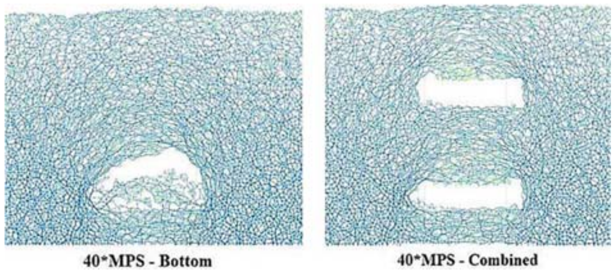

Figure 9. Force chains for single and double cavities (from Frost et al.)

2. 2 Construction and Monitoring

With the uncertainties and natural variabilities of geotechnical structures, it is imperative that the construction stage is monitored and followed closely by the design team. Apart from the opportunity to calibrate the construction process, and to gain solid empirical experience, monitoring data can also be used to understand mechanisms that are not always clear in the design phase.

An example of this is the paper "3D numerical back-analysis of measured strains in a segmental tunnel lining during" from Fabozzi et al., where the deformations of a segmental lining ring (8+1), measured in the Metro Line 6 in Naples (Italy), are used to estimate the pressures acting during an EPB excavation. The lining elements are 1.7 m wide, 30 cm thick, 7.25 m in diameter (intrados), and are connected through flat longitudinal joints and bi-block circumferential joints. The section instrumented with vibrating wire gauges is in a sandy layer, 16.3 m underground and 10.3 m below the groundwater level. The 3D FEM, calibrated to match the measurements, considered a tapered cylindrical plate as the shield, a horizontal face pressure, jacking forces, grout pressure, and a hardened grout with increasing stiffness. The segmental lining was modelled in detail, with explicit joints with specific failure criteria. The calculated results (Figure 10) reflect the irregular distribution of forces in the lining, due to the point loads of the TBM jacks, which cannot be obtained from simplified analytical solutions. The authors stress the importance of considering the thrust from the jacks and the time-dependent grout behavior to achieve a realistic model.

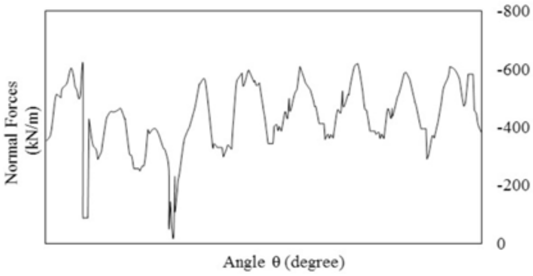

Figure 10. Numerical results of axial force (from Fabozzi et al.)

Measurements of lining deformation can also be used to trigger mitigation plans during construction, as discussed by **Arturo** in his paper "Diametric deformations in the concrete segment lining of a tunnel excavated in soft soils. Criteria for their evaluation and mitigation actions for their control". The author applied a simple methodology to identify the necessary mitigating measures around tunnels built in clayey deposits of the Valley of Mexico. Whether the tunnel is stable, should be closely monitored, if the ring's annular space should be re-injected or a metallic frame should be installed, are actions defined based on how the horizontal diameter of the lining increase with time, as plotted in a 2D graph. The author discussed two examples of stable and unstable scenarios, and their results within the methodology.

When two tunnels are built in close proximity and around the same period, it is always a question if and how much their interaction should dictate the overall design. The paper from **Chang et al.**: "Proximity Effect on Closely Spaced Shield Tunnels - Analysis, Design, and Feedback" attempts to answer this. Existing sources recommend a minimum clearance between 1 and 2D. For cases between 1 and 0.5D, a Japanese guide suggests that an equivalent surcharge in the segments should guarantee a safe design. However, for less than 0.5D, as it is the case in parts of the Taipei Rapid Transit System (TRTS), a more detailed analysis is required. Most tunnels of the TRTS Green line are built in a sandy-clayey-interbedded alluvium layer.

The authors present the TBM thrust and the tail gap as the two factors that have the highest potential to cause adverse effects on the preceding tunnel. Previous numerical analyses have shown, for example, that the vertical pressure increases with a smaller clearance, which directed a more extensive instrumentation campaign and the design of a few protection measures for the TRTS: Temporary bracing for 0.8D; Ductile segments and grouting treatments for 0.5D (Figure 11). The instrumentation records reveal that the maximum induced pressure followed the orientation of the TBM, with a maximum of 200 and 110 kPa increase of total stress and water pressure respectively. When the tunnels were directly above each other, there was a total stress decrease at the tunnel crown. All these stress variations did not disappear after the TBM has passed.

Figure 11. Protection measures for tunnels in close proximity - Temporary bracing (b) Supplementary grouting (c) Ductile segment (from Chang et al.)

The paper "Field Performance of Twin Bored Tunnelling in Different Geological Conditions - Construction of MRT Downtown Line 3 in Singapore" was also focused on Twin tunnels and was written by **Su et al.**. The study evaluates the settlements around the 42 km long Downtown line (DTL) in Singapore. The 6.6 m diameter twin tunnels vary from parallel to stacked layouts, through depths from 8 to 55 m. When tunneling through the Old Alluvium layer, a volume loss between 0.2 and 1.2% was recorded at the surface. The settlements started 3D before the section, and progressed until the TBM face had passed the section 3 to 5D. Through the Kallang formation, a much more recent deposit, the settlements were higher, up to 3% volume loss. When tunneling in mixed face conditions, the authors noticed that abrupt changes from hard to soft soil can induce high localized settlements. Overall, the line was completed without damaging the existing structures.

Even tough underground construction can be deemed a safe technology, accidents still occur. While undesirable, these events offer the possibility to learn what went wrong and to test emergency plans. "An incident of runway heaving due to shield tunneling for Taipei MRT construction", from **Fang et al.**, describes a heave/blow-out incident under the runway of the Taipei Songshan Airport. The 6 m diameter TBM was centered at a depth of 27.5 m in soft silty clay, with the groundwater level 2 m below the surface. In the design phase, the maximum face pressure was the at-rest lateral earth pressure at the tunnel crown plus 60 kPa, which set a 379 kPa limit. To reduce the settlements, the design team considered a higher lateral pressure coefficient, which set the limit at 486 kPa. Backfill grouting was designed at a volumetric rate of 150%, and at a pressure defined as the hydrostatic pressure plus 100 to 200 kPa, limited at 491 kPa.

At 2 a.m. of a regular day, a 1.5x3.8 m heaving area was observed in the main airport runway with a maximum heave of 65 mm. Material like the mud injected at the face was seen in the pavement. Fast responses were necessary to open the airport in a few hours. The contractor removed and rebuilt a 10x10 m, 12 cm thick asphalt layer. The high pressures of mud injection at the face and backfill grouting were probably the cause of the accident (Figure 12). After the incident, the face pressure was limited at 432 kPa, the grouting at 392 kPa, and all possible seeping paths from the TBM to the surface were investigated.

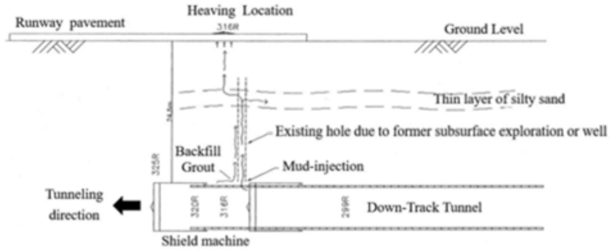

Figure 12. Possible cause of runway heaving (from Fang et al.)

The previous papers highlight the importance of controlling the construction steps and monitoring the surface settlements, which is the focus of the last two papers of this section. **Jefferis and Lam** make a compelling case, in their paper "Using density to determine the solids content of construction slurries", that the mud balance, the traditional instrument to assess fluid density in the field, doesn't have the necessary precision to control the production of slurries. Density is used as a surrogate measure for the solids content of the fluid, but typical mud balance measurements can only capture errors of bentonite content exceeding 40% (about 18 kg/m^3), which is unacceptable. The authors suggest the use of a fixed-volume container, such as a 1 l spring-top bottle, and an electronic balance, which can achieve a resolution of 2 kg/m^3 under isothermal conditions. For even higher resolution, the geotechnical moisture content test can be used, where a resolution of 1 kg/m^3 can be achieved without the need to assume the bentonite's grain density (such an assumption is necessary for the container method).

A new technique for surface settlement measurements is proposed by **Chian and Yang** in their paper "Use of Photogrammetry for Ground Settlement Measurement". The authors review the basic principles of photogrammetry, and present a surveying exercise in Singapore, where there is an extensive network of reference points. In a roadway the photogrammetry technique, at 95% overlap, reached a mean error of 1 mm and a root mean square error of 5 mm. The test around a building at 90% overlap, 10 m from the building, reached 1 and 3 mm for the mean and root mean square errors, respectively. On top of a grass patch, where the elevation is quite irregular, the mean square error jumped to 8 mm. Overall the authors conclude that this technique could be applied to monitor surface settlements.

2. 3 *Operation and Maintenance*

The construction of an underground structure is just the beginning of its lifecycle, which can last for hundreds of years. To achieve that, owners should have a clear inventory of their infrastructure, with as build and current state information.

The paper "Study of an old railway rock tunnel: site investigation, laboratory tests, weathering effects and computational analysis" from **Futai et al.**, presents how the current state of existing railway lines in Brazil, built in joint rock masses with no concrete liners, can be updated with state-of-the art methods. The paper presents the application of 3D terrestrial laser scanner (TLS) to map the discontinuities and model a discrete fracture network (DFN) of the exposed rock perimeter. The authors propose a methodology to analyze automatically the point cloud of the discontinuities, determining their probability of size and orientation, characterize their roughness, create the DFN and generate the block in the 3DEC software.

They applied the methodology to the 1 km long Monte Seco tunnel, excavated in a gneiss rock mass (Figure 13). Field tests were conducted to quantify the weathering grades of the rock, their porosity, visual description, and the rebound number from the Schmidt hammer test, used to estimate the JCS parameter. The estimated parameters for the joints were used in the Barton-Bandis failure criteria. For the 3DEC numerical model, these were replaced by equivalent Mohr-Coulomb failure envelopes, while the rock blocks were considered rigid. The calculated block displacements resemble the typical cross sections measured with the laser scanner.

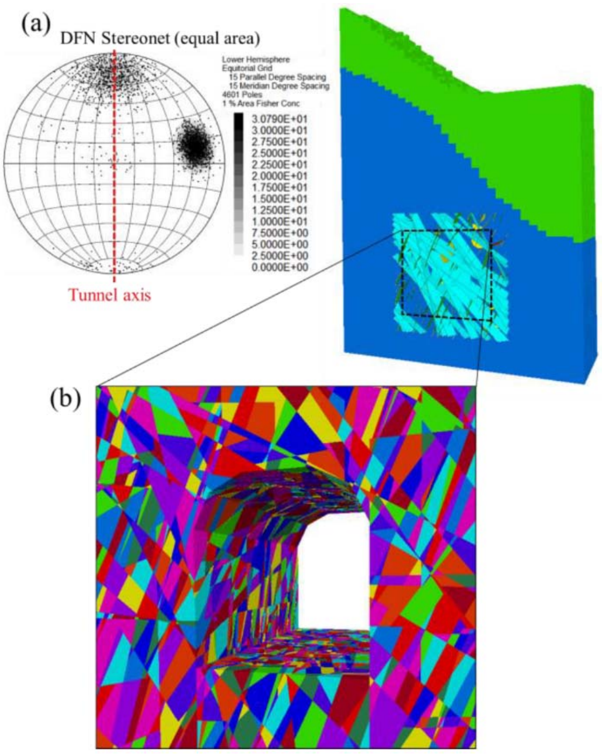

Figure 13. (a) Model of a 10-m-long section of the Monte Seco tunnel, indicating the DFNs generated. (b) Block model created after discontinuity intersections (from Futai et al.)

Perminov et al., in their paper "Geotechnical aspects of security for long-operated underground collectors in conditions of soft soils and increasing technogenic influences", discuss an assessment of 2300 km of sewage collectors in 15 Russian cities, with an average physical wear of 60%. The case of Saint Petersburg, with 270 km of tunnels, mostly in the historic center under difficult geotechnical conditions, is discussed in more detail. The inspections, from 1970 to 2016, included: radar scanning of internal surface, core sampling, concrete strength, chemical analyses, corrosion evaluation, and vibro-dynamic tests. With time, the identified defects evolved from shrinkage cracks and gas corrosion (15-20 years), to force cracks (20 years), and finally to heading leakages. Based on this experience, a series of analyses and preventive measures have been designed. The authors present the case of a tunnel with a wear degree of 79% in the Tovarischecky prospect. The maintenance works included: cleaning and preparing the surface; plastering the concrete and tubing lining; reinforcing the arch surface with carbon fiber; coating the tunnel surface with PVC coiling and injecting polymer-cement mortar. Vibro-dynamic tests reveal a significant structural improvement of the tunnel, which should guarantee its operation for the next 50 years.

3 DEEP EXCAVATIONS AND CAVERNS

This topic was discussed in the 10 remaining papers of this session.

3. 1 *Design and Modelling*

While the focus on settlement control extends from tunneling to open excavations, here the construction is more localized, which allows more complex structural solutions to be implemented. Among those are auxiliary buttress and cross walls, perpendicular to the diaphragm walls, to reduce the induced settlements, as discussed by **Ou et al.** in their paper "Use of rigid support system to reduce movements in deep excavations". A 56x80 m pit, excavated in seven stages until a maximum depth of 20 m, was modelled in the software Plaxis 3D considering the Hardening Soil constitutive model for the ground, and linear elastic model for the structural elements.

Numerous scenarios were simulated, including: Buttress walls, with several lengths, both demolished and maintained along the excavation (here named rigid support system), and considering frictionless and frictional interfaces with the ground; Cross walls, with several heights, maintained and with frictional interfaces; and a proposed U-shape system (Figure 14) where both elements are combined.

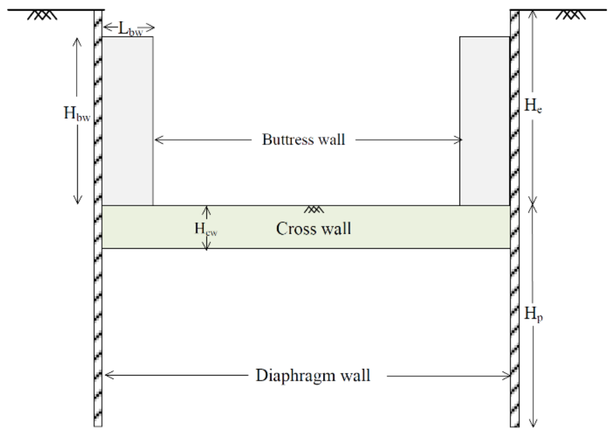

Figure 14. U-shape rigid support system (from Ou et al.)

Overall the authors concluded that when the buttress walls remain installed during the excavation, and 50% reduction of wall deflection and surface settlements can be achieved with a 4 m buttress, while an 8 m wall is required if it is demolished during the excavation. The stiffness of the system controls the wall deflections and ground settlements, but the latter also depends on the friction with the surrounding soil. The U-shape system achieved the best performance: a 50% reduction could be achieved with a buttress wall length and cross wall height of only 2 m.

The paper from Ilyichev et al., “A settlement calculation for neighbouring buildings with mitigation measures upon underground construction”, moves the discussion to a simple equation to predict the settlements induced in buildings with strip footings, and how to adjust this prediction when mitigation measures are used. A straightforward multiplication factor, calibrated with a database of 52 buildings considering different types of mitigation measures, is proposed for this adjustment. The data analysis makes it possible to rank the techniques based on their average settlement reduction coefficient. From higher to lower, the list was: compensation grouting, jacked piles, and a tie between cut-off walls and jet grout columns.

Still connected to the settlements induced on buildings, the paper “Excavation-induced ground settlements and responses of adjacent building at various positions using 3D decoupled analysis method”, from Lin et al., takes a different perspective on the problem, focusing on how the relative position of the building to the excavation can affect their interaction. Similarly to the study on pile-tunnel interaction, the authors propose that an analysis where the induced ground settlements are decoupled from the calculation of the structural reaction can produce significant results despite this simplification.

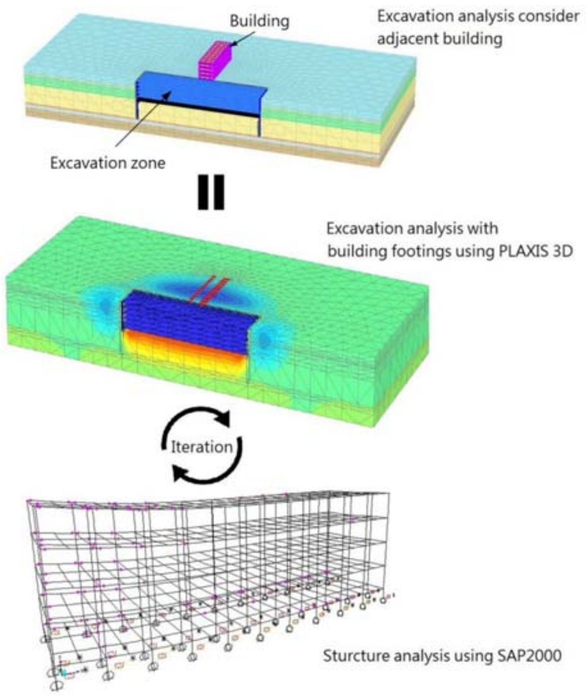

Figure 15. Decoupled analysis method (from Lin et al.)

A 21.5x105 m excavation is combined with a 14.4 m high framed building, supported by spread footings, set at 8 different positions relative to the tunnel, both parallel and perpendicular to its alignment. The excavation is simulated with Plaxis3D, while the structural reaction is calculated with SAP2000. The 2 models interact as the ground settlements are transferred to the foundation elements, which alters the foundations loads that are part of the input in the geotechnical calculation. This is calculated iteratively until the values stabilize. The authors conclude that the ground settlements are altered by the presence of the building, and increase with the building’s proximity to the excavation. Buildings in the sagging zone are more susceptible to damage, mostly due to the high angular distortion induced. On the other hand, buildings in the hogging zone, and in transition between the zones, are susceptible to tensile strains, which can crack tie beams.

Transitioning from general analysis to more specific modelling aspects, the paper from Nejjar et al., “Apport de la modélisation aux éléments finis des excavations profondes dans l’Argile Plastique dans le contexte particulier du projet du Grand Paris”, discuss different ways to achieve realistic calculation results for deep excavations. The example of the Fort d’Issy Vanves Clamart station, a 110x26m and 30m deep excavation supported by a 1.2 m thick, 40 m deep diaphragm wall, passing through overconsolidated soft clay under a thick rigid layer of limestone, was considered in a 2D FEM parametric analysis. The Hardening Soil (HS) model was compared with the traditional Mohr-Coulomb (MC) to correlate the deformability moduli of both models to achieve similar results. While good results were obtained for the deformations, the forces at the top floor levels were higher for the HS model, as it considers a stress dependent stiffness. The authors also discuss different modelling strategies for the earth pressure at rest, the overconsolidation ratio, and undrained loadings. These complex numerical calculations are then compared with the more straightforward subgrade reaction method. Even though a similar displacement profile could be calculated with this simple method, the authors highlight its limitation to consider soil arching and the compressive forces on the supporting elements.

Penzes et al. presented a similar comparison of different constitutive models in their paper “Numerical 3D Modelling of a Quay Wall System in Soft Ground Conditions”. A case study is analyzed with two walls, a combi front wall, and back secant pile wall, at a distance of 30.5m, topped with capping beams, and connected with transverse beams, which were supported by bored piles. In between these beams, plate anchors were attached to the walls with connections to limit the bending moment and deformations of the front wall (Figure 16). The soil profile comprises of a uniform clay layer until -55m covering a thick dense sand layer. Three constitutive models were compared: Mohr-Coulomb (MC), Modified Cam-Clay (MCC), and Soft-Soil Creep (SSC).

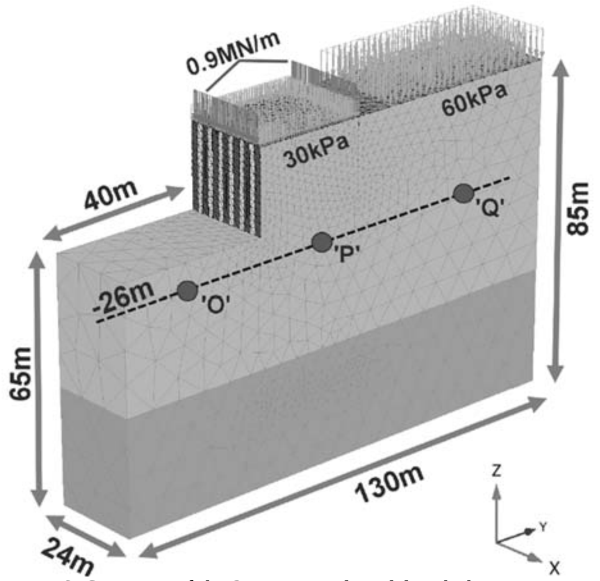

Figure 16. Geometry of the 3D numerical model with the main vertical loads and pre-defined measurement nodes (from Penzes et al.)

The calculated settlements under the walls were quite similar for all constitutive models and assuming an undrained response. On the other hand, during consolidation the settlements according to the MCC model were significantly higher than the other two. A closer look into the stress paths for each case reveals that in the MCC the soil reached the yield surface, causing plastic deformations, while the other two remained in the elastic stage, with no creep occurrence. Under the excavated area the picture was different, as the unloading effects with the SSC model are dictated by the creep rate, which resulted in the largest heave during the consolidation phase.

One of the only 2 papers dealing with seismic effects was written by Ding et al., and entitled "Research on seismic performance of large underground structures of urban rail transit". The study proposes a calculation method to check the seismic performance of large-scale underground structures in China based on a 3D dynamic FEM. As an example, the authors considered a massive 144,000 m^2 three-floors double station, and commercial center, build through a profile of fill, silty clay, and weathered mudstone. Three sets (along different directions) of two artificial seismic waves (design and high level) were calculated. Both the maximum horizontal displacement, and relative story displacement, depended more on the wave intensity than on its direction. The internal structural forces tend to increase under the action of these example earthquakes, which can be particularly harmful to the openings in the structural elements.

3. 2 *Case Studies*

The paper "A case study of deep excavation near a historic building in Toronto" from Cao et al. details several steps of a challenging project in Canada, where an 11 m deep excavation was built at a 2-3m distance to a 140-year-old building, to separate the grades of two underground rail lines. An interlocking steel pipe pile wall was installed based on an installation test where a 90 cm diameter pile was deemed suitable. The building was reinforced with stabilization grouting (foam injection) under the existing foundation, jet grouting at the interlocks to seal the joints, and external struts connected to the retainment wall.

The first measure was tested varying the drilling angle and the distance from the building. SPT tests were performed before and after grouting, showing an increase of 3 to 5 SPT-N values. During grouting the wall showed a small heave (4 mm) and lateral movement. The jet grouting operation was also tested with a trial operation that was inspected visually. Real-time monitoring indicated that the wall moved less than 3 mm during grouting, while instruments in the ground revealed a maximum heave of 6 mm and settlement of 9 mm. The authors also made use of FEM calculations to back-analyze the excavation, and were able to obtain a very good agreement of lateral movements with the inclinometer readings.

The authors Jeng and Chang, in their paper "Case study on numerical analysis of retaining wall deflection and ground settlement of Top-down and Bottom-up construction methods.", compare and contrast six cases of top-down and bottom-up excavations, and compare them with a popular empirical nomogram to determine wall deflection and ground settlements. An overall analysis reveals that the ratio of the maximum wall deformation to the excavation depth is 0.2 to 0.5%, and 0.16 to 0.5% for top-down and bottom-up cases, respectively. The examples indicate that reinforcing structures (buttress and cross walls) have little impact in the top-down cases. In terms of ground settlements, their ratio to the maximum horizontal wall deformation in top-down construction is 0.19 to 0.78, while in bottom-up construction it between 0.15 and 0.49.

The paper "Deep Basement Excavation - Ground Movement and Groundwater Response Observed in Different Geologies in Singapore", from Win and Angeles, report on two case studies in Singapore: Tanjong Pagar and Southbeach developments.

The first one is a 290 m tall building over an 18 m deep car park in a residual soil of weathered siltstone or sandstone, with SPT-N values above 100 below 8 m, and a groundwater table at a depth between 1 and 2 m. The ground permeability was measured at 10^{-7} m/s. A top-down excavation was adopted combining contiguous and secant bored piles. Monitoring results of groundwater levels indicate immediate drops, as high as 11 m close to the toe of the wall, as the excavation progressed. These caused a maximum surface settlement of 20 mm, above the normal 15 mm limit in Singapore. The second development is a two tower (45 and 34 stories) complex occupying a whole street block in an area of reclaimed ground close to a railway station, where there are strict displacement restrictions. The very dense silty sand/hard clay old alluvium is set under a recent marine/fluvial deposit of sands and clays. The 18 m deep excavations were supported by two circular diaphragm walls, reaching 90 m in diameter. Monitoring data shows good performance of this system, with wall deformation of only 0.4% the excavation depth. The piezometric drops in the marine clay increased with depth, reaching a maximum of 9 m, causing settlements of up to 90 mm, that continue to increase after the excavation was completed.

Also in Singapore, the study from Dong and Whittle: "Importance of considering variable ground stratigraphy in underground construction", focus on modelling how the boundaries between different ground layers vary along the horizontal direction, and that the practice of simplifying that to horizontal boundaries can have serious implications for the model's accuracy. To interpolate between boring lines around the Singapore Post Center project, the authors use ordinary Kriging methods. A single diaphragm wall, 6m wide, and 55.5m deep, is modelled in 3D with the ABAQUS software, considering a time-dependent model for the concrete lateral pressures and the MIT-E3 constitutive models for a Marine clay layer. For both the surface settlements, and the horizontal settlements with depth, the variable stratigraphy models yielded results that were closer to the field measurements than the flat stratigraphy models.

4 CONCLUSION

This report presented an overview of the papers submitted for the TC204 session of the 19th International Conference on Soil Mechanics and Geotechnical Engineering. The papers covered several aspects of "Underground Construction in Soft Ground" and also some topics on rock tunnelling that were assigned to this session. The 32 papers were divided between tunnels and deep excavations, and classified as studies on design, construction or operation of these structures.

The comments on this report are an image of the information displayed on the papers, the validity of the results and conclusions are responsibility of the authors of the papers. We thank the 19 nations that contributed to this session, and would like to invite all member societies to take on active roles in the activities of the committee.

All scientific publications should be reproducible, which requires all the conditions guiding the results and data analysis to be depicted. Unfortunately, several papers in this session didn't mention important details to enable a reader to fully assess the contents of the study. We believe that the the quality and explicitness of the results and the description how they were obtained is what really makes the content of a paper to arouse interest and encourage the application of that knowledge.

We expect that this TC 204 session has brought significant contributions to the underground space research and practice fields, so that the important advantages of the use of the underground space for our cities can be fully exploited and is not limited by our technical capabilities.