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Bored tunnelling – NATM tunnelling

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ABSTRACT: Twenty-three papers were submitted for this session and tunnel excavation stability was the topic most dealt with, several case histories being presented to illustrate construction details and sequences adopted either to ensure stability or as collapse remedial measures. Groundwater control was a key topic in many papers, use of compressed air being one of the techniques employed. Several ground improvement schemes using jet grouting, impregnation grouting, and one compensation grouting trial are described. Use of ground anchors in weak rocks was reported as permitting much lighter sprayed concrete linings, and inducing smaller settlements than for a support based exclusively on heavier linings. Some cut&cover tunnels are also reported, including a case history involving a building collapse.

1 INTRODUCTION

In order to avoid a lengthy review, an attempt was made to classify the submitted papers according to main topics focused by Authors. This division is shown below, and it should be pointed out that it is rather an arbitrary one, since several papers could clearly fit into various of the below listed topics:

- Tunnel excavation in waterbearing soils,
- Tunnel excavation in weak rocks
- Tunnel-building interaction behaviour
- Laboratory and field tests and advanced monitoring system
- Cut & Cover tunnel excavation
- Environmental aspects

It should also be mentioned that to avoid conflicts about the appropriateness of the term NATM, "Sprayed Concrete Lining" will be used wherever a single or multi staged excavation relies on sprayed concrete, wire mesh, steel arches and ground anchors as a primary ground support.

This is similar to the approach adopted by the Institution of Civil Engineers (UK) (Sprayed Concrete Linings (NATM) for Tunnels in Soft Ground, 1996), refer to Chapter 2 "What is NATM?", "...Any use of sprayed concrete support for tunnel is often erroneously referred to as NATM..."

2 TUNNEL EXCAVATION IN WATERBEARING SOILS

2.1 *Pedestrian and Utilities Tunnel in Budapest*

Müller, Harsányi & Czap report on the construction of a 3,70m diameter sprayed concrete lined tunnel built in sandy gravel. This tunnel has three sections, named "A", "B" and "C", which interconnect the basement of three buildings, two existing and one building under construction.

Soil in the zone of tunnelling consists of a sandy gravel just above the groundwater level; a 13kPa cohesion was determined through back analysis of an unsupported vertical face. This cohesive strength is probably due to capillary effects and eventually due to some slight cementation (no mention was made to the time span the unsupported wall could remain stable).

Sections "A" and "B" run beneath the existing buildings, and the sandy gravel was grouted from surface or basement level with a cement bentonite slurry injected through tubes "à manchette".

Section "C" is directly beneath a road, with about 5,50m soil cover between tunnel crown and road level. This road is congested with services, including gas and water pressure ducts, a sewage duct and electric and telecom cables.

No soil grouting was carried for this Section; the excavation proceeded in a top heading and invert sequence, in 40cm steps at heading and 80cm steps at invert. The stability of the top heading excavation was relied solely upon the sandy gravel cohesive strength. The invert front wall excavation was protected by a sheet pile system, installed at each 80cm step.

Some analytical and numerical stability and deformation analysis (no details given) were previously carried out to ensure the adequacy of this construction sequence.

The main point to be emphasised is that the tunnel Section "C" was successfully built with settlements of the order of 5mm or less, in a low cohesion sandy gravel, without resource to soil improvement or forepoling ahead of tunnel face.

2.2 *Railway tunnel in Frankfurt*

The Frankfurter Kreuz rail tunnel construction is reported by Katzenbach, Arslan, Festag & Rückert. This is a 150m^2 cross section sprayed concrete tunnel, excavated entirely in a pleistocene gravel layer, crossing under the Autobahn A3, some 15m above crown.

Construction sequence consists of top heading, bench and invert. Stability of top heading excavation was ensured by 14,5m long longitudinal jet grouted columns installed along the upper arch, and complemented by 8 columns installed through the excavation face to act as a face support.

Inclined transversal jet grout columns, 4m long, were used as underpinning piles at the arch footings; also a temporary invert was sprayed to increase top heading arch bearing capacity.

Groundwater level is near top heading floor level; in order to avoid groundwater lowering for bench and invert excavation, a ring of jet grouted columns were installed around the bottom arch perimeter. To complete this sealing barrier, bulkheads of jet grouted columns were installed through the bench area in cross sections spaced 30m to 60m.

A block failure mechanism was adopted to investigate the stability conditions at the tunnel face. It would be interesting to know how this mechanism compares with known lower bound plasticity solutions. Soil parameters for these analysis were derived from in situ large scale shear tests, performed on top heading floor samples. The best fitting shear envelope was that of a non linear power law. These results compared well with laboratory triaxial tests on undisturbed samples.

Maximum recorded settlements at the Autobahn A3 were 42mm, lower than the 50mm design allowance.

The Authors conclude their paper by describing future challenging tunnel works belonging to the Frankfurt 21 Project, in heavily built areas. Some extensive groundwater lowering appears to be necessary, similarly to past experiences quoted by the Authors; this procedure seems to be acceptable in Frankfurt, contrary to many other cities where groundwater is not to be lowered, mainly because of fear of consolidation settlements.

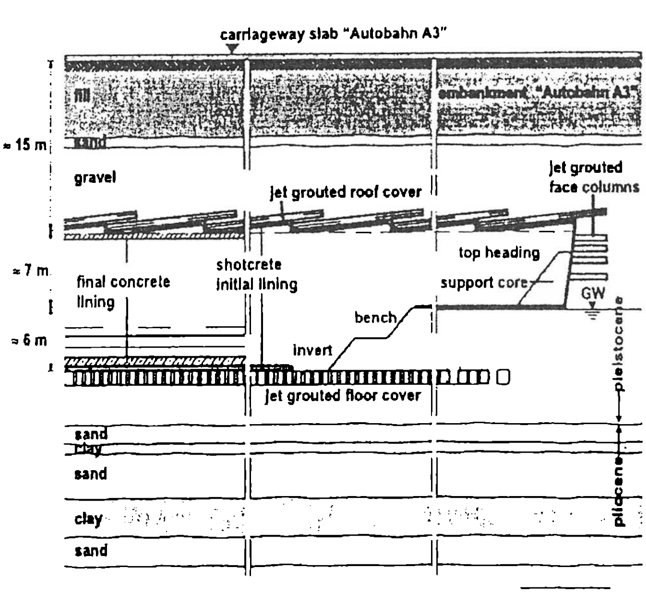

Figure 1: The Frankfurter Kreuz Tunnel

2.3 *Railway tunnel Cologne – Frankfurt*

Quick, Michael & Arslan describe two case histories, the first one being a 11,5m shield tunnel (Wandersmann Tunnel) driven through intensely fissured Tertiary clays, below groundwater level.

A large mass of failed slope was detected at one of the portals, which will require a series reinforcing piles extending below slip surface.

The shield machine is provided with steel support plates at the face, covering up to 75% of the exposed face area. Since this is not a slurry type or earth pressure type shield, a groundwater lowering system was employed. A face instability occurred just after shield departure at one of the portals, and this was remedied by freezing a 20m long section with vertical freeze pipes.

A comprehensive monitoring system was implemented, including inclinometers, multiple extensometers, surface settlement marks, tunnel convergence and radial pressure cells. Inclinometers were also used to monitor the failed slope mass.

The instrumentation was useful in determining the main cause for ground loss during shield driving: the unsupported length at the crown of the tunnel, which extended from the shield face (due to overcutting) up to the erected ring behind shield tail, where void grouting was applied. By carrying out a preliminary grouting through the shield skin (details of which are not given), it was possible to reduce 6cm in surface settlements from a total of 14cm previously observed.

The Schuwald Tunnel, with a cross section area of 156m^2 is a multi stage excavation lined with sprayed concrete in intensely weathered phylites below groundwater level.

It starts with a pilot tunnel, some 30m^2 in area, widening to the top heading, and followed by bench and invert. In the worst soil conditions the pilot tunnel was divided in head and bench and a temporary invert was applied to the top heading.

Driven steel bars are used as forepoling to avoid local instabilities. Grouted anchor bars up to 12m long are installed in a radial array. Some of these bars were installed from the pilot tunnel and were later reused during excavation of the top heading.

Horizontal drainage pipes are driven ahead of tunnel face to lower the groundwater (no mention if vacuum was applied).

The surface, roof and arch footing settlements are presented along the length of the tunnel. Maximum settlement at surface was 21cm, the average being some 13cm. The roof settlement figures shown in this paper are not greater than surface settlements 20m above crown, and probably mean lining settlement only, not total soil settlement at crown level.

2.4 *Compressed air in sprayed concrete lined tunnels*

Sprayed concrete lined tunnels excavated with aid of compressed air have been used in 19 contract sections so far, 8 of these in Munich, according to Kammerer & Semprich.

An illustration of a typical application of this method in Munich shows a tertiary sand, permeability to water 10^{-5} m/sec, extending about one metre below to 1 metre above crown, overlain by a gravel layer with permeability to water greater than 10^{-3} m/sec about 13m thick. Groundwater level is about 4m above crown. The applied air pressure was 0,6bar.

The authors report the experience in Munich which indicates that surface settlements may be typically reduced from 13mm, if compressed air is not used (supposedly a groundwater lowering is applied to such case), to 9mm, when a 0,7bar air pressure is used. Similar results were reproduced in Finite Element elastic analysis, simulating an upward flownet, and applying corresponding percolation volumetric forces $f_s = \Delta p/\Delta s$, where Δp is the air pressure loss through a path length Δs. The lining stiffness was disregarded, i.e. no support pressure would result from lining soil interaction.

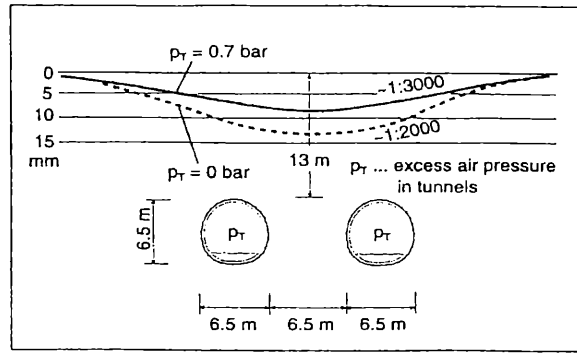

Figure 2: Settlement reduction due to compressed air in Munich

Through such numerical analysis it was possible to establish the influence of the relative permeability between lining and soil. If the lining is impervious related to the ground the air pressure acts primarily as a support pressure holding the lining and this is the most effective in reducing settlements (quoted value of 40%). If the lining is relatively permeable related to the ground (e.g. the lining is fissured and soil is a silt), the air pressure is dissipated in the ground, and settlement reduction efficiency drops (quoted value 20%).

A series of laboratory tests are being performed to simulate such process and also investigate air loss through the ground, since the Munich experience has shown that large flows (values of 600m^3/minute) can result depending on the tunnel length, which ads a considerable cost to the compressor systems.

This very interesting combination method deserves to be further investigated and it is in the Authors plan to extend their analysis to a more sophisticated numerical model, including constitutive relations other than elastic, and two phase flow system. The beneficial effects of air pressure derive from an outward water flow, away from the tunnel, since percolation forces are proportional to γ_w . i, i being the head loss. If the soil looses its saturation and air flows freely to the surface that benefit is much lost (there is also a risk of a blow out pressure); therefore the Authors intention to analyse a two phase flow system is more than justified.

2.5 *Tunnelling through a paleo valley in the French Alps*

Guilloux describes the extremely difficult tunnelling conditions during crossing of a paleo valley filled with cohesionless alluvium and 60m water head.

The double lane twin tunnels belong to the A43 expressway, and were being excavated exclusively in shales and gneiss by drill and blast method. Seven hours after a blast operation on the western tunnel, a violet rush of soil and water occurred through a small opening near tunnel face, following by another two days later, totalling about 2000m^3 of soil debris.

It was found that a deep and narrow paleo valley, formed by glacial erosion, intercepted the tunnels, and this was totally unforeseen in earlier geological investigations (no mention if horizontal probings were routinely done through tunnel face).

A cement bentonite impregnation grouting scheme was applied with grouting tubes being drilled from the neighbour east tunnel, which was entirely in rock and stable. This grouting scheme proved to be unsuccessful since a new failure of 3000m^3 occurred at the restart of excavation in the west tunnel. It was later found that grouting was not homogeneous as some zones of alluvium remained untreated; moreover, some heavy rainfalls occurred the week before and increased 15m in water head.

A second scheme of treatment was implemented, this time involving not only cement grouting but silica gel grouting as well, installing drainage pipes through the alluvium and installing steel pipes 89mm in diameter to form an umbrella over the excavation area. Excavation of the west tunnel proceeded successfully under this scheme.

The detailed geological investigation carried out after the accidents revealed that also the east tunnel would cross the paleo valley at an acute angle. For this crossing it was decided to adopt a structural solution based on the following:

- steel tube reinforced jet grout columns around the tunnel perimeter,
- fibreglass reinforced jet grout columns at the tunnel face,
- inclined micropiles under the top heading arch footings,
- sub vertical jet grout columns around invert level,
- drainage pipes ahead of excavation.

This solution proved to be well suited to these conditions, and measured displacements did not exceed few millimetres.

Since this tunnel intercepted the paleo valley at an acute angle, for a certain length the tunnel cross section was partially in soft alluvium, partially in hard rock. Detailed numerical analysis by finite element numerical models were used to evaluate the effects of such asymmetry in the final lining design, the main conclusion being the necessity of relieving water pressure in the lining through permanent radial drains.

2.6 *Groundwater lowering analyses in Belgrade*

Rasula describes use of a 3-D finite element model in hydrodynamic calculations to assess feasibility of drainage of an inclined sand layer some 2 to 5m thick of K= 1 to 5 x 10^{-6}m/sec, with artesian conditions, by vertical wells, to enable excavation of a future metro tunnel.

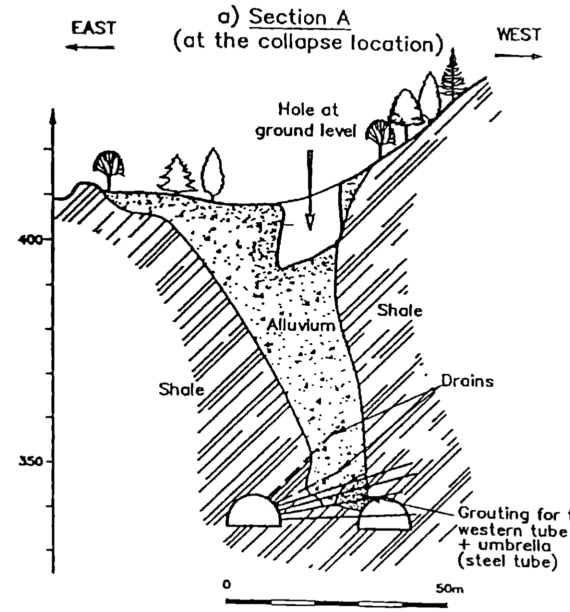

Figure 3: The A43 Motorway in the French Alps

A series of parametric analyses were carried out, including variations in the permeability coefficient, well diameter, boundary recharge conditions, and a series of different well arrangements and working regimes (use of vacuum in the wells is not mentioned).

The Author main conclusion was that a residual groundwater column of the order of 1,5-2,5m would remain undrained, therefore such vertical well arrangement would have to be complemented by another scheme (possibly horizontal drainage pipes, vacuum assisted, drilled ahead of tunnel face).

3. TUNNEL EXCAVATION IN WEAK ROCKS

3.1 *Radial ground anchors to control deformations.*

Launay reports on three case histories of underground excavations where ground anchoring is an integral part of the support system, significantly reducing the concrete lining thickness.

At the Sèvres-Achères sewage system, a cavern of elliptical shape, 12,4m wide, 16,5m high and 60m long, was built in weathered sandstone, weak marl, chalk and swelling clay, by top heading and successive benches.

The original design called for a concrete lining 1m thick at the crown and 1,6m thick at the sidewalls. This was changed by a combined system of 25mm grouted anchor bars, 6m long, at 1m to 2m centres, completed by a 40cm thick sprayed concrete lining.

An extensive monitoring system was implemented, including radial pressure cells in the sprayed concrete lining. The Author's interpretation of measurements indicate that the ground anchors are the main elements of the cavern support, while the sprayed concrete lining are practically not loaded.

Similar design approaches were adopted in the Magenta-Eole Metro Station in Paris, and also in the Ambelokipi Station in Athens. For the latter a comparison is made with a similar Station in the same Geology and overburden, but where ground anchors were not used.

In Ambelokopi, 25mm grouted anchor bars, 4-5m long, were used at 1-1,5m centres, combined to a 150mm thick sprayed concrete lining. An interpreted 50% reduction in settlements was observed as compared to the sister station, where a 250-350mm thick sprayed concrete lining was the sole support system. The Author quotes some research work on deformability modulus of early loaded sprayed concrete, where it could be as low as one fifth of a conventional concrete modulus, therefore with a significant efficiency loss as a support in weak rock.

3.2 *Use of rock mass classification system coupled with numerical models in Greece*

The design of the Mpozaitika road twin tunnels in weak sedimentary rocks in Greece was carried out in two distinct stages, as reported by Sofianos, Kalkantzi and Giannaros.

Their design approach was basically:

- define the primary support based on the Barton's Q index rock classification system,
- monitor the behaviour of excavation in terms of displacements, strains in rock and anchor pull out measurements in ground anchors,
- carry on numerical analyses to adjust design parameters with feedback from the monitoring system,
- define the final support system based on numerical models and back calculated ground parameters.

The primary support consisted of 4m long grouted ground anchors on a 1,3m x 1,3m pattern at maximum, combined with lattice arch girders and sprayed concrete lining. The final lining consisted of an additional sprayed concrete lining.

The Authors stress that the anchors reinforcement system was fundamental to ensure successful tunnelling operations and enabled optimisation of the final lining design The shear bond at grout rock interface was found to lie in the range 0,6-1,1MPa.

3.3 *Use of rock mass classification system and deformation observations in Thailand*

Gurung, Iwao, Ishibashi, Kusakabe & Hongo describe design aspects and construction monitoring of the tunnels and caverns for Lam Ta Khong hydro power project. The area consists of Jurassic sedimentary rocks with variable degrees of weathering, and soft quaternary deposits.

The Bieniawski's Rock Mass Ratio classification system was used to define standard support sections to be used in all cases.

The authors found that such method is an useful design tool but need to be complemented by deformation measurements, and propose use of elastic analytical models such as the Kirsch equations (homogeneous isotropic linear elastic close form solution for a hole in infinite medium) to evaluate stress conditions around the openings, from the measured excavation displacements.

Two collapses in soft soil conditions are described and related to excessive water pressure; remedial measures included soil drainage and strengthening of support, in form of steel ribs, ground anchors and sprayed concrete lining with wire mesh.

3.4 *Squeezing rock conditions in Greece*

Tsatsanifos, Mantziaras & Georgiou describe the exceptional squeezing behaviour of a slickensided argillaceous schist, during excavations for the Tymfristos road tunnel. The tunnel section cross section area is $120m^2$, and was divided into top heading, bench and invert.

Tunnelling works suffered two major interruptions, one of those occurring after breakthrough of the top heading. At that time there was no limitation concerning ring closure maximum distance behind the face, and the top heading lining suffered significant overstressing and was badly damaged, affecting the required tunnel clearance.

The attempts to re-excavate the top heading resulted again in serious difficulties even with an invert closure as close as possible to the re-excavated face.

The monitoring system was based on convergence and levelling measurements; the interpretation of such data was used to investigate the strains developed in rock during excavation, by calculating the ratio of the inward radial displacement to the tunnel radius u/r (this ratio was named radial strain, but in fact it should be understood as tangential strain at the excavation boundary, $\varepsilon_\theta = u/r$ since $\varepsilon_r = \delta u/\delta r \neq u/r$).

After a careful and detailed analysis of the interpreted data, the Authors have reached to the following main conclusions:

- the threshold strain level beyond which tunnel become unstable is close to 2%,
- at the above strain value and beyond, ring closure had no positive effect in stabilising the tunnel, regardless of its distance from re-excavated face, since the lining had already been badly damaged.
- ring closure should ideally be achieved at strain levels not greater than 1%.

3.5 *Displacement extrapolation models in Korea*

An analytical model to estimate the total settlements occurring in a weathered gneiss rock mass is proposed by Chung, Kim & Hwang. They have performed 3-D finite element analysis to fit a series of displacement data obtained from four tunnels in Korea. The best fitting function is of an exponential form, although a linear form would also be suitable.

By measuring crown settlements occurring from tunnel face to a section one tunnel diameter behind face, it would thus be possible to anticipate the total rock settlements occurring at crown level.

The Authors found that for the cases analysed the final total crown settlement is 1,5 times the settlement measured from tunnel face to one tunnel diameter behind face. It is implicit from such analyses that tunnel is stable, i.e. excessive plastic deformations leading to failure might not be anticipated by such a model.

4. TUNNEL-BUILDING INTERACTION BEHAVIOUR

4.1 *Inclined tunnel under basement structure in London*

Bloodworth and Macklin relate the construction of an electrical cable tunnel, 2,44m in diameter, at a 1:6 gradient, under a basement structure in London clay. The basement is part of a demolished building and is currently a car park, consisting of a ground floor slab, a basement slab, and concrete columns founded on individual shallow footings.

The tunnel was hand excavated and lined with bolted concrete segments 0,75m wide, annulus grouting following two rings behind face, at most.

The fact that the tunnel was inclined to horizontal provided the chance to observe ground settlement behaviour at different depths in just one location. The monitoring system consisted of surface and deep settlement marks, and levelling pins at the columns and basement slab.

The main observations from the interpretation of the settlement data are:

- ground loss volume is in close agreement with prediction models developed for London clay
- the shape of the settlement trough is much wider than would have been predicted for greenfield conditions,
- as a consequence of the above, maximum settlement of the basement is about 50% of what would be predicted for greenfield conditions,
- the basement behaviour is similar to a beam with the bending section consisting of basement and ground slabs, and strong axial stiffness (which also restrain horizontal movements).

By interpreting these data and by comparison with published soil structure interaction models, the Authors were able to show that the basement structure modified the development of the settlement trough, not only at basement level but also below, at subsurface level. This is a fundamental indication that when predicting building settlement behaviour due to tunnelling, it is imperative to account for its rigidity.

4.2 *Pile-tunnel interaction problem in London*

Higgins, Chudleigh, St John & Potts describe a challenging engineering problem which is becoming more frequent in dense environments such as London, which is re-development of sites involving installation of foundation piles close to operating tunnels.

Available case history records of such situation are scarce, thus calling for a research into this problem, aiming at a development of rational guidelines.

The Authors briefly comment on the current available criteria, and analyse the most common prescription which is use of slip liners in piles, above tunnel level. Through numerical studies they are able to conclude that slip liners may not be necessarily advantageous and under certain conditions could cause more harm than an unlined pile. It is pointed out that longitudinal bending of the tunnel, acting as a tube, can be more damaging than plane bending of the lining.

A method of analysis based on plane strain and axi-symmetric numerical model has a good reliability potential, and avoids complex, time consuming and difficult to interpretate 3-D analysis. The proposed calculation steps are basically:

- plane strain simulation of tunnel construction,
- axi-symmetric analysis of pile installation using the stress distribution from the preceding stage. Pile installation is simulated by shaft excavation, and subsequent concrete casting (loading fresh concrete),
- axi-symmetric analysis of pile loading, first undrained, and subsequent consolidation,
- return to plane strain tunnel problem, and impose stress distributions resulting from the axi-symmetric analysis (including pile excavation, loading undrained, and consolidation).

It is hoped that further interesting contributions of this subject be published in near future.

5. LABORATORY AND FIELD TESTS AND ADVANCED MONITORING SYSTEM

5.1 *Improvement of tunnel face stability with reinforcing rods*

Hallak, Garnier & Leca carried out centrifuge test experiments to determine the behaviour of PVC rods as tunnel face reinforcement. The tested soil was a Fontainebleau sand, and face support pressure was simulated by air pressure applied to a rubber membrane. This support pressure was gradually decreased until face instability was observed.

Four test sequences were carried out: a) no rods installed, in this case good agreement was found with limit analysis solutions; b) 1 rod/2,8m^2 at the prototype scale, and rod length 1,5 times tunnel diameter; c) 1 rod/1,6m^2 at the prototype scale and rod length 1,5 times tunnel diameter; and d) 1 rod/2,8m^2 at the prototype scale and rod length 0,65 times tunnel diameter.

The Authors main conclusions are:
- the ultimate face support pressure could be reduced by 30%, for the lower density rod arrangement, or by 50% for the higher density rod arrangement.
- the higher density arrangement did not influence face displacement prior to failure, as compared to the lower density arrangement.
- use of shorter rod length did not cause any change regarding ultimate support pressure and support pressure Vs face displacement curve.

Centrifuge testing is a powerful investigation tool, and Authors should be encouraged to extend their testing program, to include different rod axial stiffness, e.g. carbon fibres, which are stiffer than PVC, as well as a range of different soil deformabilities.

5.2 *Compensation grouting trials in Singapore*

Shirlaw, Dazhi, Ganeshan & Hoe report on compensation grouting trials in soft Singapore marine clay (undrained shear strength 18kPa), in order to assess its feasibility for use in Mass Rapid Transit construction sections.

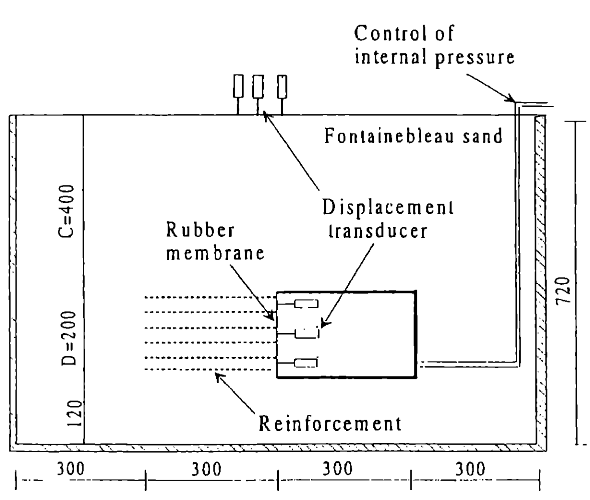

Figure 4: Centrifuge tests in France

Four test arrangements were selected, with different valved grouting pipe ("tubes à manchette") arrays, grouting sequence and volumes injected.

Instrumentation consisted of surface settlement marks, subsurface settlement magnets, inclinometers, and vibrating wire piezometers.

Heaving of the order of 15-25mm was easily achieved and smoothly distributed according to an inverted error function shape.

Large pore pressures (up to 150kPa) developed in the grouting process. The subsequent consolidation stage took some 100 days to reach equilibrium.

After consolidation it was observed that the short term heave was lost, and in some cases even a settlement occurred.

The Author's main conclusion is that compensation grouting is not an efficient settlement reduction measure in Singapore soft clays.

5.3 *Freezing-thawing effects on a clay strength*

Nishimura & Kondou performed a series of triaxial tests in normal consolidated and over consolidated clay subjected to closed system freezing and thawing. Consolidation ratios tested were 1 (NC), 2 and 3, and each sample suffered three cycles of freezing-thawing.

The main conclusions are:
- unconfined shear strength of thawed soils decrease due to freezing and thawing
- effective stress envelopes remain unchanged upon freezing-thawing.

5.4 *Monitoring ground deformation around a tunnel heading in London*

Van der Berg & Clayton describe the use of chain deflectometers and inclinometers to measure displacements ahead of an approaching tunnel heading. The object is the Heathrow Express Concourse Tunnel, in London clay, some 20m below surface level.

Both inclinometer and deflectomer casings were grouted against the ground. Considering the geometries involved and displacement magnitudes measured it seems reasonable to neglect the casing and grout bending stiffness.

The Inclinometers were installed from surface level, but did not extend below Concourse tunnel invert a sufficient length to warrant a fixed point. Absolute displacements were then interpreted by surveying the inclinometer top at surface level.

Deflectometers of the Interfels type were installed in horizontal boreholes drilled from neighbour platform tunnels. Each measuring section comprised three deflectometer levels. Two measuring sections were employed. The deflectometer extremes were

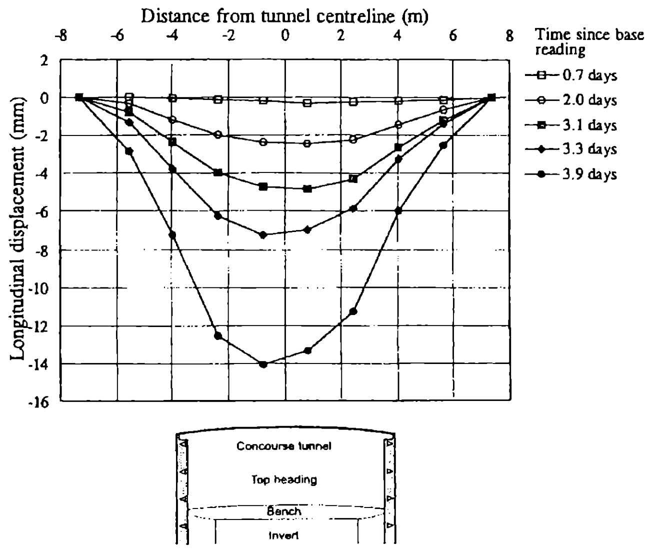

Figure 5: Horizontal deflectometer readings ahead of tunnel face, Heathrow, London

surveyed by optical targets, in order to determine absolute displacements.

Maximum longitudinal displacements were of the order of 16 to 19mm, and with good agreement between deflectometers and inclinometers. This instrumentation scheme proved to be reliable and its results are a valuable data source for future 3-D modelling.

6. CUT & COVER TUNNELS

6.1 *Accurate modelling of a London underpass*

Higgins, Fernie, Edmonds & Potts describe the design studies carried out for an unsymmetrical (i.e. one wall lower than another and inclined slabs) underpass in London clay.

Soil model is non linear elastic combined with Mohr Coulomb yield criteria. Comparisons with high quality swelling tests indicate the suitability of this model to reproduce long term behaviour in London clay.

A finite element numerical model was used to simulate construction sequence, and simulations included "top down" and "bottom up" sequences, connections between top slab and walls, pinned or full moment transfer. Also a series of different prop stiffness (bottom up sequence) were tested.

The resulting structural loadings and displacements corroborate the importance of properly accounting for the above details. Bending moments, for instance, varied as much as 50% from one hypothesis to another. Soil displacement was also sensitive to the case considered.

It should be noted that variations between the above cases would remain undetected with conventional methods and software packages that assume symmetry.

This paper shows the importance of considering the correct structural behaviour and construction sequence to perform realistic analysis.

6.2 *Building collapse in Calcutta*

The collapse of a building adjacent to a Calcutta Metro cut & cover trench is reported by Som.

Predominant soil is a silty clay / clayey silt with an undrained shear strength of 20kPa, and high water table. The cut & cover consists of diaphragm walls built in 3m wide panels, and steel struts.

Close inspection of the diaphragm wall system revealed visible gaps at the panel joints. Water and eroded soils were flowing through these joints. An earlier sign of inadequate behaviour occurred at the opposite side of the above mentioned building, when a portion of the surface adjacent to the wall subsided.

The building accident was triggered by bursting of a water main, which caused a sudden rush of soil through the panel joints filling the bottom of the cut & cover with about 400 m^3 of eroded soils.

The 5 storey building was founded on shallow footings, and gradually tilted towards the cut & cover until collapse; due to rapid action by authorities the building had been evacuated, and no victims were claimed.

The cut & cover structured remained stable without signs of distress, which was an indication of its robustness, and that failure was caused by an improper panel joint detail.

Remedy measures were mainly based on cement grouting of the soil in the collapsed zone, and provision of valves in the water main to enable a rapid shut down of flow, in case of an emergency.

6.3 *Bottom heave in Tokyo*

Kubota, Sako, Morota & Kojima report on the excavation works for a 21m deep by 31m wide cut & cover trench in soft fills and soft alluvium, with high water table.

Preliminary analysis indicated that safety margin to bottom failure mechanism was insufficient, so a 5m thick soil strengthening was implemented by soil grouted columns with the deep mixing soil stabilisation method (characteristics of this method are not described).

The cut & cover walls consist of tangent steel pile pipes braced by 7 levels of H type steel profiles. Deep settlement marks were installed to monitor heave displacements.

As the excavation proceeded to the 6^{th} and 7^{th} level a significant bottom heave developed, of the order of

100-150mm. Excavation was immediately halted, but movements only stabilised after the trench was filled up with water, and a 4m thick soil layer was removed at the outside of trench.

Investigations carried out after this event showed that shear strength of untreated soil at the bottom of excavation was lower than previously determined, probably due to significant swelling. Back analysis of a heave mode of failure indicated that, even accounting for untreated soil strength loss, the shear strength of the improved soil layer was about 50% of the design value. The Authors consider that the deep stabilisation method may not provide adequate interlocking between soil grouted columns, and thus low shear strength would prevail at the column interfaces.

Remedial measures were basically:
- removal of soil outside the trench to Gl –6m,
- groundwater lowering,
- strengthening of the 5th, 6th, and 7th strutting levels.

It should be commented that in the analysis of bottom heave failure mechanisms, the consideration of groundwater pressures acting against the improved soil layer at the bottom would be of significance in stability assessments. The fact that groundwater lowering was applied as a remedial measure is an indication toward that point.

7. ENVIRONMENTAL ASPECTS

7.1 *Nanjing subway environmental studies*

Ying describe the studies carried out in the assessment of environmental impacts for construction and operation of the Nanjing subway.

At construction stage the main concern is related to consolidation settlement in soft quaternary soils, if groundwater lowering is used for excavation

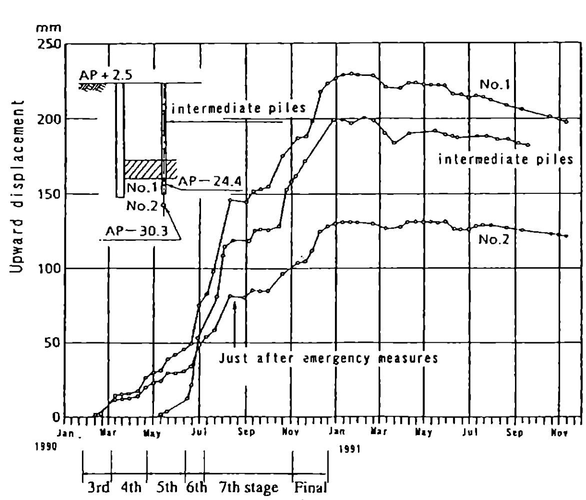

Figure 6: Bottom heave in Tokyo Cut&Cover tunnel

works, and it is understood that slurry or earth pressure balanced shields would be required.

At the operation stage the studies focused on vibration and noise of passing trains. Sand liquefaction analyses were carried out, the conclusion being that train induced vibration would not cause liquefaction.

Regarding noise, the Author preliminary analysis is that sound insulation lining boards might be necessary to comply with legislation, which specifies, for residential areas, 50dB(A) and 40dB(A), respectively at daytime and night-time.

7.2 *Hydro-geological conditions at Bucharest subway*

The complex hydro-geology and specific geotechnical conditions in Bucharest, are reported by Beldean, Ciugudean-Toma & Calinescu, related to the Metro system construction.

Bucharest is situated at the axis of a synclinal, where thickness of sedimentary deposits is greater than 1000m, with alternating clay and sand/gravel layers. There are two groundwater regimens, one phreatic and the other artesian. Groundwater level varies from 4 to 12m below surface level.

The Authors report that construction works suffered temporary interruptions due to lack of funds. At some sections the groundwater lowering system was discontinued, and soil erosion occurred through defective joints in diaphragm walls; in one case an old church suffered structural damage. Soil grouting was generally the remedy measure.

Another interesting measure was provision of permanent drainage paths, through a cut & cover structure, to avoid creating an impervious barrier to a free flowing acquifer. Without such drainage path provisions, groundwater level would rise on the upstream side of the cut & cover tunnel, and lower at the downstream side, both with negative effects on the urban environment.

8. CONCLUSIONS

Very interesting contributions were submitted for this Session, and readers are encouraged to look into the original papers, since it is impossible to duplicate their richness of details in this summary report. Some topics that deserve to be discussed further are listed below.

a) The geométrical and constructional flexibility of sprayed concrete lined tunnels are the major appeal of such technique over shield tunnelling.

However, this technique depends heavily on support construction details and also on adequate groundwater control, to be successful. Several of the submitted papers give emphasis on closing the invert arch as near as possible to tunnel face, and describe the adopted excavation stages to ensure that objective.

b) Groundwater control is fundamental to guarantee excavation stability. Groundwater lowering either with vertical wells or with horizontal drains is a common resource, but curiously the beneficial application of vacuum in the wells or drains was not mentioned in any case.

c) Compressed air was successfully applied to several construction sections in sprayed concrete lined tunnels. This technique should be considered more frequently, however careful analysis of the risks of pressure blow outs should be mandatory.

d) The adoption of empirical rock classification methods to define initial support was reported in several cases, for a variety of conditions ranging from fresh rocks to squeezing soils. The validity of these methods regarding weak waterbearing soils should be questioned, and some collapses or excessive deformation were reported in these materials; rational continuum models and limit analysis should be used to assess stability and deformation characteristics in soil type materials.

e) The benefits of ground anchors were described for some applications in weak rock, resulting in lighter sprayed concrete linings. An experimental centrifuge test in sands has also shown the benefits of ground anchors as face support systems. The use of ground anchors should be considered on a more broad basis, except for very soft soils, and similarly to what is occurring in the field of soil nailed slopes.

f) Monitoring systems constitute an integral part of sprayed concrete lined tunnel construction. However information on strain and displacement ahead of a tunnel face, which is fundamental in the assessment of stability, is restricted to shallow tunnels where deep settlement marks and inclinometers can be installed. Horizontal extensometers and deflectometers installed ahead of tunnel face (and above crown) should be further developed and used.

g) Horizontal borehole probings ahead of tunnel face were not mentioned in the reported case histories; in one of the cases a collapse occurred clearly due to unforeseen geological conditions. Horizontal borehole investigations should be a standard, particularly in deep tunnels where a vertical borehole is not practical.

h) An attempt was carried out to determine which of the case histories mentioned the occurrence of a (localised) collapse or presented excessive deformations, to the point of requiring adoption of remedial measures, compared to those where no incident was reported.

Seventeen cases were identified where such classification could be attempted, as summarised in figure 7 below.

This is rather arbitrary, and by no means should be viewed as a general trend or of any statistical value.

However it is a small sample illustration that failures are not a privilege of Sprayed Concrete Lined (SCL) tunnels, as one could be tempted to assume, after the great publicity given to accidents involving SCL tunnels, in recent years.

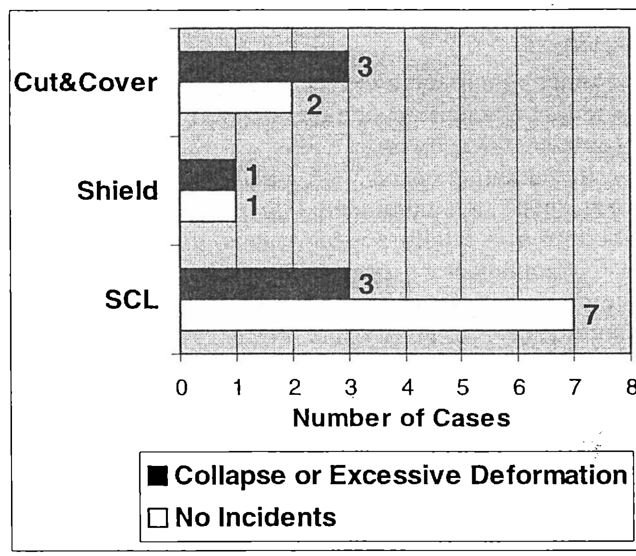

Figure 7: Performance comparisons between different construction method cases reported in this Session.