

GROUND CONTROL FOR SLURRY TBM TUNNELLING

GEO REPORT No. 249

Golder Associates (HK) Ltd

**GEOTECHNICAL ENGINEERING OFFICE
CIVIL ENGINEERING AND DEVELOPMENT DEPARTMENT
THE GOVERNMENT OF THE HONG KONG
SPECIAL ADMINISTRATIVE REGION**

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Guidelines for the Auditing of Aspects of Ground Control for Slurry
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PREFACE

In keeping with our policy of releasing information which may be of general interest to the geotechnical profession and the public, we make available selected internal reports in a series of publications termed the GEO Report series. The GEO Reports can be downloaded from the website of the Civil Engineering and Development Department (<http://www.cedd.gov.hk>) on the Internet. Printed copies are also available for some GEO Reports. For printed copies, a charge is made to cover the cost of printing.

The Geotechnical Engineering Office also produces documents specifically for publication. These include guidance documents and results of comprehensive reviews. These publications and the printed GEO Reports may be obtained from the Government's Information Services Department. Information on how to purchase these documents is given on the second last page of this report.



R.K.S. Chan

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December 2009

FOREWORD

In 2007, the Geotechnical Engineering Office retained Golder Associates (HK) Ltd to provide expert advice in relation to tunnelling. One of the tasks identified was to provide advice on the ground control during slurry TBM tunnelling. A report was prepared to provide guidelines for the auditing of the design calculations and work procedures relating to ground control during slurry TBM tunnelling in Hong Kong. There have been a number of major reports on pressurised TBM tunnelling in recent years including BTS/ICE (2005) and publications by international and national tunnelling associations (see ITA WG-14 (2007)). It is not intended to duplicate the general advice given in these documents, but to provide more specific guidelines on issues relating to ground control for slurry TBM tunnelling in Hong Kong. The focus of the report is on key issues that affect the magnitude of ground movement due to tunnelling, and therefore the potential impact on third parties. The report was prepared by Mr Nick Shirlaw, with contributions from Dr Storer Boone.

The aim of this GEO Report, which is an update of the Golder report following review (see below) and amendment, is to share technical knowledge on the subject with the industry. The content of this GEO Report may require further updating as more experience in slurry TBM tunnelling and performance data in local ground conditions become available. Users applying the guidance in this GEO Report should take into account the actual ground conditions, the monitoring and controls available for ground control in the TBM chosen for the project, and the specific risks that the TBM will introduce, in preparing the design calculations, risk registers and work procedures, and in planning the risk mitigation measures and contingency plans for ground control. In all cases, the risk owner and key personnel assigned to control ground risk posed by the TBM operation must be clearly identified, and experienced tunnelling and geotechnical professionals must be employed to carry out and take responsibility for the design, site supervision and risk management.

A draft of this GEO Report was circulated to the HKIE Geotechnical Division's Working Group on Cavern and Tunnel Engineering, the IMMM (HK), the AGS (HK) Ltd, the MTRCL, Richard Lewis of YL Associated Ltd, Lars Babendererde of Babendererde Engineers and Darren Page of OtB Engineering Ltd for review. Useful and constructive comments and suggestions for improvement were received. All contributions are gratefully acknowledged.



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ABSTRACT

In 2007, the Geotechnical Engineering Office retained Golder Associates (HK) Ltd to provide expert advice in relation to tunnelling. One of the tasks identified was to provide advice on the ground control of slurry TBM tunnelling. A report was prepared to provide guidelines for the auditing of the design calculations and work procedures relating to ground control during slurry TBM tunnelling in Hong Kong. There have been a number of major reports on pressurised TBM tunnelling in recent years including BTS/ICE (2005) and publications by international and national tunnelling associations (see ITA WG-14 (2007)). It is not intended to duplicate the general advice given in these documents, but to provide more specific guidelines on issues relating to ground control for slurry TBM tunnelling in Hong Kong. The focus of the report is on key issues that affect the magnitude of ground movement due to tunnelling, and therefore the potential impact on third parties.

This GEO Report is an update of the Golder report following review and amendment. It should be used as a guide only. The content of this GEO Report may require further updating as more experience in slurry TBM tunnelling and performance data in local ground conditions become available. Users applying the guidance in this GEO Report should take into account the actual ground conditions, the monitoring and controls available for ground control in the TBM chosen for the project, and the specific risks that the TBM will introduce, in preparing the design calculations, risk registers and work procedures, and in planning the risk mitigation measures and contingency plans for ground control. In all cases, the risk owner and key personnel assigned to control ground risk posed by the TBM operation must be clearly identified, and experienced tunnelling and geotechnical professionals must be employed to carry out and take responsibility for the design, site supervision and risk management.

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1. SCOPE AND OBJECTIVES

The objectives of this report are to provide background, calculation methods, references and other information about ground control during slurry TBM tunnelling. The scope of the report includes:

- (a) determination of the appropriate face pressure to support the ground,
- (b) establishing limiting pressures and volumes for tail void grouting,
- (c) the use of excavation management control system, and
- (d) high risk activities (break-in, break-out, head access, interfaces, wear, flushing).

The report concerns ground control during tunnelling using a slurry TBM in Hong Kong, in the superficial deposits and in weathered granitic and volcanic rocks. For tunnelling in other strata, such as in marble, additional considerations will apply, due to particular geological features. Risk assessments and slurry treatment are referred to, but not discussed in detail. Readers should refer to other references on these subjects.

This report provides guidance on the review of design calculations, drawings which show the planned face pressures, and documented work procedures. It is required that these documents and drawings are prepared prior to tunnelling, to help to minimise the risk of unacceptable ground movements. While the preparation of appropriate design calculations, drawings and work procedures is a necessary precursor to tunnelling, such preparation is only part of the process and will not control all of the risks associated with the tunnelling. Other factors can lead to excessive ground movements. These factors would include:

- (a) poor decision making by the tunnelling staff,
- (b) priority given to cost and programme control over ground control,
- (c) mechanical breakdown, and
- (d) unexpected ground behaviour.

The risk of an incident due to such factors can be reduced and/or the consequences controlled by employing sufficient qualified and experienced staff, allowing sufficient time in the construction programme for preparing and reviewing the design calculations and drawings, and developing suitable risk mitigation measures, contingency plans (including for dealing with mechanical breakdowns) and work procedures before tunnelling commences. Most importantly, management of the tunnelling by experienced tunnelling staff and geotechnical professionals is essential to manage the risk associated with TBM tunnelling.

2. SYMBOLS AND GLOSSARY

The papers referred to in this report often use different symbols to refer to the same parameters, such as the depth to the axis level of the tunnel. In order to provide a consistent document, tunnelling related symbols and a brief glossary are given below. Terms for which there are standards, such as for soil strength, are not provided in the list of symbols.

2.1 Symbols

Dimensions (see Figure 1)

C	Cover over the tunnel (depth from ground level to the highest excavated point of the tunnel)
D	Excavated diameter of the tunnel
Z_O	Depth from the ground surface to the axis level of the tunnel
Z_{S1}	Depth from the ground surface to pressure sensor 1
Z_{S2}	Depth from the ground surface to pressure sensor 2
Z_{S3}	Depth from the ground surface to pressure sensor 3
Z_W	Depth from ground surface to the water table, or to the piezometric level at the tunnel, as appropriate ¹
D_I	Internal diameter of the lining
D_O	External diameter of the lining
P	Unsupported length of the tunnel heading

Unit Weight

γ	Bulk unit weight of the ground (soil or rock, as appropriate)
γ'	Submerged unit weight of the ground (soil or rock, as appropriate) ²
γ_d	Dry unit weight of the ground (soil or rock, as appropriate)
γ_{SL}	Unit weight of the slurry
γ_w	Unit weight of water

Pressure and external loads

P_S	Slurry pressure in the excavation chamber
P_{St}	Target slurry pressure in the excavation chamber
$P_{St(Crown)}$	Target slurry pressure at the crown of the tunnel

¹ This report assumes a simple hydrostatic distribution down to the level of the tunnel. The equations provided may not be appropriate for more complex groundwater regimes, the influence of which should be assessed carefully in the design.

² This report is for tunnelling through saturated soils below the water table.

$P_{St(S1)}$	Target pressure at sensor 1 (or (S2) for sensor 2, etc.)
$P_{S(S1)}$	Slurry pressure measured at sensor 1 (or (S2) for sensor 2, etc.)
P_A	Air pressure in the plenum chamber
P_{At}	Target air pressure in the plenum chamber (setting on the air pressure control device)
P_{CA}	Compressed air pressure
v	Maximum variation in applied slurry pressure, due to control accuracy, with v being the change in pressure above OR below the target pressure (see Figure 2)
q	Average surcharge pressure at the ground surface

2.2 Glossary

General terms related to tunnelling, used in this report:

Extrados	The outside face of a structural element, e.g. tunnel lining extrados.
Invert	The bottom surface of a tunnel.
Spring line	In a circular tunnel, the spring lines are at the opposite ends of the horizontal centreline. For a circular tunnel the spring line is also known as the axis level.
Overcut Gap	The annulus around the shield skin caused by the difference between the excavated diameter and the outside diameter of the shield skin. The size of the potential annulus is affected by steering of the shield, as well as by the degree of overcutting.
Tail Void	The annulus around the extrados of the tunnel, due to the difference between the outside diameter of the lining and the outside diameter of the shield skin. Also known as the tail gap or annular space.

Terms relating to slurry TBM tunnelling, used in this report:

Intervention	Access into the forward chamber(s) of the machine.
Double chamber slurry TBM	A slurry TBM in which the pressure of the slurry is controlled by using compressed air. The forward section of the TBM includes two linked chambers: the excavation chamber and the plenum chamber (see Figures 3 and 4). The compressed air bubble is in the plenum chamber. This type of TBM includes the 'Mixshield' (a registered trademark of Herrenknecht AG) and the Benton Air TBM (a registered trademark of NFM Technologies).

Pressure bulkhead	The pressure bulkhead separates the forward chamber(s) of the slurry TBM from the rest of the TBM, separating the pressurised area from the non-pressurised area (Figure 4).
Shove	Advance of the machine to allow one ring to be installed.
Submerged wall	The wall between the excavation and plenum chambers in a double chamber slurry TBM. The submerged wall has an opening in it which connects the excavation chamber to the plenum chamber, allowing slurry to communicate between the two chambers.
Air pressure control device	The device that automatically controls the air pressure in the plenum chamber to a predefined value, in a double chamber slurry TBM.
Single chamber slurry TBM	A single chamber slurry TBM has only one (the excavation) chamber in the forward section of the TBM. The pressure in the excavation chamber is controlled by adjusting the flow of slurry into and out of the chamber.
Feed line	The pipe by which fresh or treated slurry enters the excavation chamber.
Suction line	The pipe by which slurry, mixed with spoil, leaves the forward chambers.

3. SLURRY PRESSURE ASSESSMENT

Establishing, and then maintaining, the correct face support pressure (face pressure) for the ground and groundwater conditions is critical to the safe operation of a slurry TBM. If inadequate face pressure is applied, this will lead to excessive ground movement, and may result in collapse of the tunnel face. Instability leading to major loss of ground or collapse at the face of the tunnel is an Ultimate Limit State (ULS). Tunnelling in Hong Kong will generally require stringent control of the ground movements due to tunnelling, to minimise the effect of overlying and nearby buildings, structures and utilities. In this case, the Serviceability Limit State (SLS) has to be considered as well as the ULS.

Generally, slurry TBMs are selected for use in granular ground conditions and the minimum face pressure is calculated based on effective stress. However, where the tunnel is excavated in or just below cohesive materials, such as clay, it will be necessary to check whether total stress calculations govern. The maximum acceptable face pressure to prevent heave and blow-out will also have to be calculated.

The target face pressure should be calculated at intervals along the tunnel alignment.

The following are discussed below:

- (a) factors of safety,

- (b) effective stress calculations for the minimum face pressure,
- (c) total stress calculations for the minimum face pressure,
- (d) the effect of interfaces,
- (e) dealing with highly variable ground conditions,
- (f) assessing the maximum acceptable face pressure,
- (g) pressures using compressed air for head access,
- (h) adjustment to the target face pressures based on observation,
- (i) presentation and communication of target face pressures,
- (j) why the intended face pressure might not be applied in practice, and
- (k) some key issues for designers and design checkers.

3.1 Factor of Safety

Rather than using a global 'Factor of Safety', partial factors are applied to the parameters used in design, including the shear strength parameters of the ground (soil or rock), and the imposed loads, such as surcharge. Where reference is made to parameters, such as $\tan \phi'$, c' , s_u , or to the surcharge, q , in the rest of the report, the equations refer to factored values of the parameters/surcharge.

3.1.1 Partial Factors on Shear Strength Parameters

In deriving the factored shear strength parameters for ULS design calculations, the following partial factors are applied:

- on $\tan \phi'$: divide by 1.2
- on c' : divide by 1.2 (noting that c' is generally taken as zero for soil in these calculations)
- on s_u : divide by 1.5
- on unit weight: multiply by 1.0

For SLS calculations, a partial factor of 1.0 is applied to all of the soil parameters.

3.1.2 Partial Factors on Surcharge

For ULS calculations:

- When the surcharge is unfavourable (i.e. for inward yielding of the face): multiply by 1.5
- When the surcharge is favourable (i.e. for heave of the ground surface): multiply by 0

For SLS calculations, a partial factor of 1.0 is applied to the surcharge loads.

3.1.3 Partial Factor on Water Pressure

The water pressure to be used for the calculations should be the most onerous likely pressure at the level of the tunnel, based on a critical assessment by the geotechnical engineer of the available piezometric data and the groundwater flow regime interpreted from the piezometric measurements. Allowance should be made for seasonal or tidal variation, where appropriate. On this basis, a partial factor of 1.0 is applied to the water pressure.

Where the groundwater level is more than about one tunnel radius above the tunnel crown, the water pressure will generally be the dominant factor in the target face pressure calculated based on effective stresses. In these conditions, the practical effect of the partial factors applied to the shear strength parameters is usually very small, as a proportion of the total face pressure. The use of the partial factors will not compensate for poor assessment of the water pressure. It is essential to have as accurate an assessment of the water pressure at tunnel level as possible. Sufficient piezometers should be placed along the whole tunnel alignment, at appropriate elevations to establish the piezometric level at tunnel level with reasonable accuracy.

It is quite common to define the water pressure in relation to ground level. If this approach is adopted, care must be taken in areas where the ground level changes suddenly, for example at a cutting. Water pressure does not change as abruptly as ground levels can, particularly where the natural terrain has been modified.

One of the effects of tunnelling with slurry is that an excess pore pressure is created ahead of the face. The constant destruction and reforming of the filter cake during excavation leads to filtrate water being expelled into the ground. This excess pore pressure reduces effective stresses in the area of the face, and increases the support pressure required (Broere, 2005). The net effect on the face pressure required is small unless the tunnel is in a confined or semi-confined aquifer with a permeability of between 10^{-3} and 10^{-5} m/s. These conditions are most likely to occur in Hong Kong when tunnelling through beach or marine sands. Where these conditions are identified, reference can be made to Broere (2005) for the influence of this effect.

3.2 Effective Stress Calculations of Minimum Face Pressure

In general terms, the target (minimum) slurry pressure is calculated from the minimum face pressure required plus an allowance for variation, using the following equation:

$$P_{St} = \text{water pressure} + \text{pressure to balance effective pressure from soil and surcharge} + \text{allowance for variation in pressure} \dots\dots\dots (1)$$

The water pressure varies from crown to invert (Figure 5), so the minimum face pressure required to balance the water pressure also varies. The pressure calculation therefore needs to be specific to a particular level. Typically, calculations are either for a particular pressure sensor, or for the air pressure control device.

For soil pressure, two particular methods of calculating minimum face pressure using effective stress methods are summarized here: the method based on Anagnostou & Kovari (1996) and the method based on Proctor & White (1977). In this report, both of the methods in the original publications have been adapted for use in slurry TBM tunnelling. Only a brief summary of the application of these methods will be given here. The original publications should be consulted for the basis of the calculations.

The equations provided in this report are for the calculation of the pressure at sensor 1 (see Figure 1). The pressure at sensors 2 and 3, or for the air pressure control device, can be derived by substituting the appropriate terms for the respective sensor into the equations. In order to carry out the calculations, it is necessary to know the location of the sensors, relative to the axis or crown of the TBM, with reasonable accuracy (+/-200 mm). The position of the sensors should be shown, with dimensions, on the drawings giving the internal arrangement and key dimensions of the TBM. These drawings should be included as part of the design document setting out the basis for the calculation of the face pressures.

It is assumed here that the slurry is designed to form a filter cake in the ground conditions encountered, and that there is no significant penetration or loss of the slurry into the ground. It is also assumed that the ground is fully saturated. Whether or not an effective filter cake forms is dependent on a number of factors, in particular, the nature of the soil and the slurry.

It should be noted that the slurry properties can be affected adversely by contact with saline groundwater, cement or other chemicals. Examples would include:

- (a) cement grouts used for ground treatment, such as for jet grouting or deep soil mixing,
- (b) chemical grout used for ground treatment, see Jefferis (2003) for an example,
- (c) tail void grout, where this runs forward over the shield skin,
- (d) diaphragm walls or piles, where the machine has to break through them, and

- (e) chemicals used as part of the treatment process, such as flocculants, if these remain in the slurry after processing.

The assumption that the slurry forms an effective filter cake therefore requires that the appropriate material(s) are used for the slurry, that these are properly hydrated and mixed, and that the slurry is treated or replaced as needed to maintain the required properties.

Guidelines on the choice, mixing, quality control testing and treatment of slurry are given in AFTES (2005). In order to achieve effective support during slurry TBM tunnelling, it is necessary to ensure that appropriate materials are used, to carry out regular quality control for the slurry properties, and to ensure unobstructed flow of feed and return lines. Quality control testing and the measurement of slurry flow rate in the feed and suction lines should be specified. The detailed testing schedule and acceptable ranges for test results should be included in the work procedures. Compliance with the requirements should be checked by the site supervisory staff.

The calculations outlined below are based on a filter cake developing and forming a membrane on the tunnel face. If a highly permeable zone (such as a bed of coarse sand or gravel, soil pipes or other voids, or an old sea wall) is encountered in the tunnel, the slurry may penetrate into the ground, and the assumed filter cake may not be formed. Guidance on the effect of this on the required face pressure can be found in Anagnostou & Kovari (1996). Where a highly permeable zone, which could lead to loss of slurry, is identified in advance of tunnelling, grouting should be considered as a means of sealing the zone. The contractor should also provide, as part of the work procedure, a contingency plan for identifying and responding to any previously unidentified highly permeable zones encountered during the tunnelling. The work plan should include the maintenance of an adequate store of mixed slurry to compensate for losses. Please see also Section 3.6.2.

3.2.1 ULS Calculation - Method Based on Anagnostou & Kovari (1996)

In the case of a filter cake developing, the target slurry pressure at the crown of the tunnel is calculated, using the membrane model, from:

$$P_{St(Crown)} = \text{pressure due to water} + \text{pressure due to soil} + \text{pressure due to surcharge} + \text{allowance for variation in pressure} \dots \dots \dots (2)$$

The basis of the Anagnostou & Kovari method is a simple limit equilibrium calculation. The result of the equation provides the minimum pressure required to avoid face collapse (ULS condition).

The terms in Equation 2 can be evaluated as follows:

$$\text{The pressure due to water at crown} = (C - Z_w) \gamma_w \dots \dots \dots (3)$$

The pressure due to the soil = $F_0 \gamma' D - F_1 c'$, where F_0 and F_1 are factors derived from the charts in Anagnostou & Kovari, and depend on ϕ' and the ratio C/D . [Note: in the paper H is used as the symbol for cover over the tunnel, where C is used here]. This equation is a truncated version of the equation in their paper, as the membrane model is assumed for slurry

tunnelling, so $\Delta h = 0$.

The pressure due to surcharge at or near to the ground surface can be assessed using the charts in Atkinson & Mair (1977). However, for critical locations, such as tunnelling under buildings supported by piles, the pressure needed to control the foundation movement should be assessed by more detailed calculation. It may be necessary to use numerical analysis to take into account the interaction between the tunnelling and the piles.

The surcharge used as a basis for the calculation should be a realistic assessment of the actual ground surface load applied during tunnelling; it is not intended that design loads such as prescribed nominal H_A or H_B traffic loadings should be used in the calculation of target face pressures during tunnelling, as these are unlikely to reflect the actual loading condition at the time of tunnelling.

A typical allowance for the variation in pressure, v , is 0.2 bars (20 kPa), i.e. the actual face pressure may be higher or lower than the target face pressure applied by 0.2 bars. Consistently achieving a target face pressure within this tolerance requires a skilled operator, the correct machine configuration, and an appropriate slurry for the ground conditions. The value for the variation in pressure to be used in the calculations should be based on documented experience for the type of TBM proposed, and should be regularly reviewed during tunnelling.

The pressure applied varies depending on where on the face the pressure is measured, which depends on the vertical distance below the crown and the unit weight of the slurry. In order to achieve the required pressure at the crown of the tunnel, the target pressure at sensor 1, $P_{St(S1)}$ can be derived from:

$$P_{St(S1)} = P_{St(crown)} + (Z_{S1} - C) \gamma_{SL} \dots\dots\dots(4)$$

As the unit weight of the slurry is more than that of water, the critical point is at the crown of the tunnel. Hence the initial calculation is for the tunnel crown, and the target pressure at the sensor required to achieve the calculated pressure at the crown is then assessed.

The equation given above is for the target pressure. During tunnelling, the actual pressure should not be allowed to fall below P_S , where,

$$P_S(S1) = P_{St(S1)} - v \dots\dots\dots(5)$$

One assumption used by Anagnostou & Kovari in deriving their charts is that $\gamma_d/\gamma' = 1.6$. Since $\gamma_d/\gamma' = G_s/(G_s - 1)$, the assumption is that $G_s = 2.67$. This assumption is reasonable, within practical limits, for most saturated soils in Hong Kong.

The charts in Anagnostou & Kovari are derived from calculations using a limit equilibrium model proposed by Horn. An alternative to using the charts is to derive the necessary pressure directly, by carrying out calculations specific to the project. Broere (2001) provides guidance on such calculations.

3.2.2 SLS Calculation - Method Based on Proctor & White (1977)

There is limited basis to relate ground movements to face pressures in Hong Kong. A simple method is to adapt the analytical methods proposed by Proctor & White (1977). These were developed to derive the pressure against the walls of supported shafts or tunnels in various soils, and, by analogy, can be used to assess the pressure exerted by the soil at the face of the tunnel if only limited movement is allowed.

The calculation for the total face pressure is the same as that given in Equation 1, except that the slurry pressure to balance the soil pressure is given as $0.2 \gamma'D$ for dense sand to $0.6 \gamma'D$ for loose sand.

There is limited information on which to base the application of the method to soils in Hong Kong. The following are suggested, subject to review as experience is gained:

- (a) for saprolite with $\text{SPT-N} > 30$, use $0.25 \gamma'D$,
- (b) for saprolite or residual soil with $\text{SPT-N} < 30$ but > 10 , use $0.4 \gamma'D$, and
- (c) for granular superficial deposits with $\text{SPT-N} < 10$ use $0.55\gamma'D$.

Generally, the method based on Proctor & White gives a minimum face pressure slightly higher than that from Anagnostou & Kovari (1996) for dense saprolite, and significantly higher for loose sand.

The current (2009) experience is that a face pressure calculated from the method based on Proctor & White (1977) is the minimum necessary in granular soils to achieve a volume loss that is in the region of 1%. The face pressure is not the only factor that influences the volume loss over the tunnel; controlling the volume loss also requires effective grouting of the tail void. The actual minimum face pressure should be reviewed regularly and adjusted based on observation.

In determining the target face pressures, consideration needs to be given to the anticipated ground and groundwater conditions along the tunnel alignment. Particular concerns would be areas of high permeability, which could lead to slurry loss, or soils with a low shear strength (such as loose sand and soils rich in mica).

3.2.3 Assessing Pressures for Tunnels in Rock Using Effective Stress Methods

It is common for tunnels to be driven through stable rock without a face pressure. Where part of a slurry TBM drive is in a full face of stable rock, then the slurry system simply becomes a transport mechanism, as face pressure is not required for stability. Typically a minimum chamber pressure at crown of 0.8 to 1 bar is used to ensure that the excavation chamber is full of slurry. This reduces the risk that a fluctuating slurry level could result in damage to the suction pump.

If the rock is jointed and the TBM operating pressure is less than the water pressure, then seepage will develop towards the TBM. The effect of this seepage on pore pressures in compressible soils above the rock and any resulting consolidation settlement should be considered in setting the target face pressure.

In generally stable ground, there is a risk of encountering local features that require face pressure for stability. An example would be a fault zone in an otherwise stable rock mass. The zone around the fault may be further weakened by weathering extending from the fault. The contractor's work procedures should include methods for identifying and responding to the presence of such local features; see also 3.5.

Zones of highly fractured rock may be treated as a granular soil for the purposes of assessing the face pressure.

3.3 Total Stress Calculations of Minimum Face Pressure

For slurry TBM tunnelling, effective stress calculations generally govern the minimum face pressure. However, where the tunnelling is in clay, total stress calculations may govern. In Hong Kong, this would typically apply in cohesive marine deposits, and may apply in residual soils, and clayey colluvial or alluvial deposits. Where it is uncertain whether effective or total stress calculations are appropriate, the minimum face pressure required can be checked using both types of calculation, and the more onerous used for design. The basis for assessing the minimum face pressure, and settlement due to ground movement at the face of the tunnel, using total stress calculations, is given below.

3.3.1 Total Stress ULS Calculations

The minimum face pressure required to avoid collapse of the face of the tunnel can be calculated from:

$$P_S = (\gamma Z_O + q) - (s_u N_{TC}) \dots\dots\dots(6)$$

where N_{TC} is the stability number at collapse, and can be assessed from charts in Kimura & Mair (1981) (reproduced in O'Reilly (1988)). The value for N_{TC} varies based on the ratios C/D and P/D , P being the length of the unsupported heading as defined in Kimura & Mair. For the ULS case, P can be taken as zero. If there is sufficient movement, the ground will close around the TBM skin, so that the ground will be fully supported except at the face.

The target pressure is then:

$$P_{St} = P_S + v \dots\dots\dots(7)$$

This target pressure is the average pressure over the face, and can be taken as the pressure at the spring line of the tunnel. The target pressure at sensor 1 is then:

$$P_{St(S1)} = P_{St} - (Z_O - Z_{S1}) \gamma_{SL} \dots\dots\dots(8)$$

3.3.2 Total Stress SLS Calculations

The potential volume loss at the face and along the body of a slurry shield can be evaluated using the results of the model tunnel tests presented in a chart in Kimura & Mair (1981) (reproduced in O'Reilly (1988)). The chart relates volume loss to the Load Factor (LF), the ratio of N and N_{TC} :

$$LF = N / N_{TC} \dots\dots\dots (9)$$

where $N = (\gamma Z_O + q - P_S) / s_u$.

SLS calculations for slurry tunnelling should generally be based on $P = L$, the length of the shield. The slurry will flow into the overcut gap around the tail skin, and the face pressure will be transmitted to support the ground around the skin. The ground will thus be supported by the slurry pressure, rather than the shield skin, unless sufficient movement can be tolerated for the overcut gap to close.

The volume loss derived from the charts needs to be adjusted for slurry TBM tunnelling, as the calculated value would represent only the volume loss due to movement at the face and along the shield skin. Any volume loss at the tail of the shield (at the tail void) will be additive to the loss due to movement at the face and along the shield skin. The volume loss at the tail void will depend on the effectiveness of the grouting of the tail void around the tunnel lining (Shirlaw *et al.*, 2003).

It is unlikely that the ULS calculations will satisfy SLS requirements for an urban area, and, in consequence, a lower LF is likely to be needed. The allowable LF can be evaluated from the allowable volume loss: after subtracting an allowance for the volume loss at the tail void, the residual allowable volume loss due to movement at the face and along the shield skin can be established. Then, the allowable LF can be read off the chart in Kimura & Mair or calculated from the equation in Dimmock and Mair (2007), and the minimum face pressure required to achieve the allowable LF can be calculated from:

$$P_S = (\gamma Z_O + q) - (s_u N_{TC} LF) \dots\dots\dots (10)$$

P_{St} and $P_{St(S1)}$ can then be evaluated using Equations 7 and 8.

3.4 The Effect of Interfaces

The tunnel may pass through one or more interfaces between soils or rocks of very different nature. Examples are:

- (a) a palaeo-channel eroded into weathered granite, with the palaeo-channel infilled with superficial deposits,
- (b) the boundary between the rock and soil grades of weathered granite (e.g. a soil/rock interface or a corestone-bearing layer), and

- (c) in superficial deposits, there may be several beds in the face, with each bed having different geotechnical properties.

The equations given in Section 3.2 are for homogeneous ground conditions. The application of the equations in practice has to consider the effect of heterogeneous conditions. There are two simple ways of doing this:

- (a) where the tunnel is being driven in clay, and there are granular soils or significantly weaker clay above the tunnel, the cover over the tunnel (C) is taken as the cover of the stiffer clay. In this case the weaker soils are treated simply as a load (see Dimock & Mair (2007) for the application of this in London Clay), or
- (b) an assessment is carried out for each of the units present in or close to the face of the tunnel. The analysis is carried out as if the tunnel were in homogeneous conditions based on that unit, except for the overburden pressure, which is based on the assessed unit weight and thickness of each of the units over the tunnel. The face pressure selected is that required to control the weakest unit (the one requiring the highest face pressure). In practice, it is generally simple to identify which of the units present will be critical, and carry out calculations only for that unit.

In the second of these approaches, it is implicitly assumed that the minimum face pressure is controlled by the pressure required to support the weakest of the units in or close to the face, and that the pressure required to support that unit in mixed conditions is not greater than if the conditions were homogeneous. Sample calculations for two layers of contrasting strength in the tunnel face were presented by Broere (1998). The results of these calculations show that for those cases where the calculated minimum pressure with two units in the face is greater than for homogeneous conditions, the difference is small and can be ignored for practical purposes.

The effect of contrasting permeability between units on the minimum face pressure has to be considered in addition to the effects of differing strength. A confined or semi-confined aquifer in the face will require a higher support pressure than required in homogeneous conditions, as discussed in Section 3.1.3, and the pressures calculated in the second approach would need to be adjusted for this.

The methods outlined above for assessing tunnelling in heterogeneous ground conditions are highly simplified. Depending on the complexity of the ground conditions, the size and depth of the tunnel, and the environment over the tunnel, it may be appropriate to refine the initial calculations by more detailed analysis. This can be done by:

- (a) carrying out calculations based on the model proposed by Horn (1961). The charts in Anagnostou and Kovari (1996) are for homogeneous conditions, using the model proposed by Horn (1961). Heterogeneous conditions can be

assessed by calculation based on the original model. However, the Horn (1961) model is only appropriate for sands; for clay, the failure surface above the tunnel proposed by Horn (1961) is inappropriate.

- (b) using finite element analysis. Finite element analysis can be used to assess the face pressure in heterogeneous conditions, and can be used to calibrate the simpler analyses discussed above.

Where the minimum face pressure has to be higher in one unit than another along the tunnel drive, it is too late to wait until the weaker unit is encountered to raise the face pressure. The face pressure needs to be adjusted in advance of the interface, if the pressure needs to be raised, or after passing through the interface, if it can be reduced. As a rule of thumb, the pressure changes need to be managed such that the higher pressure required to support the weaker unit is applied while there is still at least one tunnel radius of cover of the stronger unit over the crown of the tunnel. Detailed calculations using numerical methods or the results of centrifuge modelling can be used to assess the minimum cover required in critical locations. Any uncertainty in the location of such interfaces needs to be considered in the planning of the target face pressures, and a conservative assumption made where there is uncertainty.

3.5 Dealing with Highly Variable Ground Conditions

In order to plan the target face pressures to be used during tunnelling, it is essential that sufficient site investigation is carried out to identify the ground and groundwater conditions along the tunnel route. However, even a comprehensive site investigation will not provide complete information on the conditions to be encountered during tunnelling. Therefore, in assessing the minimum face pressure, consideration has to be given to the variation in ground and groundwater conditions between investigation points.

The location of interfaces between units requiring a significantly different target face pressure must be assessed conservatively, recognising the uncertainty between investigation points. The lower face pressure required in the stronger unit should only be applied where there is a high degree of confidence that the tunnel will be in that unit, with sufficient cover to the weaker unit.

The soil shear strength parameters that are used in the face pressure calculations should be the lowest credible parameters for a particular section of tunnel, to cover the likely range of conditions that will be encountered. How far to divide the tunnel into different sections for the purpose of selecting the design parameters depends on the nature of the ground and the groundwater conditions, the level of investigation and judgment.

A slurry TBM may be excavating in generally stable ground conditions, but with local features that require face pressure for stability. An example would be a TBM operating in rock with local faults or deeply weathered seams. The possible presence of the faults or seams of soil grades may be inferred, but their location may not have been identified in the site investigation. In this case there are three options:

- (a) select the minimum face pressure based on the worst expected conditions, i.e. for the faulted or weathered zone(s) or soil,
- (b) make provision for identifying the location of the weaker zones along the tunnel by methods such as directional drilling prior to the commencement of tunnelling or probing from the TBM while it is still in good rock or strong soil, and adjust the minimum face pressure depending on the revealed conditions, or
- (c) select the minimum face pressure based on the good rock or strong soil conditions, and accept the risk of loss of ground in the event that a weaker zone is encountered - this would normally only be considered if the consequences of a loss of ground were acceptable to all stakeholders.

The selection of the first option would give the highest minimum face pressures, and therefore might appear to be the 'safest' option. However, the wear on a TBM increases with increasing face pressure. Increasing wear results in more frequent stoppages for head access to change cutting tools and repair the machine. Head access is one of the activities associated with a higher than normal risk of loss of ground. Excessive wear on the machine may also result in the machine malfunctioning, and being unable to exert the desired face pressure, when weak soils are encountered. Thus, grinding through rock or strong soils with an unnecessarily high face pressure may not reduce the overall level of risk.

3.6 Assessing the Maximum Acceptable Face Pressure

It is necessary to consider the maximum acceptable face pressure, as well as the minimum. Excessive face pressure can result in excessive heave, or loss of slurry to the ground surface (Plate 1). Heave of the ground surface is potentially more damaging to buildings than a similar magnitude of surface settlement. Loss of slurry to the surface may affect the safety of road users. Loss of slurry to the ground surface may also be followed by a loss of ground, as the loss of slurry causes a reduction in the face pressure. The face pressure to cause loss of slurry depends on whether the ground is intact, or whether there is an open path for the slurry to follow. Working in urban areas means that there is a chance of encountering ungrouted old boreholes, instrumentation, old water wells, areas where temporary works (such as sheet piles) have been removed, or other man-made open paths.

3.6.1 Maximum Pressure in Intact Ground

It is not unusual to have a small (< 5 mm) heave at the ground surface as the TBM face passes under a monitoring point; this level of heave is not the concern here. The concern is with heave that is sufficient to cause damage, or to rupture the overburden leading to loss of slurry and a reduction in the effective support pressure.

The pressure required to cause unacceptable ground heave will be the lowest of:

- (a) the pressure required to lift the block of ground over the tunnel - the reverse of the problem analysed by Anagnostou & Kovari,
- (b) the pressure required to initiate cavity expansion, and
- (c) the pressure required to initiate fracturing.

In each case, the pressure required is equal to the in-situ stress at tunnel level plus a value based on the strength of the ground mass. If the maximum face pressure is kept at or below the total vertical overburden pressure, there should be no risk of excessive heave or loss of slurry to the surface, in intact ground. For the pressure at sensor 1, this can be achieved if the target pressure is checked for:

$$P_{St(S1)} < C \gamma + (Z_{(S1)} - C) \gamma_{SL} - v \dots\dots\dots (11)$$

where $C \gamma$ is the total overburden pressure at tunnel crown, $(Z_{(S1)} - C) \gamma_{SL}$ is the difference in slurry pressure between the crown and the sensor, and v is the variation in pressure. In the equation, v is subtracted to derive the maximum target pressure.

The measured pressure, $P_{S(S1)}$, at sensor 1, any time, should not exceed:

$$P_{S(S1)} < C \gamma + (Z_{(S1)} - C) \gamma_{SL} \dots\dots\dots (12)$$

The check against total overburden pressure is simple, and in most cases will be sufficient to confirm that there will not be unacceptable heave at the maximum design face pressure. However, in some cases, such as relatively shallow tunnels in the superficial deposits, it may be found that the maximum design face pressure (maximum target pressure + v) exceeds the overburden pressure. In this event, a more detailed calculation can be carried out, allowing for the resistance provided by the shear strength of the ground. Where the shear strength is taken into account, the appropriate partial factors for shear strength parameters and surcharge should be used, as this represents an ULS.

3.6.2 Maximum Pressure with an Open Path to the Ground Surface

Where there is a pre-existing open path to the surface, the measured pressure at sensor 1 required to expel slurry to the surface is:

$$P_{S(S1)} = Z_{(S1)} \gamma_{SL} \dots\dots\dots (13)$$

The pressure to expel slurry up an open path is always significantly less than the overburden pressure, because of the low unit weight of the slurry compared with the unit weight of the soil.

Where the groundwater level is close to the ground surface, it will be found that the pressure required to expel slurry up an open path is generally less than the pressure required

to maintain face stability, in ground that is characterised by an effective angle of shearing resistance with little or no effective cohesion. It is essential that the face pressure is sufficient to maintain stability, so the risk of slurry loss if an open path is encountered has to be managed. This is generally done, in urban areas, by:

- (a) identifying any likely open paths and grouting them in advance of tunnelling, and
- (b) maintaining continuous surface watch during slurry TBM tunnelling, and implementing control measures if a loss of slurry is observed.

Both of these measures are normally implemented in urban areas because it is unlikely that all of the open paths can be identified from existing records.

Selecting a face pressure that is appropriate to the ground and groundwater conditions, but is not conservatively high, will minimize the slurry pressure, and help to limit the consequences of any loss of slurry.

3.7 Pressures Using Compressed Air for Head Access

Compressed air can be used to allow access into the forward chambers in unstable ground, for inspection and maintenance. Compressed air was used to allow open face tunnelling in Hong Kong in the 1970s and 1980s, particularly for tunnelling through saprolite and superficial deposits.

The pressure exerted by compressed air is constant over the exposed face (see Figure 6). With an exposed face of granular soil, the compressed air penetrated into the soil pores. The pressure of the compressed air minimised groundwater flow towards the face, but did not provide support to the effective stresses in the soil skeleton. For open face compressed air tunnelling in saprolite, the air pressure was typically set to balance the water pressure at a level about 1m above the base of the exposed face. For a full face this would mean that:

$$P_{CA} = [Z_O + (D/2) - (Z_w + 1)] \gamma_w \dots \dots \dots (14)$$

This resulted in the air pressure in the upper section of the face being higher than the water pressure, tending to dry out the soil in the upper section. CDG, when dry, will tend to ravel, and for long stoppages additional face support was provided using timbering or sprayed concrete. At the base of the exposed face the air pressure was slightly less than the groundwater pressure, leading to some seepage into the tunnel. The CDG was generally able to tolerate a very small seepage head, over a limited period, due to the residual cementation of the soil. The effect of the air pressure could be observed in the face, and the pressure was adjusted as necessary, based on those observations.

In sand with a low fines content, such as in the beach and marine deposits in Hong Kong, there were a number of issues with the use of compressed air. These included:

- (a) Instability of the dried out soil: the sand near the crown was dried out by the effect of the compressed air and became running ground.
- (b) Instability of the soil at the base of the exposed face: the superficial sand deposits are less stable under seepage forces than saprolite. Even the minimal net water pressure could cause some erosion of the sand at the base of the exposed face. This was exacerbated if the sand was a discrete lens. In this case the water could not be pushed away from the face by the compressed air. Instead, the compressed air pressure raised the water pressure, causing increased flow and erosion at the base of the lens (Figure 7).

The experience from the 1980s was for the use of compressed air in conjunction with open face shields or sprayed concrete lining. For slurry TBM tunnelling, the stability of the face can be improved by ensuring that a good filter cake is formed at the face prior to intervention. The filter cake will help by:

- (a) allowing the compressed air to support the soil particles, and not just push the water ahead of the face,
- (b) reducing the amount of air lost through the face, and
- (c) penetrating coarse sand (such as beach sand), thus creating a zone of bentonite impregnated sand at the face.

To ensure that a good filter cake is formed, the TBM head should be charged with fresh bentonite and the face pressure maintained at above the groundwater pressure for a period. Leaving the face in this condition will allow the fresh bentonite to penetrate into the soil (where the ground is coarse) and to form a filter cake, provided that the head of the machine is not turned when the cutting tools are in contact with the face. The combination of sand permeated with bentonite, a filter cake, and compressed air pressure will help to stabilise the soil while reducing loss of compressed air.

If an effective filter cake is formed, the compressed air pressure can be increased from the value given by Equation 14 until the support pressure is equal to that calculated for slurry mode tunnelling, after allowing for the different distribution of pressure over the face. While increasing the compressed air pressure will provide increased support, there are disadvantages in increasing the pressure. In particular, an increase in pressure will:

- (a) shorten the working time in the pressurised chamber; this can increase the total time for the intervention. The longer the intervention the greater the risk of a loss of ground,
- (b) increase the rate with which the filter cake dries out and starts to peel off the face, and
- (c) increase the overpressure at the crown of the tunnel,

reducing the factor of safety against a blow-out.

Setting the compressed air pressure involves a delicate balance between providing sufficient support, but minimising the adverse effects listed above.

The compressed air pressure needed will depend on the level to which the slurry is drawn down and the nature of the ground conditions in and over the face. For most interventions, the slurry level in the excavation chamber is not fully drawn down, but is maintained at about axis level. This level of slurry allows the cutting tools to be changed, as the head can be rotated to allow access to all of the tools. However, for major maintenance it can be necessary to completely empty the excavation chamber.

For planning purposes, it is suggested that:

In CDG, Equation 14 is used to assess the face pressure for full slurry drawdown, while Equation 15 (below) can be used for drawdown to axis level.

$$P_{CA} = [Z_O - (Z_w + 1)] \gamma_w \dots\dots\dots (15)$$

In superficial deposits, the compressed air pressure should be at least as high as for CDG. Provided that a good filter cake is formed, the compressed air should be checked for:

$$\begin{aligned} &\text{Average } P_{CA} \text{ over the area of the exposed face } \geq \\ &\text{Average } P_{St} \text{ over the area of the exposed face } \dots\dots\dots (16) \end{aligned}$$

and the higher of the face pressures from Equations 15 and 16 used. In Equation 16, P_{St} is calculated on the basis given in Sections 3.2 and 3.3 to satisfy ULS, using a pressure variation (v) of 0.1 bars. The ‘exposed face’ refers to the area of the face exposed to compressed air. In settlement sensitive areas, the need to satisfy SLS should be considered, but the benefit of increasing the air pressure, in terms of limiting settlement, may be outweighed by the risk factors discussed above.

The equations given above are for the planning of face pressures during interventions. The actual pressure used should be adjusted based on regular inspections, the behaviour observed during those inspections and monitoring data.

The air pressure is constant over the exposed height of the face, creating a significant overpressure at the crown of the tunnel. It is necessary to check that the pressure at the crown does not exceed the total overburden pressure. Otherwise, the ground may rupture, leading to a blow-out of the compressed air. This is a severe risk to the safety of those working in the compressed air, and to the public at the surface. In heterogeneous soils, the tunnel may be driven in a permeable soil (an aquifer) with a lower permeability soil (an aquitard) providing a cap above the tunnel. In this case the compressed air pressure can develop at the base of the aquitard, i.e. at a level higher than the crown of the tunnel.

For slurry TBM tunnelling, compressed air is typically applied only to the forward chambers, to allow head access. This is done where the machines are large enough to accommodate airlocks within the TBM, in which case the TBM pressure bulkhead is also used as the compressed air bulkhead.

For smaller machines, airlocks can be provided further back in the tunnel, so allowing part, or all, of the tunnel to be pressurised. In this case, the full length of the pressurised tunnel should be checked to ensure that the weight of the overburden at all points is greater than the compressed air pressure, to avoid the risk of a blow-out. Calculations to demonstrate that there is factor of safety of at least 1.1 against blow-out should be carried out at all locations where compressed air may be applied.

During a long intervention, the compressed air will dry out the filter cake, and the dried filter cake will start to peel off the face. Exposed ground can then ravel or run, depending on the nature of the ground. For such situations, provision should be made to refresh or replace the filter cake; alternative materials/products may be used to provide a seal at the face to prevent the compressed air from drying out the ground.

Interventions continue to be a major source of ground loss during TBM tunnelling. This risk, and some possible control measures, are discussed in Section 6 below.

3.8 Adjustment to the Target Face Pressures Based on Observations

The minimum and maximum design face pressures and the target face pressures should be reviewed regularly by the designer and the design checker, and where necessary they should be adjusted, as part of the overall management of the tunnelling. The work procedures developed by the contractor should include a clear process for the review and adjustment of the target face pressures, and the updating of documentation and staff instructions. The key personnel to be involved in this process will depend on the contractor's organisation. The work procedure should identify the minimum frequency of review, and identify the geotechnical and tunnelling professionals with the authority to adjust the minimum/maximum design face pressures and to make changes to the target face pressures.

In order to review the performance of the TBM, it is essential that the key operating parameters and the monitoring data are recorded, stored and available to those who need to access it. The key TBM operating parameters should be recorded in the on-board computer in real time. The work procedures should identify how the TBM operating data and the monitoring data are stored, and how access to the data is provided.

An example of a TBM target pressure review process is given below for reference, but this needs to be adapted to the particular circumstances on each project.

3.8.1 Adjustment of Target Face Pressures

A regular review of the target face pressures should be carried out. Typically the site and supervisory staff directly involved in the tunnelling will hold a daily meeting to review the progress, performance and monitoring data from the previous day, and to confirm the planned activities for that day.

Unless the designer and design checker attend the daily meeting, it is useful to hold a weekly meeting to assess whether the basis for the target face pressures needs to be adjusted.

This review meeting would typically be held every week, and confirm the target face pressures for the next one to two weeks. The review should consider any information on, *inter alia*:

- (a) deviations from the target face pressures,
- (b) any evidence of loss of ground, from the EMC records, monitoring or observation,
- (c) any deviation from the planned ranges of grout volume and pressure,
- (d) any incident of loss of slurry,
- (e) any new information on ground or groundwater conditions,
- (f) construction or excavation works in the vicinity of the tunnel alignment,
- (g) instrumentation readings,
- (h) observed ground behaviour in the face,
- (i) measured ground movements and the settlement of buildings, structures and utilities, and
- (k) the actual variations in the pressure applied.

Based on this review, it may be necessary to adjust the target face pressures.

3.8.2 Adjustment of Actual Face Pressures

The actual face pressure used should generally be within the target face pressure range. However, the operator will have to respond to observations during tunnelling, which may result in or require the use of a pressure outside of the range set. Examples of observations that could lead to such an adjustment include:

- (a) evidence of excessive excavation, based on the excavation management control system,
- (b) the slurry pressure during ring building, when the TBM is stationary, building up over the target value or dropping off below it,
- (c) monitoring data, such as excessive ground settlement, and
- (d) the escape of slurry at the surface.

The face pressure will need to be raised if there is evidence of face instability, over-excavation, excessive settlement or increasing slurry pressure, or may need to be reduced if there is loss of slurry. The necessary action will depend on the particular circumstances and will need to be implemented quickly, so the decision has to be devolved down to the shift engineer or TBM operator and the key site supervisory staff responsible for ensuring public safety. The key TBM management staff (such as the tunnel manager) and the key site supervisory staff at management level (such as the geotechnical professional responsible for risk management) should be available at all times, to review the actions needed and the adequacy of the actions taken in response to such observations, where there is significant risk to public and worker safety. The work procedure should identify critical observations, and the decision making process for responding to those observations.

3.9 Presentation and Communication of Target Face Pressures

The target face pressures should be communicated to various parties involved, including the designer and the design checker, the TBM operator and other tunnel staff. The means by which this is done should form part of the contractor's works procedures for the tunnelling. Examples of works procedures are given below for reference, but they need to be adapted to the particular circumstances on each project.

3.9.1 Presentation of Face Pressure Calculations Prior to Start of Tunnelling

The calculation of the planned minimum/maximum design face pressures and the target face pressures is carried out before the start of tunnelling. The calculations should be documented in a design report by the geotechnical professional (the designer) and checked by the design checker. The basis of the calculations should be provided, as well as the detailed calculations and assessed pressures. The methods outlined above can readily be set up on a spreadsheet. Typically calculations are carried out at intervals of 10 to 50 m along the tunnel, depending on the variability of the ground and groundwater conditions, and the rate of change in the depth of the tunnel. More closely spaced calculations or interpolation will be necessary where:

- (a) the tunnel is close to or at a major interface between different geological units or where there is shallow ground cover,
- (b) the ground level changes suddenly,
- (c) there are major underground structures (i.e. bored tunnels, underpasses or culverts) over or under the tunnel,
- (d) there are piles over the tunnel,
- (e) there is a significant (e.g. > 0.1 bar) difference in the calculated pressure between adjacent sets of calculation, and
- (f) at the break-in and break-out, and at planned intervention locations.

The minimum/maximum design face pressures and the target face pressures at intervals along the tunnel alignment should be summarised on drawings, in combination with the anticipated ground conditions and piezometric levels, the location of key facilities affected, instrumentation, and anticipated obstructions or open paths to the ground surface.

3.9.2 Presentation of Target Pressures for Regular Review During Tunnelling

The focus of the regular review of target face pressures is typically on the next 1 to 2 weeks of production. The detailed calculations are typically too large to be readily used for this purpose. It is useful to provide a simple summary sheet for the next 100 to 200 m of tunnelling, depending on the tunnelling rate and variability in ground conditions. The target face pressures, as well as the compressed air pressures to be used at intervention locations, can be summarised together with:

- (a) other key TBM operating parameters,
- (b) the anticipated ground and groundwater conditions, including the anticipated location of key interfaces,
- (c) the location of buildings, structures, tunnels, roads and utilities over or close to the tunnel,
- (d) the location of subsurface instrumentation and known boreholes and piezometers, and
- (e) the location of piles, known obstructions or wells on or close to the line of the tunnel.

The information for the immediate 1 to 2 weeks of production can typically be summarised on a single A4 or A3 sheet, for ease of use. This can provide a ready reference for planning of the work and when assessing the appropriate response to any problems during the tunnelling. The target pressures summarised in the sheet should be reviewed (see Section 3.8.1), and adjusted as necessary, based on the experience gained during tunnelling.

3.9.3 Communication of Target Face Pressures to the Operator and Other Tunnel Staff

The operators and other staff in the tunnel are typically provided with information for the next 1 to 2 days of production. The tunnel manager, or a key staff designated by the tunnel manager, provides to them a simple summary of the target face pressures together with other key operating parameters, such as maximum torque, grout volume and pressure, etc.

3.10 Why the Target Pressure Might Not be Applied in Practice

Even though the target face pressures are reasonably calculated and communicated to the tunnel crew, they may not always be correctly implemented in practice. This may arise from human or mechanical problems.

Human error could result in inadequate face pressure if, for example:

- (a) the target face pressures are not communicated to the TBM operator,
- (b) the target pressures are misread,
- (c) the operator deliberately reduces the pressure below the target pressure, for example to improve the speed of tunnel advance, and
- (d) the operator is not sufficiently experienced to maintain the face pressure within the minimum/maximum planned pressure range (the target pressure plus or minus the variation in pressure used in the design calculations).

Mechanical problems could result in inadequate (or excessive) slurry/compressed air pressure if, for example:

- (a) there is a blockage at the opening of the submerged wall, leading to no connection between the excavation and plenum chambers. The feed line pumps slurry into the excavation chamber and the suction line draws slurry from the plenum chamber. If the two chambers are not connected, the pressure will spike upwards. As the operator struggles to control the surge in pressure, the pressure in the excavation chamber can fluctuate by a large margin [measures to control this risk are given in Table 1],
- (b) there is a failure of the main pressure bulkhead due to excessive abrasion,
- (c) there is a blockage in the feed or the suction lines,
- (d) the pressure sensors malfunction, leading to incorrect information on the face pressure, or
- (e) the submerged wall between the excavation and plenum chambers develops a leak, such that compressed air enters the excavation chamber. [If this happens the pressure at the top of the excavation chamber will increase. The compressed air will also exacerbate the situation if an open path through the ground is encountered, as the system then works like an airlift on the slurry.]

The risk that the target face pressures are not used in practice is something that has to be considered in the risk assessment, and appropriate risk control measures should be established. The contractor's work procedures, organization, supervision and maintenance regime should be developed to reduce the risk to an acceptable level, reflecting the results of

the risk assessment. The action parties for implementing the risk control measures should be clearly identified in the drawings or work procedures, and adequate training and audits should be provided.

3.11 Some Key Issues for Designers and Design Checkers

Outlined above are some simple methods for establishing the minimum/maximum design face pressures and the target face pressures for slurry TBM tunnelling. The methods given are not the only ones available. There are viable, alternative approaches; the alternative approaches should provide similar, but not identical, values. Broere (2001) includes a summary of various methods that have been developed to assess the minimum face pressure, including those by Jancsecz & Steiner (1994) and Anagnostou & Kovari (1996). The important result is the proposed face pressure, not the means of establishing that value.

The equations given above are simple, and a spreadsheet can be set up quickly to derive appropriate limiting values for the face pressure. Carrying out a check using the methods given here is therefore simple. Providing the original calculations give reasonably similar values to the check calculations, there is no need to debate the respective merits of the basis of those calculations.

In practice, the face pressure is defined with reference to the ring number. It is essential to correctly relate the location of the face to the ring number being used as the basis for monitoring progress in the tunnel. The advance of a TBM tunnel is defined in terms of the ring number that is built at the end of that shove. The ring is built in the tailskin at the back of the shield, so the face during that shove is in advance, by slightly less than one TBM length, of the plan location of the ring being used as a reference. It is important to understand this, as the number of the ring to be built is also the basis for defining the target face pressure to the TBM operator. If the operator is told to use 2 bars of pressure during the shove for ring 134, the face at this point will not be at the location of ring 134, but at the location where, say, ring 138 or 139 (depending on the relative length of the ring and the TBM) will later be built (Figure 8). It is common to calculate the face pressures in relation to length along the tunnel drive or to a chainage based on a project datum, initially, and then translate this into ring numbers for use by the TBM operator. This translation is a common source of error, and needs to be checked carefully.

The assumptions that are used in deriving the face pressures need to be reviewed in detail, and be regularly revisited during tunnelling. Some questions that should be posed are:

- (a) is there enough ground investigation to define the ground conditions along the tunnel alignment, especially the location of interfaces and the potential presence of boulders or corestones?
- (b) is there enough piezometric data to allow the design piezometric level at tunnel level to be defined along the alignment?

- (c) has a review of records and site history been carried out for the presence of old boreholes, instrumentation, underground structures, utilities (including abandoned utilities) or weakened/open zones (such as occur due to the removal of temporary works in past projects)?
- (d) is the assessed variation (v) in the face pressure achievable in practice?
- (e) is the behaviour of the ground and the facilities affected (from instrumentation and monitoring) consistent with the calculations? For example, if the tunnel is driven close to a vibrating wire piezometer in soil, does the piezometric pressure rise as the face is driven past, reflecting the higher than hydrostatic pressures calculated for the target face pressure?
- (f) is there any performance data on slurry TBM tunnelling in similar ground and groundwater conditions, and is this consistent with what is being proposed?
- (g) is the quantity of excavated material calculated by the excavation management control system consistent with the face pressures used and settlements recorded?
- (h) are the results of the settlement monitoring above the tunnel within the predicted range?

4. TAIL VOID GROUTING, LIMITING PRESSURES AND VOLUMES

Tail void grouting is essential, both to minimize ground settlement due to tunnelling and to ensure the load on the tunnel lining is reasonably uniform. If the tail void grouting is not carried out properly, the load on the tunnel lining may be highly uneven. This can lead to distortion of the lining, cracking and, ultimately, collapse of the tunnel. The tail void grout must also quickly develop sufficient internal shear strength to stop the ring from floating upwards within the grout.

The purpose of the grouting is to completely fill the annulus around the lining with grout. The annulus is caused by the shield machine cutting a larger hole than the extrados of the lining. The size of the annulus may be further increased by, *inter alia*:

- (a) the use of gauge cutters (in rock) or hydraulically extendable overcutters (in soil),
- (b) any vertical or horizontal curves that have to be negotiated,
- (c) the inclination of the shield machine relative to the direction of tunnelling, and

(d) any loss of ground at the tunnel face.

Where the ground is unstable without a support pressure, the tail void will close rapidly unless a support pressure is provided. Slurry TBM tunnelling will generally involve such ground conditions. In order to provide a continuous support pressure at the tail void, it is necessary to grout continuously during the advance of the TBM. This can be done by providing grout pipes which are laid along the tail skin and through the tail seals. Such grout pipes allow the injection of the grout simultaneous with TBM advance, commonly known as 'simultaneous tail void grouting'.

The effective volume of the grout injected behind the lining is also affected by losses in the system and by bleeding of the grout. It is therefore normal to inject more grout than the minimum theoretical volume of the tail void (annulus).

It is normal to define, for the tail void grouting: a minimum volume, a minimum pressure and a maximum pressure. The work procedure should also state the means of injection, the grout mix, testing to confirm the properties of the grout and methods of checking that the grout is spreading around the ring and filling the full tail void. Examples of grout mixes, testing and injection methods are discussed in Shirlaw *et al.* (2004).

The theoretical minimum void can be calculated. This is typically increased by at least 20% to arrive at the minimum volume to be injected.

The injection pressure to be used has to be greater than the groundwater pressure, and has to be sufficient to move the grout around the annulus around the ring. It also has to be greater than the slurry pressure in the head of the machine, as the slurry can connect to the tail void by passing around the machine through the overcut. Typically, the minimum grout pressure is set at 1 to 2 bars over the target slurry pressure. For shallow tunnels, or for tunnels in soft clay, this may equal or exceed the overburden pressure at the crown.

The maximum grout pressure should be calculated, on the basis of avoiding excessive heave of the ground surface. The pressure to heave the ground surface is the full overburden pressure plus the pressure needed to overcome the shear strength of the ground. For shallow tunnels or for tunnels in soft clay, the minimum pressure to ensure effective grouting may be greater than the pressure required to heave the ground surface. Where the two criteria are in conflict, the critical criterion is to apply sufficient pressure to ensure effective grouting of the ring. Uneven loading due to inadequate grouting of the tail void risks the stability of the tunnel. The consequences of tunnel instability are generally much more severe than a limited amount of heave, but this needs to be reviewed on a case by case basis.

Effective tail void grouting is one factor in ensuring that settlement due to tunnelling is minimised. Other potential sources of settlement include movement at the face (discussed in Section 3), movement into the gap around the machine caused by overcutting and negotiating curves, and consolidation.

Tail seals should be incorporated into the tail skin, to prevent grout, water or soil particles from entering the TBM via the tail void. A typical tail seal consists of multiple rows of wire brushes, with grease injected under pressure into the gaps between the wire brushes. The pressure of the tail seal grease needs to be kept at or above the grout pressure.

A lower pressure risks damage to the tail seals, which in turn risks loss of ground through the damaged tail seals.

To control movements into the overcut gap around an EPB shield, Leblais *et al.* (1999) suggest injecting bentonite around the shield skin. This is not normally adopted with slurry shields as the overcut is generally in connection with the face, and filled by slurry from this source.

Consolidation settlements can be caused by drainage of compressible soils, or by the dissipation of excess pore pressures generated during tunnelling. Positive excess pore pressures can be caused by excessive grouting, or by grouting into the tail void after the ground has already moved down onto the ring. Poorly planned or executed grouting can therefore be a factor in the development of consolidation settlements over the tunnel (Shirlaw *et al.* 1994).

5. EXCAVATION MANAGEMENT CONTROL SYSTEM (VOLUME ASSESSMENT)

By measuring the flow rate and the density of the slurry, the net dry weight of the solids removed during excavation is calculated, and compared with theoretical values. This provides the basis for assessing whether there has been significant over-excavation at the face of the machine. The measurement is done by placing a flow meter (Plate 2) and a density meter on both the feed and suction lines. The dry weight of the solids pumped into the tunnel is subtracted from the dry weight of the solids pumped out. The net dry weight of material removed is then compared with the theoretical dry weight, to determine if there has been potential over- or under-excavation. While, in principle, this is simple, there are a number of practical issues that must be considered, including the accuracy of the volume measurement and the assessment of the theoretical dry weight of the material excavated.

5.1 Accuracy of the Volume Measurement

The flow rate and density of the slurry have to be measured both on the feed and suction lines. Flow (see Plate 2) and density meters are therefore required on both lines. The location of these meters is critical to the accuracy of the readings. The manufacturer's recommendations on location should be followed. Both types of meter also need to be regularly calibrated.

However, even with the correct installation, the accuracy of current flow meters is typically about $\pm 2\%$. This can introduce significant variation in the measurement of the volume of material pumped, and therefore in the 'measured' net weight of solids pumped out. The longer the excavation cycle, the greater the potential error becomes.

The measurement of volume can also be affected by gains and losses to the system. Gains can come from groundwater entering the excavation chamber; this should only happen if the slurry pressure is lower than the groundwater pressure. Losses can occur due to loss of slurry into the ground or to the surface; maintenance work, head access and breakage of the pipes through accident or abrasion can also lead to loss of slurry from the circuit.

5.2 Assessment of the Theoretical Dry Weight of the Material Excavated

There is a significant difference in the dry weight of 1 m³ of soil and 1 m³ of rock. This introduces a particular problem for tunnelling through weathered rock. In partially weathered, heterogeneous rock masses, the tunnel will pass through a highly variable mixture of rock and soil grades of weathered rock. Nakano *et al.* (2007) discuss this issue for tunnelling through weathered granite in Singapore. They give the example of the dry weight of the solids from Grade III granite being nearly 50% higher, for each cubic meter excavated, than Grade VI granite (Residual Soil). During tunnelling in heterogeneous weathered rock masses, the relative proportions of rock and soil grades in the face will be constantly varying, and so the dry weight of the excavated material per ring will also vary significantly. Nakano *et al.* suggest that a subjective assessment can be made by watching the change in the colour of the solid material discharged at the slurry treatment plant. However, this assessment will be qualitative, not quantitative.

5.3 Use of the Information from the Excavation Management Control System

The information from the Excavation Management Control (EMC) system is important in assessing the degree of ground control being achieved in slurry tunnelling. The guidelines for best practice with closed face tunnelling machines (BTS/ICE 2005) list the use of the EMC system in the conclusions and recommendations section, while also noting some of the potential sources of inaccuracy in measurement. Despite the limitations given above, the excavation management control system is a primary warning that significant over-excavation may be occurring, and is an essential part of slurry TBM tunnelling, particularly in an urban environment. By plotting the cumulative dry weight of material excavated against the theoretical weight, during the excavation of each ring, any trend of significant deviation from the theoretical value can be identified. Where possible significant over-excavation is observed, control measures should be implemented. Control measures typically include:

- (a) raising the face pressure (but not such as to induce excessive heave),
- (b) assessing whether it is necessary to grout, to fill the potential void due to over-excavation, and
- (c) implementing the appropriate contingency measures, as identified in the Risk Management Plan, if a potential void (due to over-excavation) is suspected. An example of such contingency measures would be to temporarily close the part of a road over the suspected cavity, for safety of the road users.

Because of the potential limitations of the system:

- (a) the results are typically considered on a rolling average over 5 or 10 rings, as well as for each individual ring; a deviation of less than 20% in excavated volume for a single ring may be due to errors in measurement, or over-excavation, or a

combination of the two, and

- (b) the results of the EMC are considered in conjunction with the actual face pressures used, the variations in the face pressure, the geological and hydrogeological conditions, the specific observations on the nature of the materials observed at the slurry treatment plant, and the instrumentation and monitoring results and other observations, before deciding whether significant over-excavation has in fact taken place.

Systems are available for real-time remote access of the TBM parameters, face pressures and grout volumes. Continual assessment of these parameters, together with the plots of deviations from the theoretical dry weight of material excavated, can indicate when over-excavation is occurring, and allow the appropriate control measures to be taken expeditiously.

As stated above, the results of the settlement monitoring should be considered in the assessment. It is sometimes possible to detect the early stages of excessive ground movement, caused by over-excavation, by frequent monitoring. However, by the time the excessive ground movement has migrated to, or close to, the ground surface, there is little time to take action to mitigate the effects. Major over-excavation typically appears at surface as a local sinkhole, and is unlikely to be captured by the settlement monitoring, as any instrument fortuitously located at the sinkhole will become unreadable.

6. HIGH RISK ACTIVITIES (BREAK-IN, BREAK-OUT, INTERVENTIONS, INTERFACES, WEAR, FLUSHING)

Based on experience with EPB tunnelling in Singapore, Shirlaw *et al.* (2003) identified the following activities where there was a higher than normal risk of loss of ground:

- (a) Break-in to the tunnel from the TBM access shaft (launch of the shield).
- (b) Break-out from the tunnel to the TBM recovery shaft (recovery of the shield).
- (c) Interfaces between different strata.
- (d) Mixed face conditions, consisting of rock and soil (or rock and soil grades of weathered rock).
- (e) Head access for maintenance.
- (f) Long drives in abrasive ground, leading to wear of the machine.

For slurry shields, these six high risk areas also apply, but with the addition of a seventh:

- (g) Extended periods of slurry circulation, without shield advance, to clear blockages of the cutter head, at the opening between the excavation and plenum chambers or in the suction line.

The appropriate mitigation measures for these activities should be identified in the risk assessment carried out for the project. As an aid to reviewing the risk assessment, reference can be made to Table 1, which lists the high risk activities and a number of possible mitigation measures. The table was based on Shirlaw *et al.* (2005), and was originally developed for EPB tunnelling. The table has been revised and adapted for slurry TBM tunnelling. The mitigation measures listed in Table 1 are not intended to be applied without a critical consideration of the site-specific circumstances. There are alternative methods for mitigating risk that may be applicable, depending on the circumstances. The actual mitigation measures to be applied should be developed on a project specific basis.

During tunnelling through mixed ground conditions, boulders or rock fragments may be removed from the ground. If these are too large to pass through the cutterhead openings, they will be rotated with the TBM cutterhead until broken up. This may cause overbreak. If a boulder/rock fragment is trapped in the cutterhead (Plate 4), the part projecting ahead of the TBM will disturb the excavated face. Whether a boulder/rock fragment is plucked from the face by the cutting tools depends on a number of factors, including the size of the boulder or the joint spacing in the rock mass, the strength of the matrix soil or rock joints, the forces applied by the TBM and its rate of advance, and the type and condition of the cutting tools. Applying the correct chamber pressure will help to minimise the likelihood of this occurring, by preventing seepage flow towards the face. Seepage will tend to reduce the strength of the soil matrix. Interventions may be necessary to remove trapped boulders/rock fragments if these are causing significant damage to the cutting tools or overbreak. Regular interventions for the inspection and maintenance of the cutting tools will help to ensure that the boulders/rock fragments are broken up into small pieces and extracted by the TBM rather than plucked out of the face.

Access to the head of the machine is a risk for both loss of ground and the safety of those accessing the head. One method of reducing the risk to safety of personnel is to temporarily close some or all of the openings in the cutterhead. Temporary closure of the openings may also help to control loss of ground during the intervention. However, if the cutterhead has been retracted to allow the tools to be changed, significant volume loss can still occur if the ground collapses onto the cutterhead.

Prior to a compressed air intervention, the slurry has to be replaced by compressed air. At the end of the intervention, the compressed air has to be replaced by slurry. Controlling the face pressure to the required value, during these changes, requires skill and experience.

The risk posed by interventions can be reduced by minimising the number of interventions necessary in particularly settlement sensitive areas, such as when tunnelling directly under buildings or other tunnels. The cutterhead should be inspected and all necessary tool changes and maintenance carried out just before entering the settlement sensitive area. This precaution should reduce the need for interventions in the settlement sensitive areas. The locations for precautionary interventions can be planned in advance of tunnelling, and reviewed and, where necessary, adjusted during tunnelling. Other than the

precautionary interventions, it is not possible to plan the location of most interventions in advance, as the tunnel crew will have to respond to any indications that maintenance is required due to wear or damage. However, general planning for face interventions should be carried out, as far as practicable, prior to the start of tunnelling, including face pressures, work procedures, precautionary measures and contingency plans. The face pressure if an intervention is required can be calculated at the same intervals as the face pressure for slurry tunnelling, so that there is a default value available if an intervention is required. The contractor's work procedures should identify the level of authorisation required in the contractor's organisation, and where necessary, by the Engineer's site supervisory staff, before undertaking each intervention. In settlement sensitive areas this authorisation should come from the senior management personnel responsible for risk management. Where tunnelling is continuously in settlement sensitive areas for long distances, risk can be reduced by preparing 'safe refuges', to allow major maintenance and repair of the TBM. The 'safe refuges' typically consist of a block of grouted or otherwise treated ground, providing improved face stability during head access. The head of the TBM is driven into the 'safe refuge' for a major intervention.

At the beginning of the intervention, the excavated face should be inspected, to check that the ground and other conditions are safe for intervention using the planned compressed air pressure (if any) and work procedures. The contractor's work procedures should identify the key staff or grade of staff, who must be suitably experienced to undertake this inspection. Unless the face is in stable rock or treated ground, the inspection should be made in compressed air, and the air pressure should only be reduced if it is confirmed from the inspection, by a competent professional who is qualified and experienced to assess the risk of ground instability, that it is safe to do so. The initial inspection should be made before entering the excavation chamber, by looking through the access door, so that the door can be closed quickly if the conditions are unsafe. After the initial assessment, a more detailed assessment can be made from within the excavation chamber.

Table 1 provides examples of possible mitigation measures which can be considered to reduce the likelihood of loss of ground during slurry TBM tunnelling. Another means to reduce risk is to mitigate the consequences in the event of a loss of ground. In Table 2, a number of possible mitigation measures are listed. Most of these measures have been used in practice, either on their own or in combination with the measures given in Table 1. The measures in Table 2 should only be applied where appropriate, based on the risk assessment for the particular project and in consultation with the stakeholders. The list in Table 2 is not exhaustive, and other mitigation measures not listed in the table may be appropriate in particular circumstances.

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Table 1 - High Risk Activities and Some Possible Mitigation Measures Which Can be Considered for Reducing the Likelihood of Loss of Ground During Slurry TBM Tunnelling (Sheet 1 of 3)

High Risk Activity	Possible Mitigation Measures
Shield launch	<ul style="list-style-type: none"> • Where possible, select site location with low risk for the launch • Provide a seal between shield and wall of launch shaft (see Plate 3) • Test the seal with pressurised slurry prior to launch beyond stable ground • Provide minimum length of ground treatment - typically length of shield plus the length of two rings. This measure does not need to be implemented if it can be demonstrated by investigation, probing and calculation that the ground for this section will be stable during tunnelling, even without a support pressure • Provide detailed plan for how face pressures will be built up to general operating level • When launching through a 'soft tunnel eye' created using steel fibres or carbon fibre reinforcement, ensure suction line is free of any blockages while the head is still within the treated zone • Where Tubes-a-Manchette (TaMs) are used for grouting, ensure that the material used for the tubes is brittle and breaks up when cut by the head, to reduce the risk of blockage of the head or suction line
Shield recovery	<ul style="list-style-type: none"> • Where possible, select site location with low risk for the recovery • Provide a seal between shield and wall of recovery shaft • Provide minimum length of ground treatment - typically length of shield plus the length of one ring. This measure does not need to be implemented if it can be demonstrated by investigation, probing and calculation that the ground for this section will be stable during tunnelling, even without a support pressure • Provide detailed plan for how face pressures will be reduced from general operating level to zero • Where TaMs are used for grouting, ensure that the material used for the tubes is brittle and breaks up when cut by the head, to reduce the risk of blockage of the head or suction line
Interfaces between stable & unstable geological units	<ul style="list-style-type: none"> • Provide detailed planning of slurry pressures approaching, through and after the interface, where identified • Where necessary, grout the ground around the tunnel at the interface. While grouting is an option, this has to be considered cautiously, as the effect of the grouting may be to just move the location of the interface from one place to another • Reductions in face pressure should only be made where there is a high degree of confidence that there is sufficient cover of stable ground. Additional investigation, directional drilling or probing from the tunnel can be used to provide more detailed information on the location of the interface. However, probing carries a risk of making a connection between the TBM and permeable strata, so it is generally preferred to use a conservative face pressure (i.e. higher than the hydrostatic pressure) where there is a risk of such an interface close to the tunnel, rather than probing

Table 1 - High Risk Activities and Some Possible Mitigation Measures Which Can be Considered for Reducing the Likelihood of Loss of Ground During Slurry TBM Tunnelling (Sheet 2 of 3)

High Risk Activity	Possible Mitigation Measures
Mixed face conditions	<ul style="list-style-type: none"> • Provide detailed planning of slurry pressures approaching, through and after soil/rock interfaces, where identified • Where necessary, grout the ground around the tunnel at the interface • Assess the interface conservatively. There has to be a high level of confidence that the tunnel is in, and has sufficient cover of rock, to operate the TBM at a face pressure lower than that required for the overlying soil. See comments above on the use of investigation, directional drilling or probing to provide additional information • Reduce the rate of advance and/or speed of cutterhead rotation. This can reduce impact damage to the cutting tools and the risk of plucking boulders/rock fragments from the face. However, this needs to be balanced against the increased risk, with a slower advance rate, that any overbreak will migrate before it can be filled by the tail void grout • Regularly inspect and maintain the cutting tools. This is normal good practice, but the frequency of inspection should be much higher in mixed soil/rock conditions than in uniform conditions, due to the higher rate of wear and damage • Provide shrouds to protect the bearings of the discs from impact damage or clogging with soil
Chamber/head access (intervention)	<ul style="list-style-type: none"> • Minimise the need for interventions in critical areas, such as under buildings. Inspect and maintain TBM as necessary just before entering critical areas • Form 'safe havens', by grouting the ground around the tunnel, to allow regular inspection/maintenance in critical areas • Define level of authorisation required before intervention in a critical area • Give detailed work procedures for access, including details of how the slurry pressure will be replaced by compressed air pressure, and vice versa • Provide compressed air locks. Compressed air access to be available within 72 hours at all times • Prior to start of tunnelling, provide planned compressed air pressure along the full tunnel alignment, in case of need for emergency access • Provide service penetrations through pressure bulkhead to avoid running lines (i.e. compressed air for hand tools, water, electricity) through air locks • Ensure any soil adhering to door through main pressure bulkhead is cleaned off at the start of intervention – so that it can be closed rapidly and securely in the event of instability. • Provide external seal around tailskin (between skin and rock/soil) and/or inject polyurethane grout around skin prior to entry, to minimise water flow along shield towards chamber • Provide means to seal off head openings during intervention • Carry out inspection of the exposed face at the start of the intervention, and at regular intervals during the intervention • Ensure that a fresh filter cake is formed prior to the start of the intervention, and that this is renewed or replaced when necessary

Table 1 - High Risk Activities and Some Possible Mitigation Measures Which Can be Considered for Reducing the Likelihood of Loss of Ground During Slurry TBM Tunnelling (Sheet 3 of 3)

High Risk Activity	Possible Mitigation Measures
Mechanical problems	<ul style="list-style-type: none">• Regularly inspect and maintain the mechanical parts• Provide hardening/wear plates for exposed areas of machine and other key mechanical parts, and have ready essential equipment for repair/replacement
Extended flushing without advance	<ul style="list-style-type: none">• Restrict flushing to no more than 10 minutes at a time, with review by an experienced engineer at the end of period to check that the flushing is not causing excessive loss of ground at the face• For double chamber slurry TBMs: provide pressure-balancing pipes between the excavation and plenum chambers. These can reduce the effect of blockages at the submerged wall, by reducing the spikes in pressure this causes. For single chamber slurry TBMs, pressure relief pipe(s) can be provided through the pressure bulkhead; these are designed to bleed slurry into the unpressurised part of the shield if the face pressure becomes excessive

Table 2 - Some Possible Mitigation Measures Which May be Considered for Controlling the Consequences of Excessive Loss of Ground During Slurry TBM Tunnelling

Purpose	Possible Mitigation Measures
Intercepting the ground loss	<ul style="list-style-type: none">• Grouting the cavity caused by the ground loss before it reaches the surface; this will work only where it is possible to detect the cavity quickly and there is enough time for the grouting to be carried out before collapse of the cavity. This measure is not suitable for soft clay or granular soils under the water table, where the cavity will likely migrate to the surface within a very short time
Mitigating the effects - buildings	<ul style="list-style-type: none">• Temporary evacuation as the TBM passes beneath the building• Temporary propping of parts of the building• Strengthening the foundations by underpinning or forming a raft under the building or by compensation grouting• Strengthening the building to span over a cavity
Mitigating the effects - roads	<ul style="list-style-type: none">• Temporary, localised, road closure above the tunnel• Construct thickened road slab to bridge over any cavity that might develop; ducts should be provided through the thickened road slab to allow any cavities that develop to be filled
Mitigating the effects - utilities	<ul style="list-style-type: none">• For water, sewage and gas pipes, ensure that there are valves to close off the utility if necessary, and that this can be done at short notice• Shut down the utility temporarily as the TBM passes beneath it• Construct a utility bridge to support the utility to cope with the event of a cavity forming

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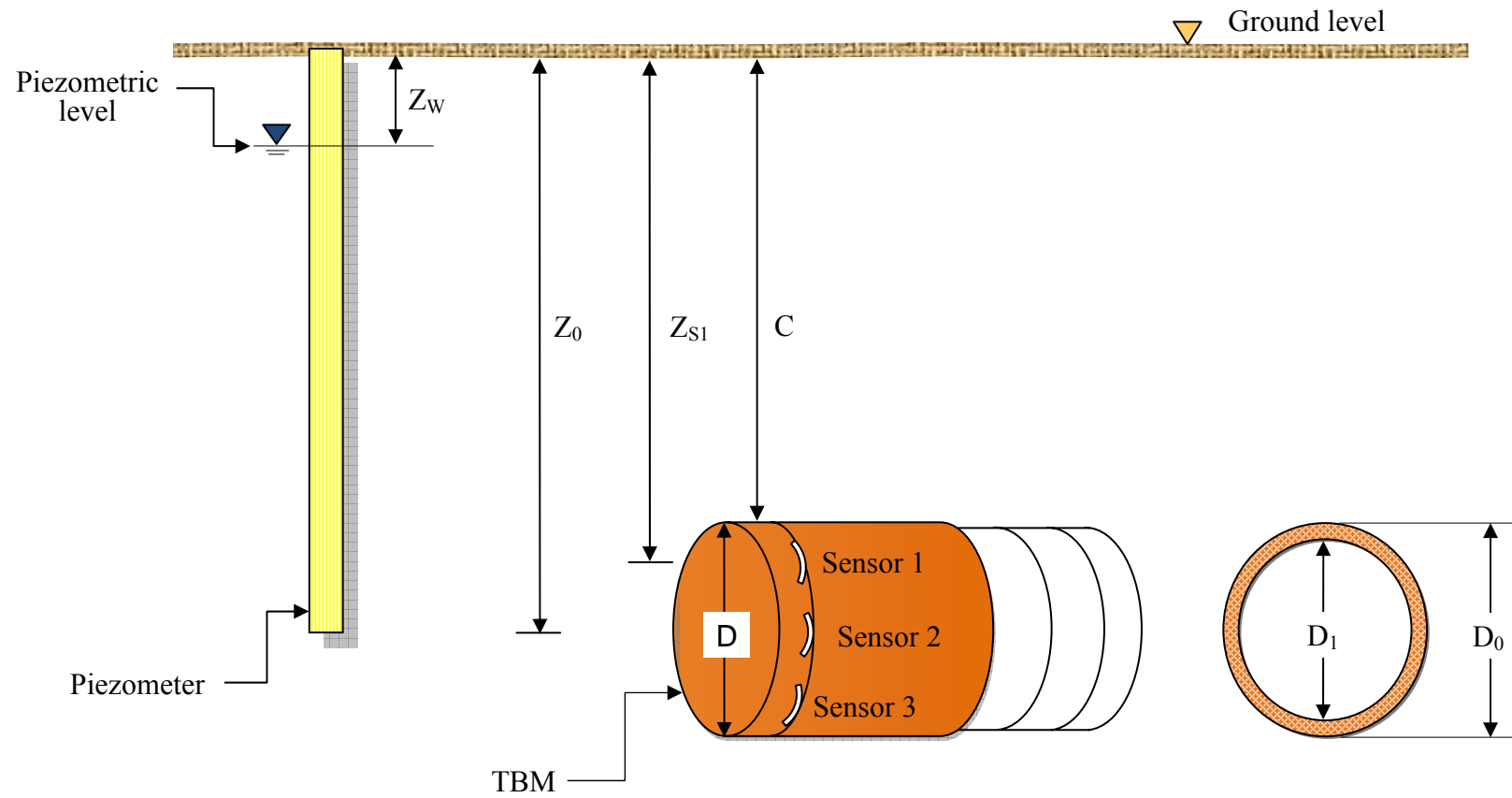


Figure 1 - Dimensions

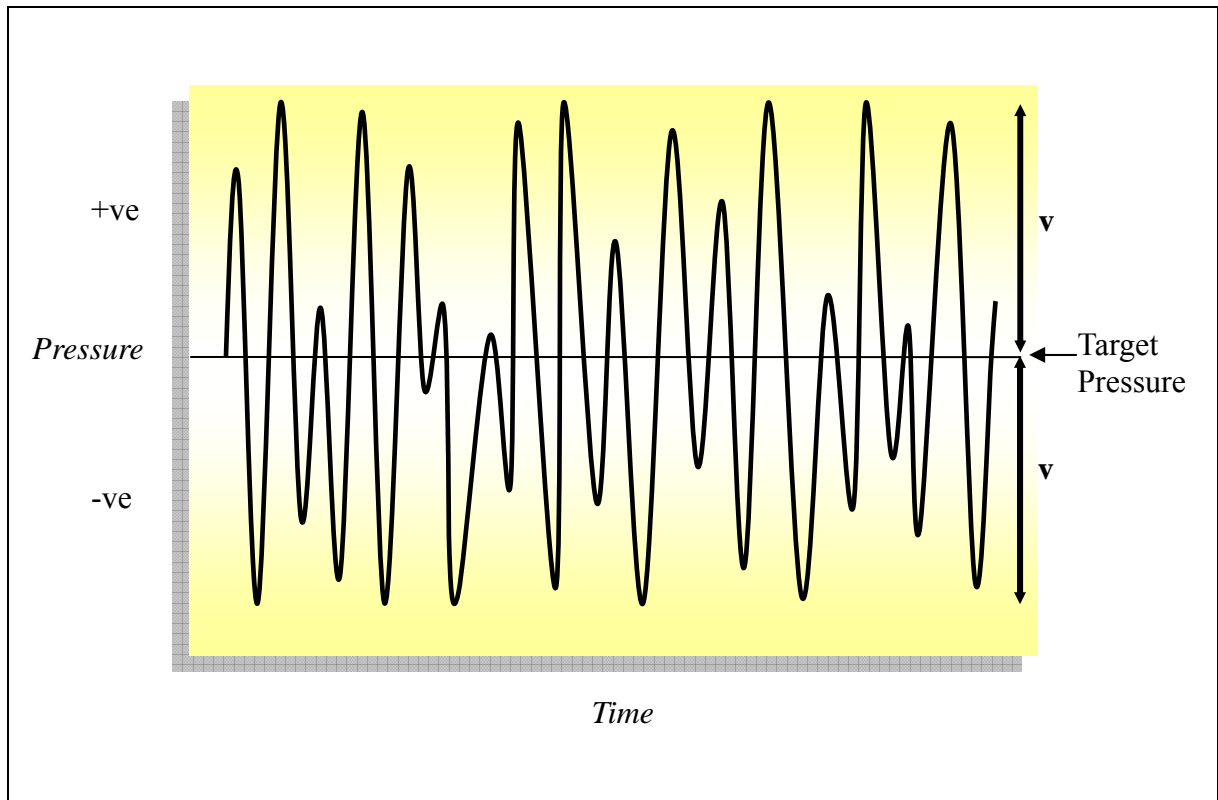


Figure 2 - Fluctuation of Face Pressure

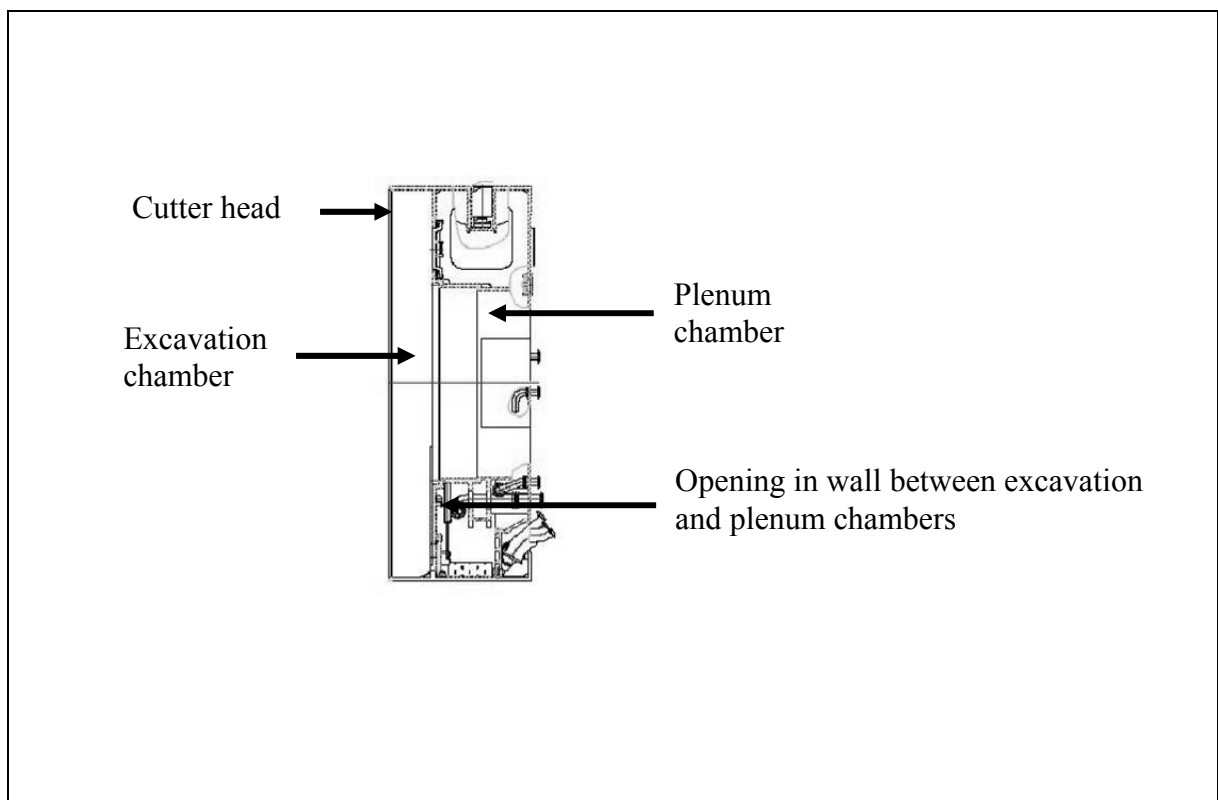


Figure 3 - Schematic of a Double Chamber TBM Showing the Two Chambers

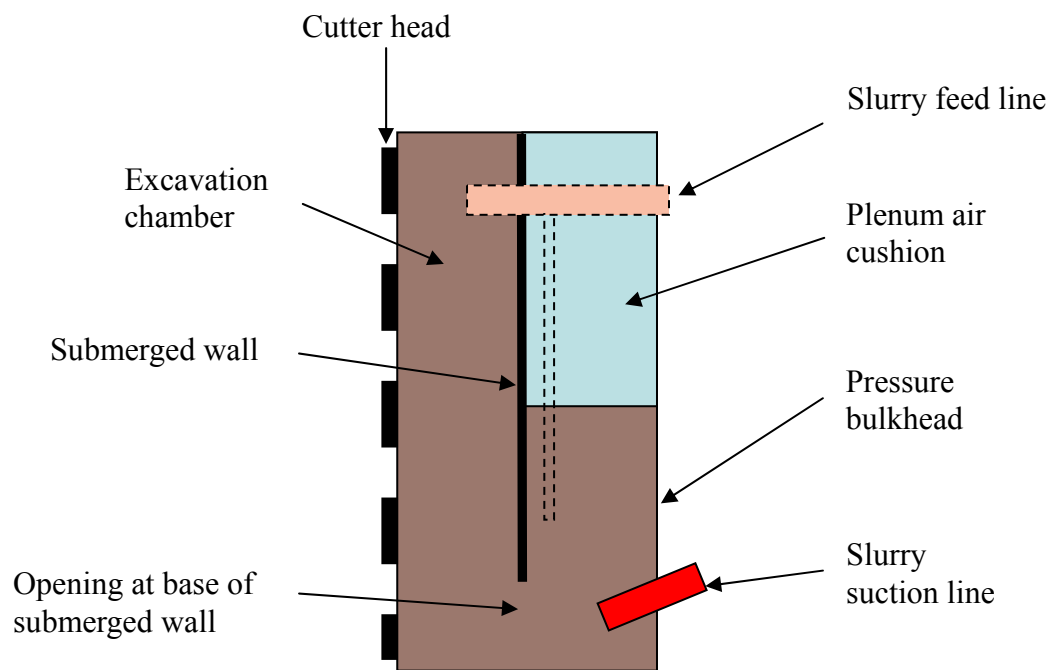


Figure 4 - Schematic of a Double Chamber TBM Showing Some of the Major Features

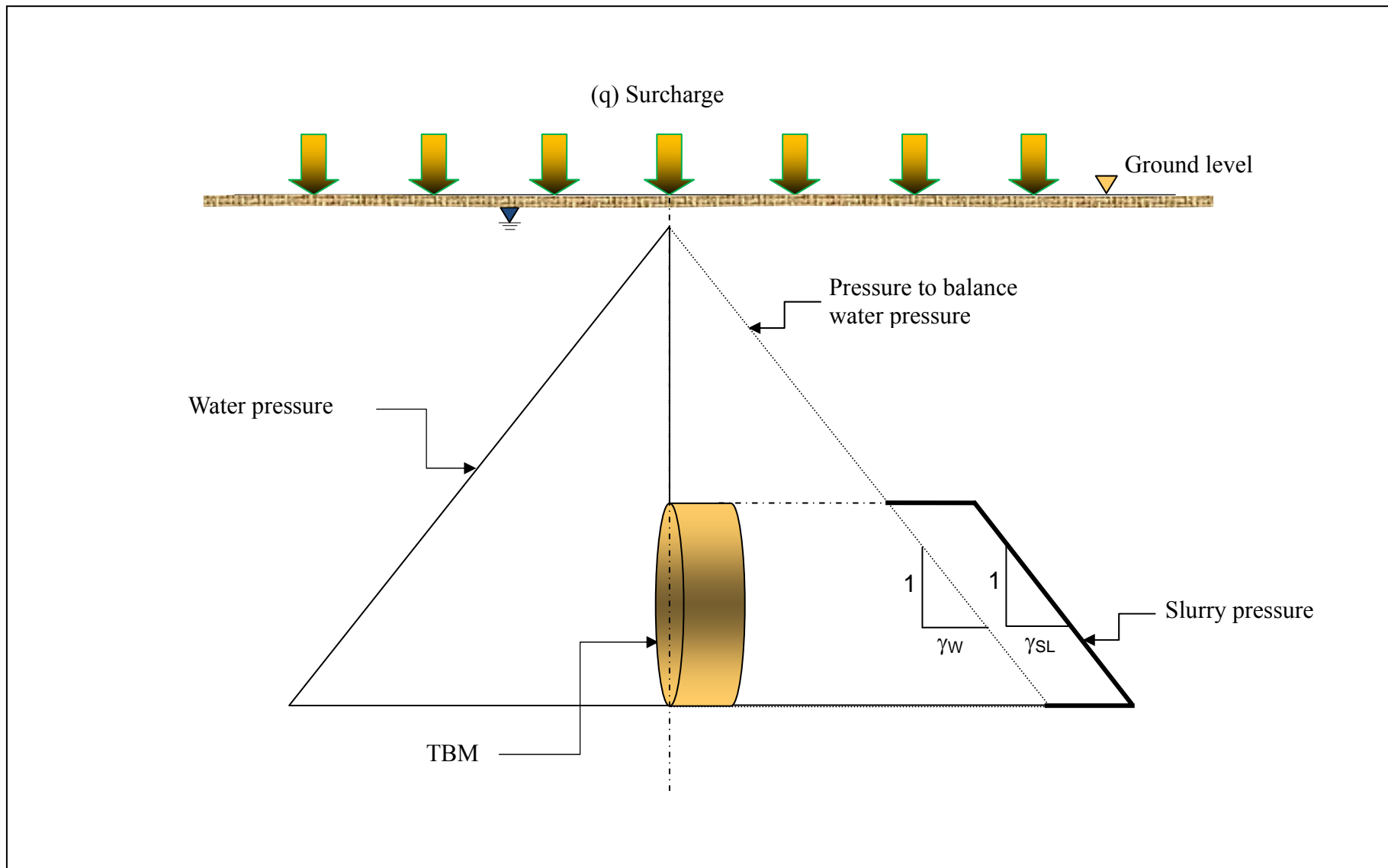


Figure 5 - Variation of Slurry Pressure Over the Face

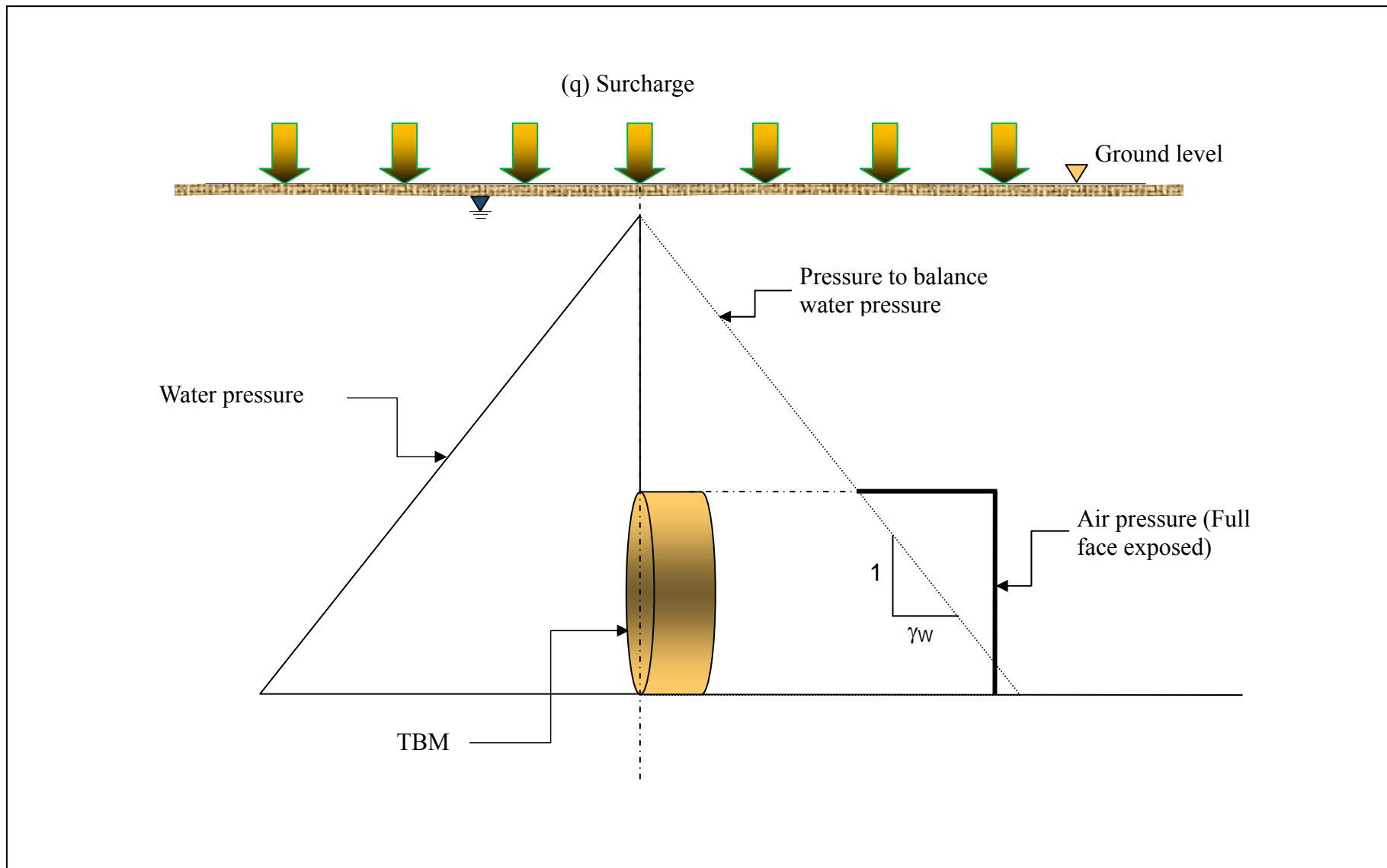


Figure 6 - Compressed Air Pressure, If Set to Balance Water Pressure 1 m Above the Base of the Exposed Face

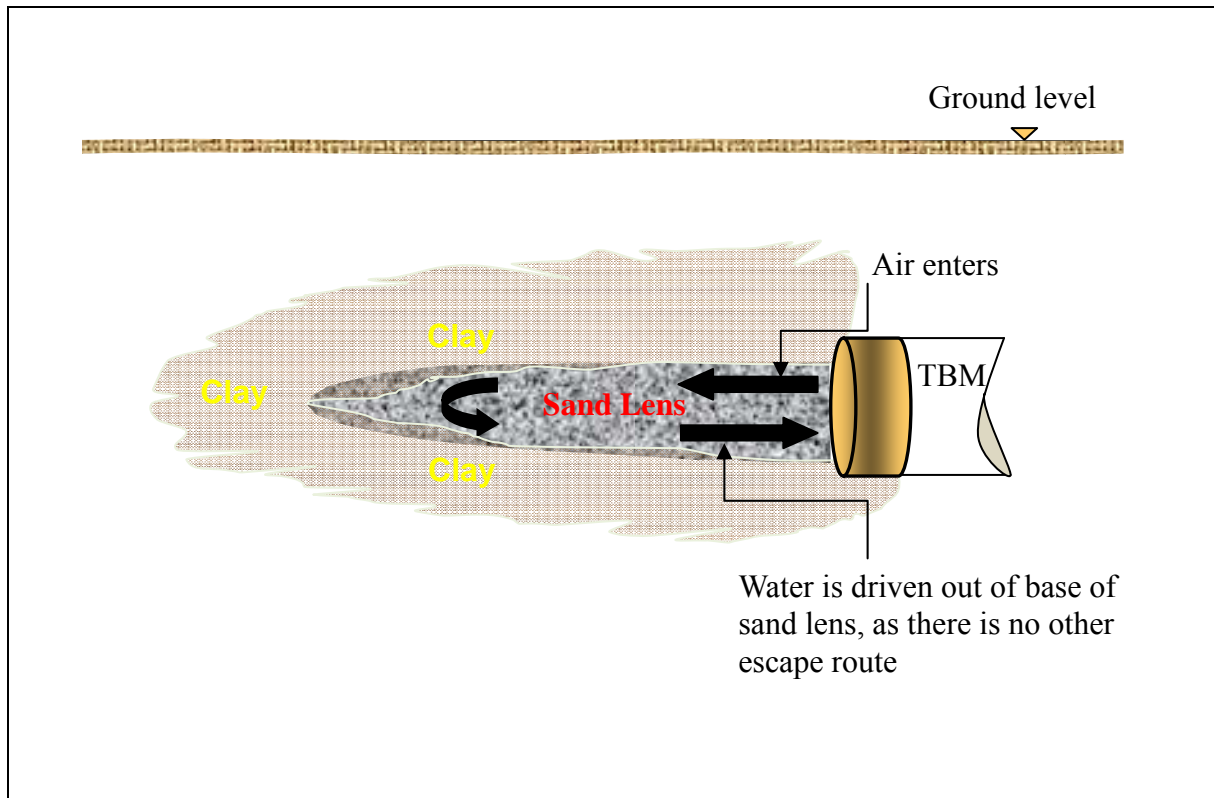


Figure 7 - Effect of Use of Compressed Air in a Lens or Pocket of Sand

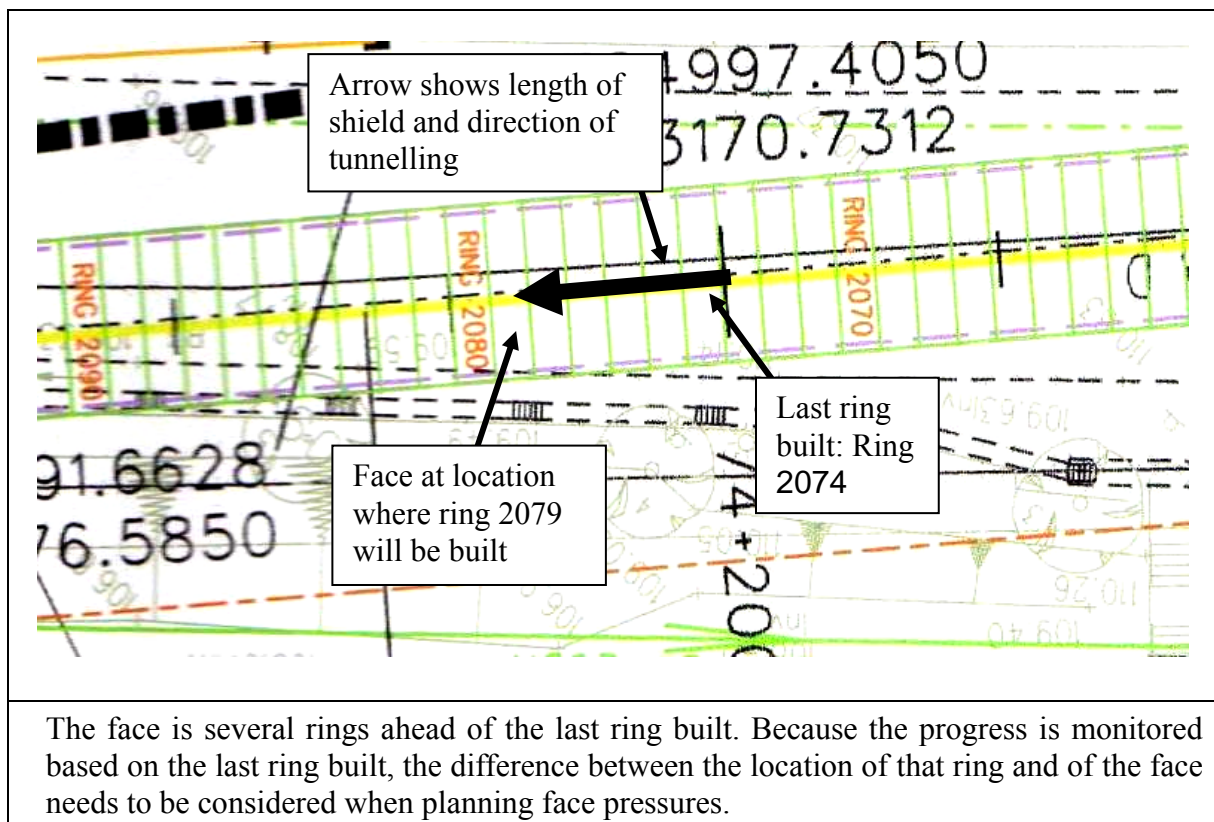


Figure 8 - Plan View of a Tunnel Showing Ring Numbers

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Plate 1 - Slurry from a TBM Running Over a Road - a Safety Issue for the Traffic

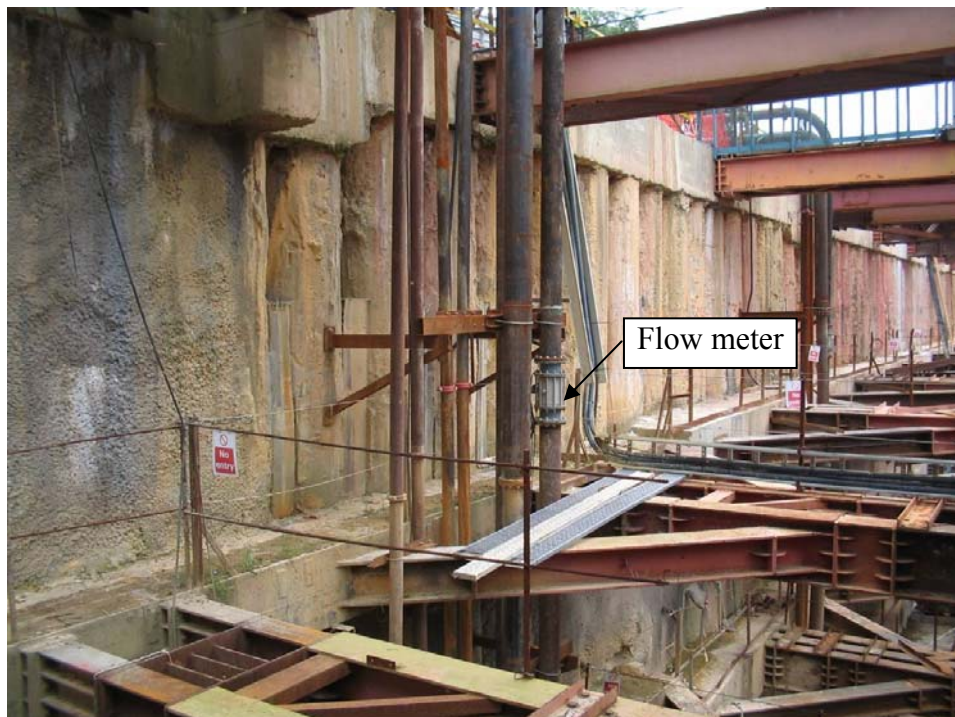


Plate 2 - Flow Meter Installed on Feed Line at the Tunnel Shaft
Where the Pipework is Vertical



The jointed steel plates, seen in the photograph, hold in place a continuous rubber seal. Grout pipes for simultaneous tail void grouting are also visible, at the tail of the shield.

Plate 3 - Seal for Launching of TBM



Plate 4 - Examples of Rock Fragments Trapped in the Openings of the Cutterhead of a Slurry TBM

GEO PUBLICATIONS AND ORDERING INFORMATION

土力工程處刊物及訂購資料

A selected list of major GEO publications is given in the next page. An up-to-date full list of GEO publications can be found at the CEDD Website <http://www.cedd.gov.hk> on the Internet under "Publications". Abstracts for the documents can also be found at the same website. Technical Guidance Notes are published on the CEDD Website from time to time to provide updates to GEO publications prior to their next revision.

Copies of GEO publications (except maps and other publications which are free of charge) can be purchased either by:

writing to

Publications Sales Section,
Information Services Department,
Room 402, 4th Floor, Murray Building,
Garden Road, Central, Hong Kong.
Fax: (852) 2598 7482

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- Calling the Publications Sales Section of Information Services Department (ISD) at (852) 2537 1910
- Visiting the online Government Bookstore at <http://www.bookstore.gov.hk>
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Chief Geotechnical Engineer/Planning,
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Fax: (852) 2714 0275
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部份土力工程處的主要刊物目錄刊載於下頁。而詳盡及最新的土力工程處刊物目錄，則登載於土木工程拓展署的互聯網網頁 <http://www.cedd.gov.hk> 的“刊物”版面之內。刊物的摘要及更新刊物內容的工程技術指引，亦可在這個網址找到。

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- 致電政府新聞處刊物銷售小組訂購 (電話: (852) 2537 1910)
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傳真: (852) 2714 0275
電子郵件: wmcheung@cedd.gov.hk

MAJOR GEOTECHNICAL ENGINEERING OFFICE PUBLICATIONS

土力工程處之主要刊物

GEOTECHNICAL MANUALS

Geotechnical Manual for Slopes, 2nd Edition (1984), 300 p. (English Version), (Reprinted, 2000).

斜坡岩土工程手冊(1998)，308頁(1984年英文版的中文譯本)。

Highway Slope Manual (2000), 114 p.

GEOGUIDES

Geoguide 1 Guide to Retaining Wall Design, 2nd Edition (1993), 258 p. (Reprinted, 2007).

Geoguide 2 Guide to Site Investigation (1987), 359 p. (Reprinted, 2000).

Geoguide 3 Guide to Rock and Soil Descriptions (1988), 186 p. (Reprinted, 2000).

Geoguide 4 Guide to Cavern Engineering (1992), 148 p. (Reprinted, 1998).

Geoguide 5 Guide to Slope Maintenance, 3rd Edition (2003), 132 p. (English Version).

岩土指南第五冊 斜坡維修指南，第三版(2003)，120頁(中文版)。

Geoguide 6 Guide to Reinforced Fill Structure and Slope Design (2002), 236 p.

Geoguide 7 Guide to Soil Nail Design and Construction (2008), 97 p.

GEOSPECS

Geospec 1 Model Specification for Prestressed Ground Anchors, 2nd Edition (1989), 164 p. (Reprinted, 1997).

Geospec 3 Model Specification for Soil Testing (2001), 340 p.

GEO PUBLICATIONS

GCO Publication No. Review of Design Methods for Excavations (1990), 187 p. (Reprinted, 2002).
1/90

GEO Publication No. 1/93 Review of Granular and Geotextile Filters (1993), 141 p.

GEO Publication No. 1/2000 Technical Guidelines on Landscape Treatment and Bio-engineering for Man-made Slopes and Retaining Walls (2000), 146 p.

GEO Publication No. 1/2006 Foundation Design and Construction (2006), 376 p.

GEO Publication No. 1/2007 Engineering Geological Practice in Hong Kong (2007), 278 p.

GEO Publication No. 1/2009 Prescriptive Measures for Man-Made Slopes and Retaining Walls (2009), 76 p.

GEOLOGICAL PUBLICATIONS

The Quaternary Geology of Hong Kong, by J.A. Fyfe, R. Shaw, S.D.G. Campbell, K.W. Lai & P.A. Kirk (2000), 210 p. plus 6 maps.

The Pre-Quaternary Geology of Hong Kong, by R.J. Sewell, S.D.G. Campbell, C.J.N. Fletcher, K.W. Lai & P.A. Kirk (2000), 181 p. plus 4 maps.

TECHNICAL GUIDANCE NOTES

TGN 1 Technical Guidance Documents