GEOTECHNICAL ENGINEERING FOR TBM TUNNELLING IN SOFT AND MIXED GROUND

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ABSTRACT: Pressurised face tunnel boring machines are widely used for construction of tunnels in soft and mixed ground. There are many geotechnical issues involved in planning and operation of this type of machine. Research and experience over the last 15 years has led to useful guidance for design and construction staff working on this type of tunnelling. Particular issues for pressurised face TBM tunnelling discussed in this paper are: site investigation, interpretative and baseline reporting, face pressure assessment, selection of conditioning agents, grouts for tail void grouting, prediction of ground movements and their effects on buildings, selection of building protection measures, assessment of geotechnical risk and instrumentation.

Keywords: Tunnel, settlement, risk, baseline, TBM

INTRODUCTION

A common method of tunnelling in soft and mixed (soil and rock) ground is to use a pressurised face tunnel boring machine (TBM). While these machines were first introduced nearly 50 years ago, the last 15 years has seen detailed research into key aspects of the operation of these TBMs, how they interact with the ground, and the resulting effect on overlying structures and utilities. This paper reviews the potential contribution of geotechnical engineers (in the broadest sense, including geologists and engineering geologists) in the planning and construction of TBM-driven tunnels in soft and mixed ground with respect to the following categories:

- site investigation and associated testing;
- interpretative and baseline reporting;
- calculation of target face pressures;
- selection of conditioning agents to suit the ground and groundwater conditions;
- grout mixes for tail void grouting;
- assistance in development of working procedures for tunnelling;
- assessment of ground movements due to tunnelling, and their effects on buildings and utilities;
- selection and design of ground treatment for tunnelling and building protection;
- input to risk assessment and risk management; and
- design of the overall layout of instruments to monitor the effects of tunnelling, and the interpretation of the results of the monitoring.

Design of the tunnel lining is not discussed in this paper. For pressurized TBM tunnels in soft and mixed ground the tunnel lining is commonly a reinforced concrete segmental lining. Although the design of a segmental lining involves some geotechnical issues, the primary issues for such design include structural capacity, corrosion protection and detailing.

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SITE INVESTIGATION AND TESTING

Site investigation and testing are a traditional area of the geotechnical engineer's practice, and geotechnical engineers have commonly been responsible for the planning and supervision of site investigation for tunnelling. Although this is a traditional area of practice, the planning of the site investigation needs to be adapted to provide particular information required for a TBM-driven tunnel.

The optimum amount of investigation

When planning a site investigation for a TBM-driven tunnel, it is useful to have a frame of reference for how much investigation would be considered adequate, based on past experience. Typically, the recommendations arising in past studies are expressed in terms of length of borehole per unit length of tunnel to be constructed. The basis for optimisation is by comparing the cost of additional borings with the cost of variations and/or claims resulting from inadequate investigation.

A study by the US National Committee on Tunnelling Technology, published in 1984, from a database of tunnelling in the USA, suggested an optimum borehole programme with 1.5m of borehole for each metre of tunnel. It should be noted that this study pre-dated the adoption of pressurised TBMs for tunnel construction in the USA. Later studies by Westland et al. (1998) and Shirlaw et al. (2003a) were for EPB tunnelling, in urban areas, and in soft and mixed ground conditions. These studies suggested that an overall programme of between 0.6m and 0.8m of borehole per metre of tunnelling was adequate to allow contractors to bid for the work. For example, if the boreholes are typically 40m deep this would give an average borehole spacing along the alignment of between 50m and 67m. It should be noted, however, that this length ratio range:

a) is for pressurised TBM tunnelling in urban areas where there is likely a substantial database of old boreholes that can be used to help define the conditions along the tunnel alignment;

b) is for new boreholes drilled specifically for the tunnel work and the old boreholes used to supplement the information should not be included;

c) should exclude any borehole that is not taken to at least the invert level of the tunnel – it has been common in Singapore to terminate boreholes once rock is reached, even if the tunnel is at a lower elevation and such boreholes are generally a waste of effort and money;

d) is to provide adequate information to allow the contractor to price the work though additional information is likely to be necessary to fill in detail for planning the construction of the tunnel

e) is derived per metre of alignment for twin running tunnels, and, in general, there is no need to double the number of boreholes for reasonably closely spaced twin tunnels, provided the boreholes alternate on either side of the tunnels to give information across and not just along the alignment; and

f) is for reasonably evenly spaced boreholes, although it is sensible to provide additional boreholes at probable shaft locations, along the tunnel alignment large gaps between borehole locations should be avoided, if possible.

Phasing of the investigation

Where possible, the investigation should be phased,. The first phase should consist of a study of background information ("desk-top" study). The second phase should include widely spaced boreholes, to confirm the main geological units along the alignment indicated by the desk-top study. Subsequent phases will involve increasingly more closely spaced investigation points, and more detailed testing. One common error is completing the full investigation before the alignment of the tunnel is finalised. Although this approach may be contractually convenient during the design phases, if the alignment is then changed significantly much of the investigation then needs to be repeated and some aspects may become irrelevant. The third and fourth phases of investigation should only be carried out once the alignment is generally fixed and should be planned to provide refined data for final design and construction planning and bidding..Westland et al. (1996) summarised a case study on the planning and use of the site investigation for the EPB driven tunnels on the Sheppard Subway Line in Toronto and this paper provides greater detail on the planning and phasing of an actual investigation.

Field and laboratory tests

In developing the field and laboratory testing programme it is essential that the particular information necessary to plan and operate the TBM is obtained. General guidance can be obtained from AFTES (2000). Some specific issues of importance for pressurised TBM tunnelling, but which are often inadequately addressed include:

a) Piezometric Pressures at Tunnel Level: For tunnels in soils which behave as 'drained' during excavation, the pore pressure is the dominant factor in assessing the face pressure to be used. It is therefore essential that accurate measurements of pore pressure are obtained at intervals along the alignment. It is also necessary to assess whether the groundwater follows a hydrostatic profile or whether upward or downward seepage gradients exist. Westland et al. (1996) state that every borehole drilled for the Sheppard Subway had at least one piezometer installed in it. They describe the use of multiple piezometric measurements to assess complex conditions that varied from downward to upward seepage. The measurements were also used to map the continuity of beds of sand.

b) Rate of Excavation: The speed of excavation in rock is dependent on a number of factors including the compressive and tensile strength of the rock, the frequency and orientation of the joints and the quartz content. Suites of testing to give rock cutting parameters are available from specialist laboratories.

c) Abrasion Characteristics: The abrasiveness of soil and rock affects the rate of wear on the cutting tools and other exposed parts of the machine. The cost of replacing cutters and of stoppage time for maintenance can be a significant proportion of the total cost of tunnelling. Abrasion testing is necessary in rock, and for some particularly abrasive soils. As an example of the latter, Peart et al. (2001) record that the Old Alluvium soils in Singapore are as abrasive as granite.

INTERPRETATIVE AND BASELINE REPORTING

Generally, Interpretative Geotechnical Reports are prepared for use in developing the design, whereas Geotechnical Baseline Reports (GBRs, ASCE 2007) are contractual documents used to define the anticipated distribution, nature and behaviour of the various ground conditions along the tunnel alignment. Interpretative Geotechnical Reports are a common part of design, so will not be discussed here.

While Geotechnical Baseline Reports have been used for several decades in North America, their use is now becoming widespread since the Code of Practice prepared by the International Tunnel Insurers Group (2006) includes a requirement for the preparation of either a Baseline Report or Ground Reference Conditions report. Guidelines for the preparation of Geotechnical Baseline Reports are published by the ASCE (ASCE 2007). Unfortunately, many Geotechnical Baseline Reports are now being prepared by engineers without experience in their use, and are often little more than Interpretative Reports given a new name. Key issues that differentiate a Baseline Report from a conventional interpretative report are:

- a focus on issues relevant to the construction of the work, and particularly those affecting cost and progress;
- assessment of the behaviour, and not just the parameters, of the ground (though GBRs prepared for Design-Build projects may not assess ground behaviour in relation to construction);
- development of the anticipated stratigraphy between borehole locations – an example from an actual GBR is given in Figure 1;
- quantification of the baseline geotechnical conditions, as far as possible – for example, quantification of likely groundwater inflow, or quantification of the volume, size, number and strength of boulders to be encountered as illustrated in Fig. 2.

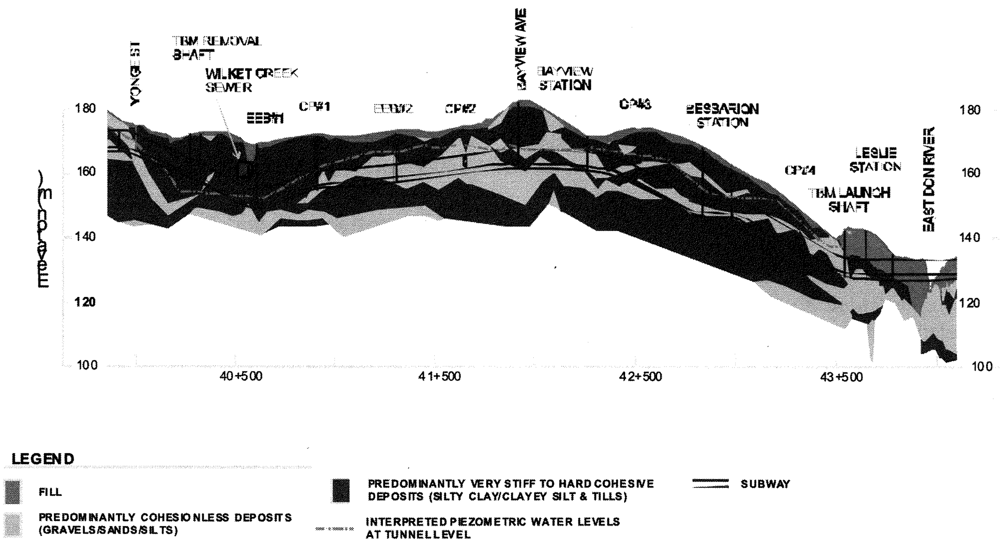

Fig. 1 The western section of the Sheppard line tunnels, the baseline stratigraphy as shown in the GBR that was issued as part of the tender documents

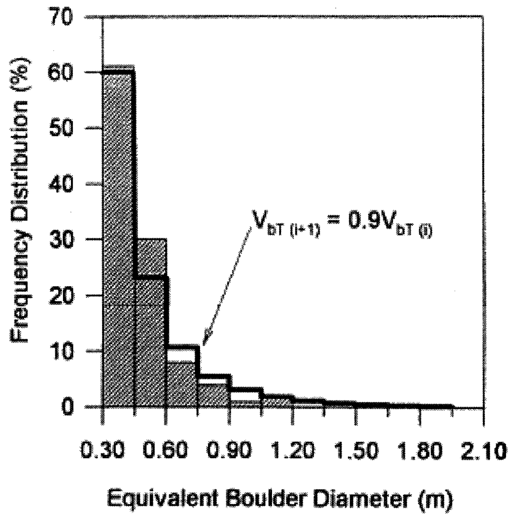

Fig. 2 Distribution of anticipated boulder sizes, as given in the GBR for the Sheppard twin tunnels, Toronto

In all GBR documents, whether these include discussions of anticipated ground behaviour in relation to construction or not, the language and definition of conditions is unambiguous. When ranges are provided for geotechnical parameters or conditions, it is generally advisable to provide clarity on the statistical basis for these ranges so that the risks associated with using values within the range can be assessed in a quantitative manner.

Quantification of behaviour during construction and baseline conditions generally prove most problematic to geotechnical engineers without previous experience of the preparation of such documents. In addition to the guidance given in Anon (2007), publications on the preparation of Baseline Reports include ASCE (2007), Essex et al. (2007), and Shirlaw et al. (2007).

The latter paper discusses the development of the GBRs for two EPB driven tunnels in Toronto.

CALCULATION OF TARGET FACE PRESSURES

When using a pressurised TBM, an assessment has to be made of the face pressure to be used. Where ground and/or groundwater conditions change, the face pressure changes have to be implemented at the correct time. As discussed in Shirlaw et al. (2003b), there is a high risk of exceptionally large volume loss when tunnelling through the interface between beds of dissimilar nature if there is inadequate planning or management of the requisite face pressure changes.

It is now standard practice in Singapore for the face pressure to be calculated at intervals of 2m to 7m along the tunnel alignment, prior to the start of tunnelling. The calculated face pressures are based on theory, such as those of Anagnostu and Kovari (1996) for granular soils, or model centrifuge test results, such as those in Kimura and Mair (1982) for clay. The calculated face pressures are then used to develop the planned face pressure, taking into account the expected fluctuation of the face pressure during excavation.

Planning the face pressures to use when tunnelling near interfaces between dissimilar materials, such as soft clay and hard clay, requires particular consideration. Even if the tunnel is in a full face of the stronger material (such as hard clay), the presence of a much weaker material above influences the stability of the face. The influence of the weaker material is greater the closer it is to the tunnel. Leblais et al. (1996) record a number of sinkholes that appeared during tunnelling for the Lille Metro when the face pressure was inadequate to cater for the presence of soft clay just above the tunnel. Mair (1993) discusses how this condition can be allowed for in calculating the necessary support pressure.

Where interfaces between materials of contrasting strength have a major effect on the calculated face pressure, the identification of those interfaces becomes a major issue. For accurate calculation, the ground investigation has to provide accurate information on the location of the interface at a similar spacing as the calculations, i.e. 2m to 7m along and across the tunnel alignment. In exceptional circumstances such a level of investigation may be warranted. However, in most cases there will be much less information, and there will be a degree of uncertainty in the actual level of the interface. In such circumstances conservative assumptions must be made about the location of the interface; in particular, no reliance should be placed on simple interpolation between widely spaced boreholes.

Although the face pressures are planned prior to tunnelling, there needs to be flexibility to adjust the pressures in response to the observed behaviour of the ground. Key observations, during TBM tunnelling, are:

- the volume of material excavated, relative to the theoretical volume;
- the face pressure when the machine is not excavating; and
- the settlement over the machine

The assessment of volume excavated is not very accurate, possibly +/- 10% for most pressurised TBMs. However, large excess volumes do indicate the potential for large settlements or the formation of a sinkhole, and are probably due to the use of inadequate face pressure. Constant monitoring of the excavated volume will provide some warning of major over-excavation, and will provide information to help assess whether the face pressure being used is appropriate for the ground and groundwater conditions. Although recent advances have been made in automatically measuring discharge volumes (e.g., through conveyor weight, laser volume measurements), these systems are also subject to systematic error and manual methods (e.g., visual observations and recording of muck car numbers and volumes) can provide valuable and immediate back-up information.

When the machine is not excavating, such as when the ring is being built, the pressure in the excavation chamber will tend to rise or fall to balance the static pressure from the soil and the groundwater, thus giving a check on the calculated face pressures. The face pressure during tunnel advance needs to be higher than the static level, to cater for fluctuations in pressure and to minimise settlement, so if the face pressure is correctly calculated the face pressure should drop slightly during the ring build part of the cycle. Geotechnical and tunnel engineers should understand the reasons for and typical range of variability in face pressure induced by machine operational parameters (e.g., changes induced by discharge gate aperture, screw conveyor RPM, face door aperture, etc.).

Excessive settlement may be due to a number of factors, including the use of an inadequate face pressure. The causes for the excessive settlement need to be investigated, as inadequate tail void grouting or consolidation of the ground can also cause excessive settlement. If there are any indications that the planned face pressure is insufficient, such as clear over-excavation, or an increase in the face pressure when the machine is not advancing, then the staff in the tunnel should quickly increase the face pressure.

In addition to the immediate review of behaviour in the tunnel, there needs to be regular review of the tunnel performance and the basis of the face pressure calculations, by the tunnel manager and the person who calculated the face pressures. These meetings are necessary to assess the information from the tunnel, monitoring data on ground movements, and any new information on ground and/or/ groundwater conditions, and decide whether there should be a revision of the face pressure calculations. In Singapore, where conditions for tunnelling are highly variable, such meetings are commonly held on a weekly basis, with the provision for additional ad hoc meetings if unexpected behaviour is observed.

When tunnelling through abrasive soil, rock, or a mixed face of soil and rock, it is necessary to access the excavation chamber on a regular basis, to inspect, and where necessary replace, the cutting tools and other exposed parts of the machine. It is common to use compressed air or ground treatment to allow safe access to the head. The distribution of compressed air pressure over the height of the face is different to that of slurry, so the target pressure for compressed air entry is generally slightly different to the pressure required for slurry or EPB tunnelling, and needs to be calculated separately. As discussed in Shirlaw and Hulme (2008), the calculated target face pressures need to be communicated to the tunnelling crew on a daily basis.

SELECTION OF CONDITIONING AGENTS TO SUIT THE GROUND AND GROUNDWATER CONDITIONS

For EPB tunnelling, it is common to inject additives to 'condition' the spoil, to improve the properties of the spoil and aid EPB operation. Facilities for the injection of conditioning agents are typically provided at the cutting head, in the excavation chamber and in the screw conveyor. Conditioning agents can be used to:

- Increase or reduce the undrained shear strength of the spoil to increases or reduce the pressure differential along the screw conveyor, between the excavation chamber and the discharge point, and also to change the torque required to turn the screw conveyor. (see Merritt and Mair 2006 and 2008);
- Reduce the permeability of the spoil – if the spoil is too permeable the groundwater can flow freely up the screw conveyor ;
- Provide some elasticity in the spoil to provide greater resilience in the pressure control;
- Reduce the wear on the cutting tools and on other parts of the machine in contact with the spoil to consequently reduce the need for interventions into the head for maintenance, saving cost and time and reducing the number of interventions reduces risk.

There are many EPB additives on the market, including various types of foam and polymers. As discussed in Boone et al. (2005) and Borghi (2006), these generic categories cover materials with significantly different properties. It is therefore important to select materials which are compatible with the ground conditions and the groundwater chemistry.

Trial mixes of additives with samples obtained during site investigation, piling or shaft excavation can be used to help identify the appropriate conditioning agents and the approximate dosage levels required. Although some generic guidelines have been produced for assessing the use of "foam" (EFNARC 2003), the physical and chemical properties of the "foam" can have a significant effect on field performance and the dosage rates suggested by such guidelines may or may not be appropriate. For example, some "foam" materials can act as dispersants (leading to greater flow properties), while others, through use of long-chain polymers, can act as coagulants (leading to more apparent cohesive behaviour in granular soils).

GROUT MIXES FOR TAIL VOID GROUTING

As discussed in Shirlaw et al. (2004) the tail void grout injected around segmental linings has a number of important functions. In particular, the grout should:

- ensure that there is uniform contact between the lining and the ground;
- reduce the surface settlement over the tunnel;
- hold the ring in place during shield advance;
- carry the load transmitted to the lining by the shield back-up; and
- reduce seepage and loss of fine particles, if the gasket is ineffective due to damage or stepping of the lining.

The grout can only achieve these objectives if the tail void is filled completely with grout that has sufficient internal shear strength to resist the imposed loads. To achieve consistent filling of the tail void, particularly in soft and loose soils, it has been found that it is necessary to inject the grout simultaneous with TBM advance. This can be achieved with grout pipes placed along the tailskin, and passing under the tail seals (Figure 3). With this method, excellent and even grouting can be achieved even in soft clay. Figure 4 shows an example of set grout around the lining of a tunnel built in the soft marine clay in Singapore; the grout had been injected using simultaneous tail void grouting.
The example shown in Fig. 4 was obtained during the construction of a cross-passage to an escape shaft, during which the extrados of the segment was exposed.

In normal circumstances the grout is not visible, and the effectiveness of the grouting can only be checked by probing through the grout ports.

The grout is fully confined by the lining and the ground, so only a relatively small shear strength is required. However, some of the most critical loads on the grout are due to the thrust from the TBM and flotation forces.

These loads occur relatively soon after the grout is injected. Bezjuin and Talmon (2008) discuss the measured buoyancy and thrust forces on a segmental lining in the Netherlands, and the importance of restricting the length of lining that is surrounded by liquid grout. The development of internal shear strength in the grout in the hours after placement is more important to achieving the objectives listed above than the long term strength of the grout.

There are two types of grout in common use for TBM tunnelling: two-component, fast gelling (accelerated) grouts; and mortar grouts, which rely on high internal friction to provide resistance to displacement.

The accelerated grouts are relatively simple: typically cement and sodium silicate are mixed at the point of injection. Initially, cation exchange between the cement and the sodium silicate causes the sodium silicate to gel, giving some short term strength. Over the longer term the cement gives increasing strength.

The mortar grouts are more complex, as the rheology of the grouts is critical to their function. A wide range of materials can be used to make such grouts, and the ingredients are determined by cost and availability of suitable materials at a particular location.

Trial mixes, and the testing of those mixes, is therefore an important part of the development of suitable, but cost effective, mortar grouts. The development of suitable mortar grouts for filling the tail void, and tests for those grouts, were the subject of the EUPALINOS project in France (AFTES 2002).

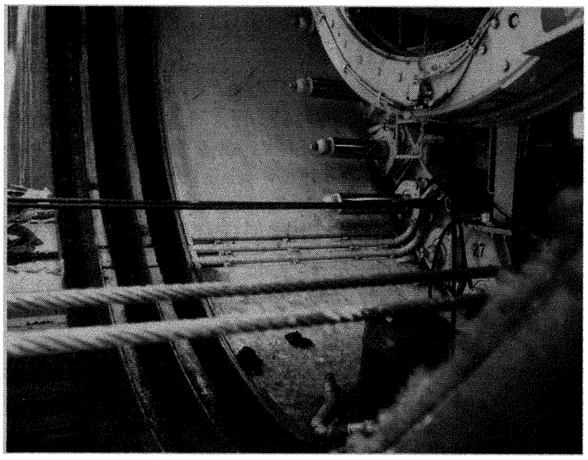

Fig. 3 Grouting pipes laid along the tailskin of an EPB TBM Figure

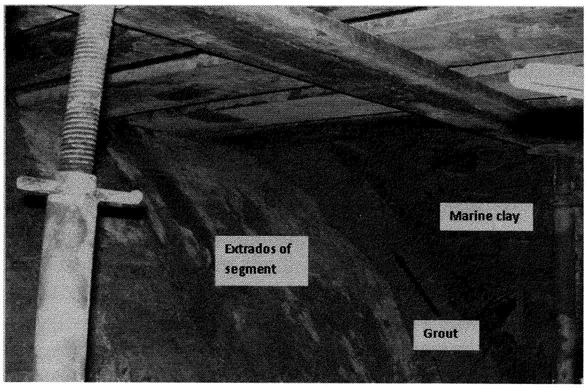

Fig. 4 Tail void grout around an EPB driven tunnel in Singapore marine clay

It is common to attempt injecting a greater volume of grout than the theoretical minimum volume of the tail void.

Typically, target injection volumes are at least 120% of the theoretical tail void, or higher. This margin allows for, inter alia:

a. Increases in the size of the tail void due to driving around curves or with the shield inclined in the vertical plane: the overcut due to these factors cannot be calculated in advance of tunnelling as this is subject to the actual operation of the machine
b. Loss of volume due to bleeding of the grout under pressure

Although bleed tests are commonly prescribed for grouts, these are rarely carried out under pressure. Pressure will increase any tendency grouts have to bleed. Komiya et al. (2001) record a significant amount of bleed, under pressure, of cement/silicate grouts, of the type commonly used for tail void grouting in Asia. Much of the bleeding under pressure occurred after the initial set of the grout.

Once the tail void is fully grouted, further injection of grout is generally of little benefit. In soft clay, the excessive volume of grout will generate positive excess pore pressures; the dissipation of these pore pressures will negate any beneficial effect the excess grout has in terms of compensating for the settlement due to tunnelling (see Shirlaw and Doran 1988, Shirlaw et al. 1999).

ASSESSMENT OF GROUND MOVEMENTS DUE TO TUNNELLING, AND THEIR IMPACT ON BUILDINGS AND UTILITIES

In an urban area, it is necessary to predict the effect of the tunnelling on the buildings, utilities and other structures above (and, occasionally, below) the tunnel. This involves predicting the ground movement induced by the tunnelling, and then assessing what degree of damage this causes.

For urban tunnelling, it is common for hundreds, or thousands, of buildings and utilities to be within the zone of influence of the tunnel, all with differing geometry with respect to the tunnel, and with varying structural and foundation details. With the need to assess the effect on so many buildings and utilities, simple methods of analysis are applied initially, to screen out buildings that are unlikely to suffer significant damage. This typically involves assessing a value for the 'volume loss' due to tunnelling, based on empirical data. The volume loss is used to calculate the settlement and horizontal ground movement along the alignment. As outlined in methods proposed by Boscardin and Cording (1989), Mair et al. (1996), Boone (1996, 1999, 2008), Burland (2008), and Cording (2008), strains due to bending, differential settlement of column foundations, and horizontal extension can then be calculated, and used to assess the predicted degree of damage. Each of these referenced methods identify important elements related to structure damage induced by tunnelling and excavation displacements and how the peculiarities of some assumptions regarding the structures or tunnelling affect the outcome of design analyses. These basic methods can be set up in spreadsheets, readily allowing the assessment of many buildings and services. Once this initial screening process is complete, a few buildings/structures or services may be identified as being at risk of excessive damage. More detailed analysis can then be carried out for these selected structures and utilities.

In Singapore it has been found that almost all of the buildings that have suffered significant damage due to tunnelling or excavations are buildings on mixed foundations or where the ground conditions vary significant under buildings on shallow foundations (Shirlaw et al, 2003c). These buildings are unusually sensitive to settlement, much more sensitive than buildings on uniform foundations or underlain by reasonably consistent ground conditions. During initial assessments the compatibility of the assessment method with the particular structure types must be clarified (i.e., one should not apply a method for assessment of load bearing masonry walls to a concrete frame building or a building on mixed foundations).

A key factor in the analysis is the "volume loss" that is assumed to occur during tunnelling. The actual volume loss expressed at the surface is due to a combination of the ground conditions, the tunnelling methods adopted and the quality of the workmanship. The volume loss is directly affected by the magnitude and variability of the face pressure, the quality of the tail void grouting, and a number of other factors. It must be recognised during any assessments of potential performance based on such volume loss assumptions, that the assumptions represent only one discrete outcome and that there can be a range of possible performance. Whether a "worst case", "average case", or "best case" scenario is chosen for design assessments, the probability of any of these actually occurring should be carefully examined in order to formulate a rational design approach (e.g., Boone 2006). Although the assessment of settlement and the effect on buildings is commonly (but not necessarily) carried out by geotechnical engineers, this should not be an isolated exercise. There need to be iterative loops, initially with the development of tunnelling procedures, and then with the monitoring of actual settlement and the re-evaluation of face pressures and procedures during construction.

It is quite common, in Asia, to consider the prediction and evaluation of building and utility damage as part of the risk assessment and management process. In fact, the calculation and prediction of building and utility damage, and the associated protection measures, should be considered as part of the standard design process for urban tunnelling. The risk assessment should also consider the risk underlying the designer's assumptions and methods of analyses regarding, inter alia,:

- ground conditions;
- tunnelling performance;
- volume loss / settlement due to tunnelling; and
- building and utility condition and response

SELECTION AND DESIGN OF GROUND TREATMENT AND UNDERPINNING FOR TUNNELLING AND BUILDING PROTECTION

After assessing the potential impact of the tunnelling on buildings, structures and utilities, it may be necessary to implement protection measures at selected locations along the tunnel. Protection measures may include ground treatment (such as grouting for compensation or strengthening, or ground freezing), ground improvement (such as protection walls or pali radice) or underpinning.

Structural solutions, such as structural strengthening, separation or propping may also be used on occasion.

In addition to building protection, it is common to use limited areas of ground treatment, typically by grouting or freezing, on projects involving TBM tunnelling. Reasons for the use of ground treatment, other than building protection, include:

- for shield launching, and/or launch chamber construction;
- for recovery of the shield at the end of the drive, or for shield to shield docking;
- for cross-passage, escape shaft or safe refuge construction; and
- for the construction of 'safe havens', where major maintenance can be carried out.

The zone, strength and testing of the ground treatment need to be defined by the designer.

INPUT TO RISK ASSESSMENT AND RISK MANAGEMENT

Key risks in tunnelling are the nature and behaviour of the ground. In carrying out the risk assessment for the tunnel, the potential degree of variation in the conditions described in the Geotechnical Baseline Report needs to be assessed, based on the geological model. The GBR defines the anticipated ground conditions in order to delineate the contractual basis for risk sharing between the owner and the contractor, and focuses on the anticipated (most probable) conditions. In contrast, the risk assessment needs to focus on unlikely, but possible, conditions where these have the potential for a high impact on cost, schedule, program or safety, if encountered. As discussed in Anon (2007), the GBR and risk assessment/risk register are complementary documents.

The settlement prediction and building damage assessment exercises are often based on discrete assumptions for the volume loss due to tunnelling. There is a risk that settlements will be higher than predicted, or even that a sinkhole will develop over the tunnel. If the settlement prediction had a reasonable basis, such higher settlements / sinkhole(s) are likely to be local, and could be due to:

- poor planning or procedures;
- failure to follow the procedures;
- mechanical failure; and
- significantly more adverse ground conditions than anticipated.

General guidance on risk management for tunnelling is given in Anon (2005b). For EPB tunnelling, Shirlaw et al. (2003) and Shirlaw and Boone (2005) provide information on the circumstances where excessive settlement or sinkholes are most likely to develop, and Shirlaw et al. (2005) discuss risk mitigation measures for the development of sinkholes.

The risk assessment for a tunnel will cover much more than just geological risk and the risk of excessive settlement or sinkhole development (e.g., TBM operations, personnel training, obstructions, mechanical failures, etc.). However, these risks are potentially some of the most severe, in terms of cost and the possible delay to the project.

It is therefore important that geotechnical engineer(s) that understand the development of the geological model, the face pressure calculations and the settlement predictions take part in the risk assessment and risk management process. Anon (2005b) states that: 'Qualified geotechnical professionals with relevant tunnel works experience should be employed to carry out the geotechnical risk identification, the GRA reviews and the preparation and updating of the risk register and the GRMP' This document refers to tunnels in Hong Kong. GRA is a Geotechnical Risk Assessment, while GRMP is a Geotechnical Risk Mitigation Plan. Other countries do not necessarily follow this practice of separating geotechnical risk from other risk during tunnelling. However, it should be a common principle that it must be geotechnical professionals with tunnel experience that evaluate the geotechnical risks in tunnelling. It is advantageous to obtain input to the Geotechnical Risk Assessment from someone not directly involved in the original design, as there is a danger that the designers and constructors are overly confident of the accuracy of their model/calculations/predictions, and do not properly recognise the limitations of their assessment and assumptions.

ASSISTANCE IN THE DEVELOPMENT OF WORKING PROCEDURES FOR TUNNELLING

Prior to the start of tunnelling, detailed working procedures need to be developed for the tunnelling. These are normally prepared by the tunnelling management team. The tunnelling procedures have to be consistent with design, risk management and planning for the tunnel, as these typically make assumptions about the nature, sequence and quality of the work during tunnelling. In particular:

- the nature and likely behaviour of the ground, as summarised in the Geotechnical Baseline Report, is a major consideration when developing the tunnelling procedures;
- there are important relationships between the tunnelling procedures (including the face pressure applied and the type and quality of the tail void grouting) and the resulting settlement and damage to buildings and utilities;
- the risk assessment and management process w ill identify risk mitigation and contingency measures and the fully developed tunnelling procedures have to be consistent with the outlined risk assessment and mitigation/contingency process.

It is therefore necessary that the procedures are either prepared or reviewed by someone with a full knowledge and understanding of the GBR, risks related to the ground, and the face pressure, settlement and building damage calculations.

DESIGN OF THE OVERALL LAYOUT OF INSTRUMENTS TO MONITOR THE EFFECTS OF TUNNELLING, AND THE INTERPRETATION OF THE RESULTS OF THE MONITORING

Geotechnical engineers have commonly been involved in the design of the surface and sub-surface instrumentation layout for tunnels.

A good knowledge of the anticipated ground conditions and the building and utility damage assessment is needed in the development of the layout. There is, however, a danger that the designer concentrates instruments at places where problems are anticipated, while ignoring the areas where problems are not expected. The actual problems during tunnelling usually develop where they are not anticipated, as measures are taken to avoid problems where they are anticipated. It is therefore necessary to have a basic monitoring grid all along the tunnel, at least in urban areas. Examples of standard instrumentation layouts for TBM tunnelling in Singapore are given in the Singapore Land Transport Authority's Civil Design Criteria (2008).Data from monitoring needs to be checked against review levels derived from the anticipated ground and buildings movements, and anticipated changes in piezometric level. The data can also be used to assess the causes of the movement, such as separating the settlement into components due to losses at the face, at the tail void, and due to consolidation. Importantly, data from areas that exhibited routine tunnelling quality control can be used to highlight areas in which significant variation (from the routine) occurred and can be used for empirical design analyses and risk assessments for future projects in similar ground conditions (and this may be of great value for large transit agencies).

DISCUSSIONS

Many activities related to the design, planning and construction of soft and mixed ground TBM driven tunnels are listed above. Some of these are commonly carried out by geotechnical engineers, such as site investigation, preparation of Geotechnical Reports, settlement and building damage assessment, and monitoring. A number of other activities, also listed above, are not necessarily undertaken by geotechnical engineers. However, these other activities need consideration of the ground conditions, and all of the activities need to be integrated for a particular tunnel, so that the design, planning and construction are consistent.

The technical issues discussed here are part of a field that is still rapidly developing. A combination of experience, operational and academic research is leading to a constant growth in knowledge on the various issues covered in this paper. The proceedings of the regular conference of Technical Committee 28 of the ISSMGE, Geotechnical Aspects of Soft Ground Construction in Soft Ground (1996, 1999, 2002, 2005 and 2008) provide a useful reference for the development of work in this field. National societies, including ASCE, BTS and AFTES have published a number of general summaries related to some of the topics discussed in this paper, and some of these are referenced below.

There has been an increasing amount of research into issues relevant to pressurised TBM tunnelling. Much of this research is of direct value for practising engineers, and some of the references in this paper are to relevant research work.

However, there is much more still be learned from further research, particularly on conditioning agents for EPB driven tunnels, and suitable tests for conditioned spoil. Other areas where research could provide useful information, with practical application, include:

a). the amplitude and frequency of vibration due to tunnelling in granular soils and mixed ground, how the vibration varies with the speed of rotation, cutting tooth configuration, and forward thrust, and the effect of the vibration on the on face pressure required for stability:

b) the bleeding of typical tail void grout mixes under pressure, both before and after setting

c) comprehensive case studies on the ranges of settlement (and volume loss) measured over pressurised TBM tunnels, with particular comment on the frequency and cause(s) of the outliers – areas of local but uncharacteristically large settlement (e.g., Shirlaw et al, 2003b); and

d) the relationships between TBM operational parameters and actual face pressures (as opposed to pressures measured on the forward chamber bulkhead) including face aperture, screw conveyor RPM, discharge gate aperture, head RPM, and forward thrust pressures (forces).

CONCLUSIONS

Geotechnical engineering in relation to TBM tunnelling in soft and mixed ground is a rapidly developing field. Much has been learned and documented, particularly over the last ten years.

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