Ground Control for EPB TBM Tunnelling

GEO Report No. 298

HKIE Geotechnical Division
Working Group on Cavern and Tunnel Engineering
and Geotechnical Engineering Office

Geotechnical Engineering Office
Civil Engineering and Development Department
The Government of the Hong Kong
Special Administrative Region

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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 in print. These include guidance documents and results of comprehensive reviews. They can also be downloaded from the above website.

The 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.

H.N. Wong
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July 2014

Foreword

In December 2009, the Geotechnical Engineering Office published GEO Report No. 249, "Ground Control for Slurry TBM Tunnelling", based on a report prepared by Golder Associates (HK) Ltd under an expert advice consultancy. Consultation had been made widely with the tunnelling and geotechnical industries before publication. One of the most frequent comments received was the need for an equivalent set of guidelines for Earth Pressure Balance Tunnel Boring Machine (EPB TBM) tunnelling.

The HKIE Geotechnical Division's Working Group on Cavern and Tunnel Engineering undertook to produce this guidance document using voluntary input from its members, with input and advice from local and international professionals in the tunnelling industry. The aim of the report is to provide guidelines for the auditing of the design calculations and work procedures relating to ground control during EPB TBM tunnelling in Hong Kong. There have been a number of major reports on pressurised TBM tunnelling in the last ten years including BTS/ICE (2005) and publications by international and national tunnelling associations (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 EPB 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 report should be used as a guide only. The content of this report may require further updating as more experience in EPB TBM tunnelling and performance data in local ground conditions become available. Users applying the guidance in this 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.

As most of the guidelines given in GEO Report No. 249 are also relevant to ground control of EPB TBM tunnelling, the relevant parts of the GEO Report, with suitable amendments where needed, are reproduced in this report. The agreement by Golder Associates (HK) Ltd for the extracts from the GEO Report No. 249 to be reproduced in this report is gratefully acknowledged.

A draft of this report was circulated to members of the HKIE Geotechnical Division's Working Group on Cavern and Tunnel Engineering, as well as to the Association of Geotechnical & Geoenvironmental Specialists (Hong Kong), the Hong Kong Contractors Association, International Society for Soil Mechanics and Geotechnical Engineering TC204, Lars Babendererde of Babendererde Engineers, Alastair Biggart of Alastair Biggart Tunnelling LLP, Richard Lewis of YL Associates Ltd, Nick Shirlaw of Golder Associates (HK) Ltd, Roger Storry and Bruno Combe of Bouygues Travaux Publics, Andy Raine of

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1 Scope and Objective

The objectives of this report are to provide background, calculation methods, references and other information about ground control during Earth Pressure Balance Tunnel Boring Machine (EPB TBM) (Figure 1.1) tunnelling.   The scope of the report includes:

    (a) determination of the appropriate face pressure to support the ground,

    (b) controlling target face pressures,

    (c) modifying excavated material parameters using additives,

    (d) establishing limiting pressures and volumes for tail void grouting,

    (e) the use of Excavation Management Control (EMC) system, and

    (f) high risk activities (break-in, break-out, head access, interfaces, wear, flushing).
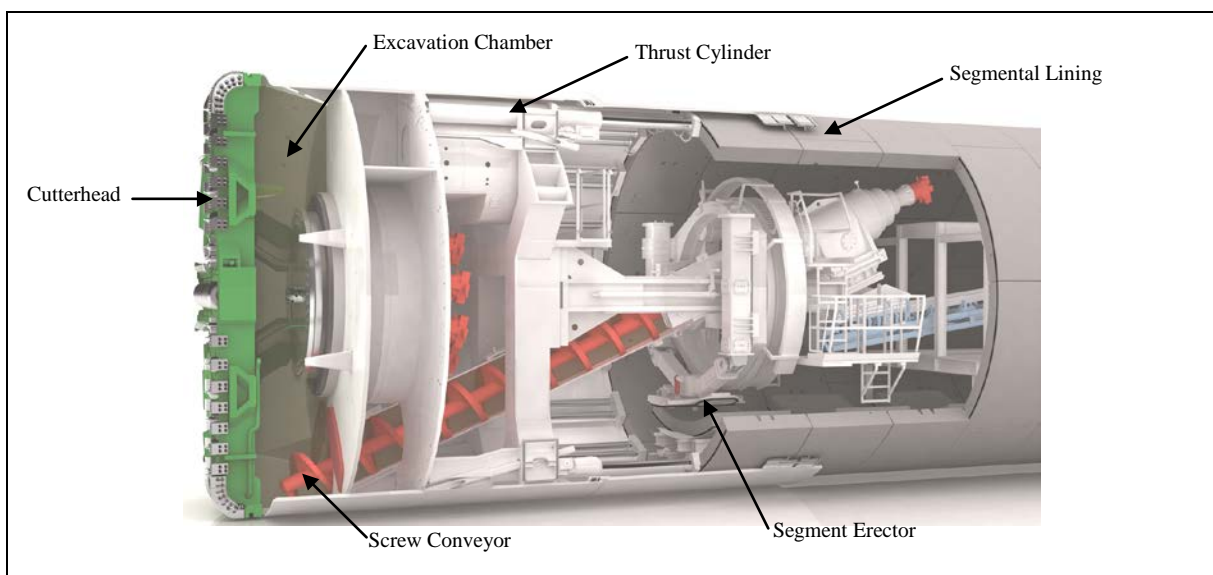
Figure 1.1   Schematic of an EPB TBM Showing Some of the Major Features (Courtesy of Herrenknecht AG)

The report concerns ground control during tunnelling using an EPB 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.   Site investigation is mentioned but not covered in detail. Readers should refer to GEO TGN 24 (GEO, 2009) and AGS (HK) (2004a, 2004b & 2005)

for reference.   Also, risk assessments are referred to, but not discussed in detail.   Readers should refer to other references on these subjects (e.g. Chiriotti et al, 2010; Kovari & Ramoni, 2006; Shirlaw et al, 2000).

This report provides guidance on the review of design calculations, drawings which show the planned face pressures, and documented work procedures.   To help to minimise the risk of unacceptable ground movements, it is required that these documents and drawings are prepared prior to tunnelling.   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 all types of TBM tunnelling.

2   Glossary of Terms and Symbols

The papers referred to in this report often use different terms/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 terms/symbols and a brief are given in the Glossary of Terms and Glossary of Symbols.   Terms/symbols for which there are standards, such as for soil strength, are not provided in the list of terms/symbols.

3   Face Pressure Assessment and Control

Establishing, and then maintaining, the correct face support pressure (face pressure) for the ground and groundwater conditions is critical to the safe operation of an EPB TBM. If face pressure is inadequate, 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 on overlying and nearby buildings, structures and utilities.   In this case, the Serviceability Limit

State (SLS) has to be considered as well as the ULS. For intact clays and clayey silts the face pressure is commonly calculated using total stress methods. Although effective stress methods can be used for intact clays and silts, this requires an advanced soil model that correctly models the stress path to failure. For sands and silty sands, effective stress methods are used.

Generally, EPB TBMs are selected for use in clays, silts and sands. The minimum face pressure should be calculated based on the effective stress method with an additional check based on total stress method for cohesive materials, such as clays and silts, and other coarse or granular materials embedded within a cohesive soil mass, to determine which governs the designed face pressure. The maximum acceptable face pressure to prevent heave must also be calculated.

EPB machines (unlike slurry machines) are reactive rather than proactive. Face pressure of an EPB TBM is maintained by a combination of propulsion thrust and removal of excavated material at the correct rate to match the rate of advance, in order to minimise the ground settlement and heave. If face pressure is lost, it can be maintained only by advancing the TBM. If the TBM cannot move forward, the face pressure can be maintained by pumping conditioners into the excavation chamber. An EPB TBM can be used in any soft or loose material, cohesive or non-cohesive, but is more effective and efficient in soft clays, silts and sands. In granular material there should be sufficient fine material to reduce the permeability of the excavated ground to minimise pressure loss along the screw. Reference to the grading curves interpreted from the ground investigation data is critical to the decision on TBM selection and its design parameters (Meritt et al, 2013). The approximate limit of permeability for the suitability of EPB machines is considered to be 10^{-5} m/s. By increasing the percentage of conditioning agent, or by adding a thickener, such as pulverised limestone, an EPB machine can be used in soil with a permeability greater than 10^{-5} m/s.

An EPB TBM can be operated in three basic modes (Babendererde et al, 2005):

(a) Open mode – No active face support is provided under this mode which will be applied for tunnels in stable rock (Section 3.2.3) or where the ground is stable and without any water inflow, groundwater drawdown and settlement problems.

(b) Semi-closed, semi-EPB or compressed air mode – The excavation chamber is approximately half full of spoil, with the spoil fully covering the inlet to the screw conveyor and the upper half of the chamber supported by compressed air. Typical EPB conditioning agents, e.g. foam and/or bentonite slurry, are used to reduce air loss. This mode may not be considered as a traditional EPB mode and is only used where traditional EPB operation is either not possible or extremely slow, or causes excessive damage to the TBM (see Section 3.8). This area of technology is still developing, with the appearance of hybrid EPB/Slurry machines now being produced for pilot use in appropriate

ground conditions.

(c) Full EPB mode or closed EPB mode – The excavated chamber is completely filled and the face support pressure is provided through the control of the rate of TBM advance and the rate of removal of the excavated spoil.

In assessing the face support pressure for the full EPB mode discussed throughout in Section 3, it is assumed that the ground is fully saturated and that the excavation chamber is completely filled with excavated material, commonly referred to as excavation paste, with or without the addition of soil conditioning additives (Section 5).

The target face pressure should be calculated at intervals along the tunnel alignment. The intervals chosen will depend upon the variability of the predicted geology, the ground cover to the tunnel and the sensitivity of adjacent structures.

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 (e.g. sand lenses or interface contact zones),

(f) assessing the maximum acceptable face pressure,

(g) pressures for using compressed air for head access,

(h) use of compressed air or semi-EPB mode,

(i) adjustment to the target face pressures based on observations,

(j) presentation and communication of target face pressures,

(k) why the target face pressure might not be applied in practice, and

(l) some key issues for designers and design checkers.

3.1 Factors 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 ϕ', c', s_u, or to the surcharge, q, in the rest of this 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:

       (a) on tan ϕ': divide by 1.2.

       (b) on c': divide by 1.2 (noting that c' is generally taken as zero for soil in these calculations).

       (c) on s_u: divide by 1.5.

       (d) 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:

       (a) When the surcharge is unfavourable (i.e. for inward yielding of the face): multiply by 1.5.

       (b) 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 closed face tunnelling is that an excess pore pressure can develop ahead of the face, provided that there is no inward seepage. This excess pore pressure reduces effective stresses in the area of the face, and increases the support pressure required (Broere, 2003). 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 (2003) for the influence of this effect. Where there is inward seepage, this will counteract against the excess pore water pressure ahead of the face. This effect can migrate a substantial distance from the excavation face.

Currently, there is limited information on the pore pressure changes near the face of an EPB TBM tunnel. In granular soils, there is typically some degree of drainage out of the face and through the screw conveyor, reducing the pore pressure but creating seepage pressure. This effect can be minimised by the correct use of conditioning agents. The use of foam, as an additive, may cause some transient increase in pore pressures just ahead of the face, and may partially counteract the effects of the drainage. Generally, it is assumed that the pre-tunnelling groundwater pressure represents an upper bound pore pressure for EPB tunnelling. However, this assumption should be reviewed for each specific case.

3.2 Effective Stress Calculations of Minimum Face Pressure

In general terms, the target (minimum) face pressure (P_{Et}) is calculated with an allowance for variation, using the following equation:

$$P_{Et} = \text{water pressure} + \text{pressure to balance effective pressure}$$
$$\text{from soil and surcharge} + \text{allowance for variation in pressure} \quad\text{............} \quad (3.1)$$

The water pressure varies from crown to invert (Figure 3.1), 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 for a particular pressure sensor at a known level below the crown of the TBM or tunnel axis.

Surcharge (q)

Ground Level

Pressure to Balance
Water Pressure

Water Pressure

Excavation
Paste Pressure

1 γ_W

1 γ_E

TBM

Note: Please refer to Glossary for the definitions of symbols.
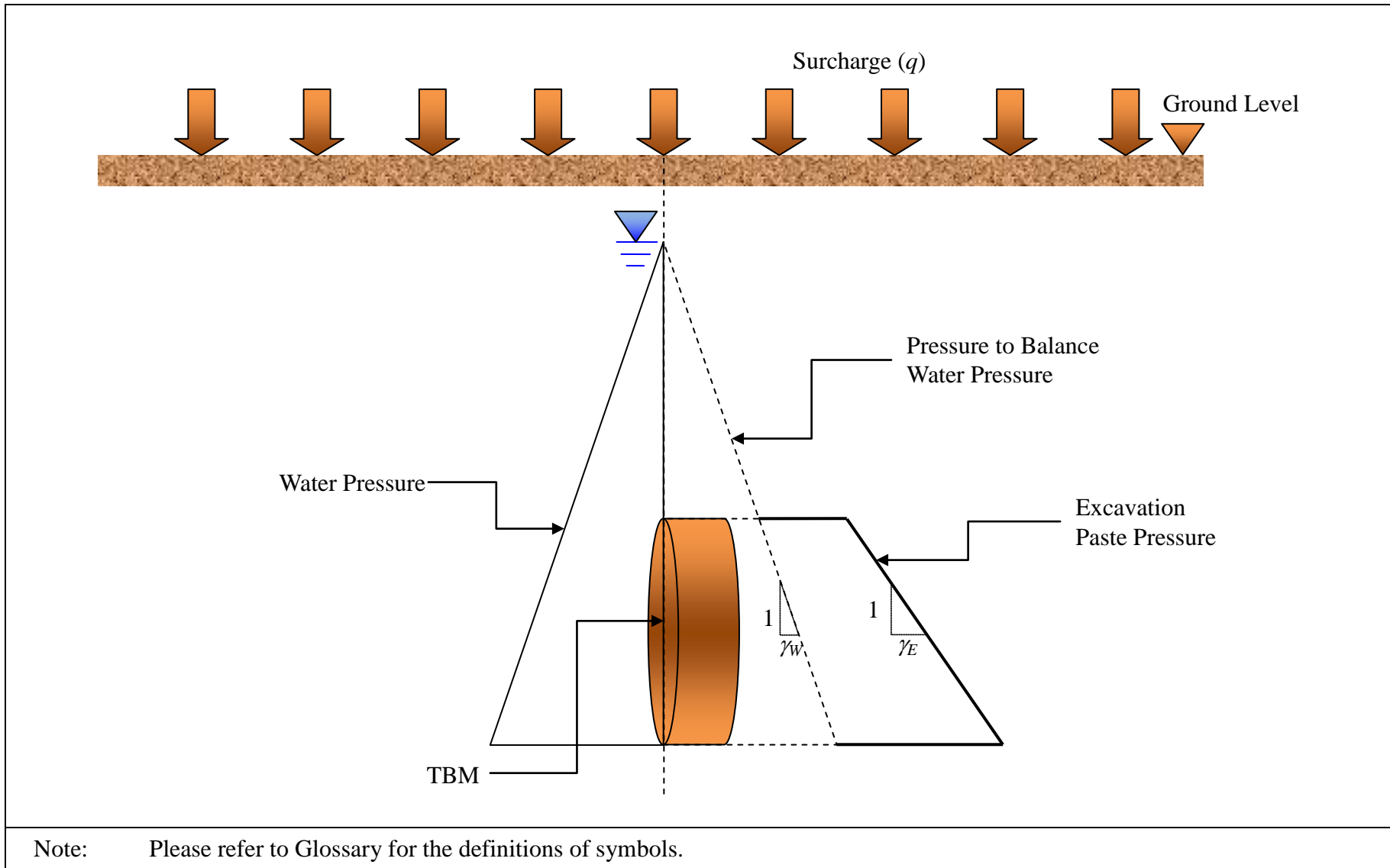
Figure 3.1   Variation of Pressure over the Face

For soil pressure, two particular methods of calculating minimum face pressure using effective stress methods are summarised 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 adopted for use in EPB 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 at the crown (Figure 3.2).   Substituting the appropriate terms for the respective sensor into the equations can derive the pressure at sensors 2 and 3.   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 should be noted that the properties of the excavation paste 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, and
>
> (d) diaphragm walls or piles, where the machine has to break through them.

For guidance on the choice, mixing, quality control testing and treatment of the excavation paste by additives, reference should be made to manufacturers' product data and laboratory testing of soil samples prior to the commencement of the tunnel excavation.   In order to achieve effective support during EPB TBM tunnelling, it is necessary to ensure that appropriate additive materials are used and to carry out detailed laboratory and site trials well before commencement of tunnelling, as the effect of additives used singularly or in combination varies.   The trials will inform the requirements for regular quality control for the properties of the excavation paste.   Tests on slump value, bleeding ratio and filtered water can be considered as quality control tests to verify the properties of the excavation paste. A detailed testing schedule and acceptable ranges for test results should be included in the work procedures.   The site supervisory staff should check compliance with the requirements.

Figure 3.2 Dimensions

Piezometric Level

Ground Level

Z_W

Z_0

C

Z_{S1}

Sensor 1

D

Sensor 2

D_0

Sensor 3

Piezometer

TBM

D_1
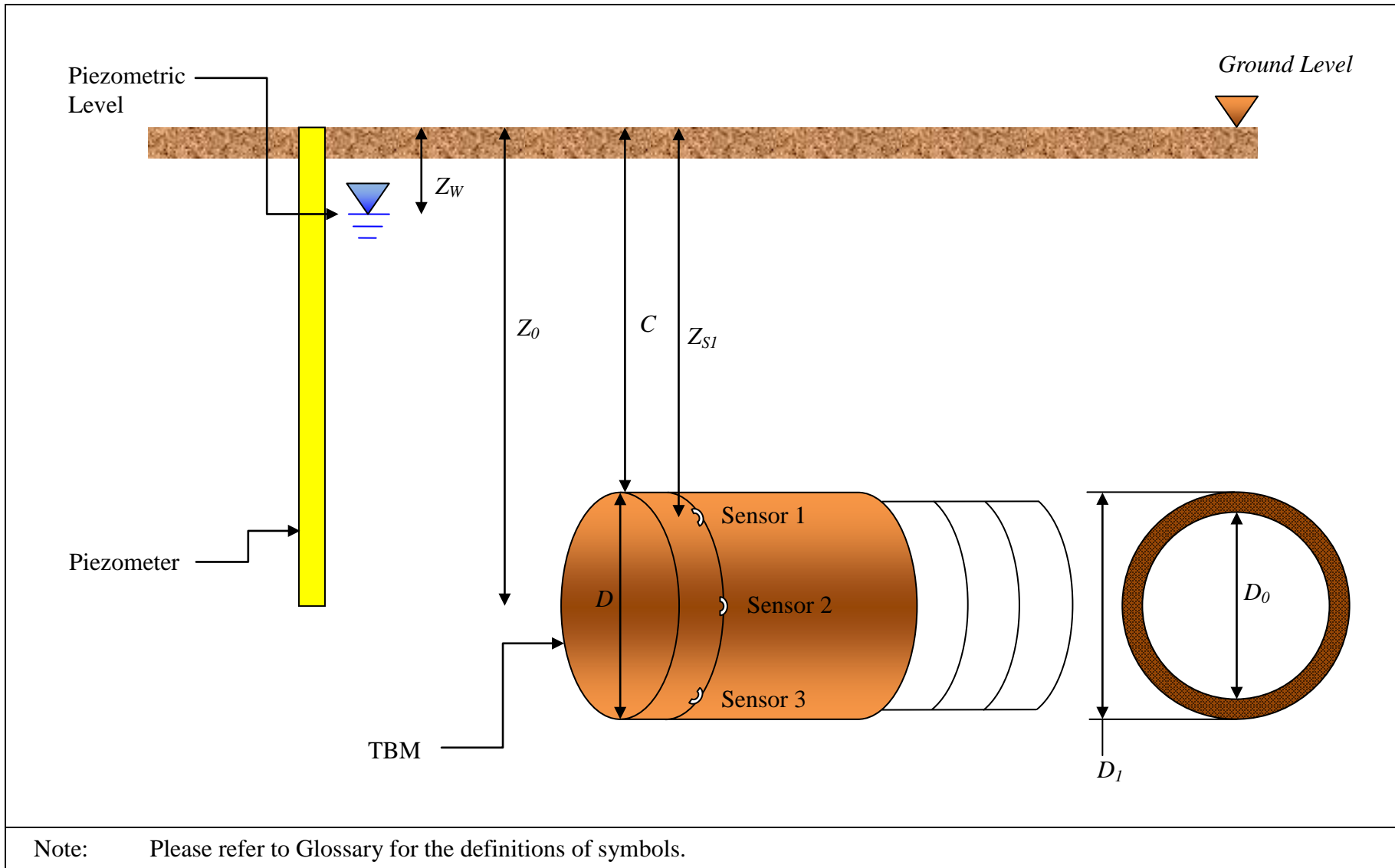
Note: Please refer to Glossary for the definitions of symbols.

The contractor should also provide, as part of the work procedures, a contingency plan for identifying and responding to any previously unidentified soft/loose soil or highly permeable zones encountered during the tunnelling. Where such risks are considered likely, which could result in large volume losses (Shirlaw, 2002), the machine specification, design and work procedures should contain adequate provisions to mitigate the risks. For example, the TBM specification and design may include a bentonite slurry injection system, sometimes known as Auxiliary Face Support (AFS) system, which can automatically inject slurry into the excavation chamber when pressure at the tunnel face drops below the agreed minimum target face pressure. This system should have an interlock mechanism to prevent further excavation advance and to close the screw discharge gate. The work procedures should include the maintenance of an adequate store of bentonite slurry.

In slurry TBMs, the slurry forms a filter cake in advance of tunnelling and there is no significant penetration or loss of slurry into the ground. Hence, the membrane model given in Anagnostou & Kovari (1996) applies. In EPB TBMs, Anagnostou & Kovari (1996) suggested use of a seepage model where face pressure significantly lower than water pressure can be used provided that the additional effective stresses due to inward seepage is considered. However, the seepage model is not reliable because it is difficult to control the amount of inward seepage, which can induce consolidation settlement and may not be tolerated. Also, it may not be possible to maintain inward seepage/drainage during the build cycle and if significant water and earth pressures build up the paste in the screw conveyor will be blown out when the discharge gate is opened, leading to loss of ground. Therefore, it is common practice to carry out calculations for EPB TBMs based on the membrane model, which gives an upper bound value for the required total face pressure. For critical cases such as shallow ground cover, highly variable ground conditions, sensitive structures in the vicinity of the tunnel, etc., it may be appropriate to use numerical analysis to provide additional justification for the target face pressure.

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

The target face pressure at the crown of the tunnel ($P_{Et(Crown)}$) is calculated, using the membrane model, from:

$$P_{Et(Crown)} = \text{pressure due to water} + \text{presure due to soil} + \text{pressure} \atop \text{due to surcharge} + \text{allowance for variation in pressure} \quad \text{............ (3.2)}$$

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 3.2 can be evaluated as follows:

$$\text{The pressure due to water at the crown} = (C - Z_w)\gamma_w \quad \text{...................... (3.3)}$$

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.[1] This equation is a

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[1] In the paper H is used as the symbol for cover over the tunnel, where C is used here.

truncated version of the equation in their paper, as the membrane model is assumed for slurry/EPB tunnelling.

The pressure due to surcharge at or near to the ground surface can be assessed using the charts in Atkinson & Mair (1981). 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, may be 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 (Figure 3.3). Consistently achieving a target face pressure within the tolerance requires a skilled operator, the correct machine configuration, and appropriate additives for the ground conditions. The value for the variation in pressure varies with the density of the ground and will usually be greater for mixed ground than for a tunnel in a single material. The value for the variation in pressure to be used in the calculations should be based on the ground conditions, its variation and documented experience for the type of TBM proposed. It should be regularly reviewed during tunnelling as it may require the face pressure calculations to be revised.
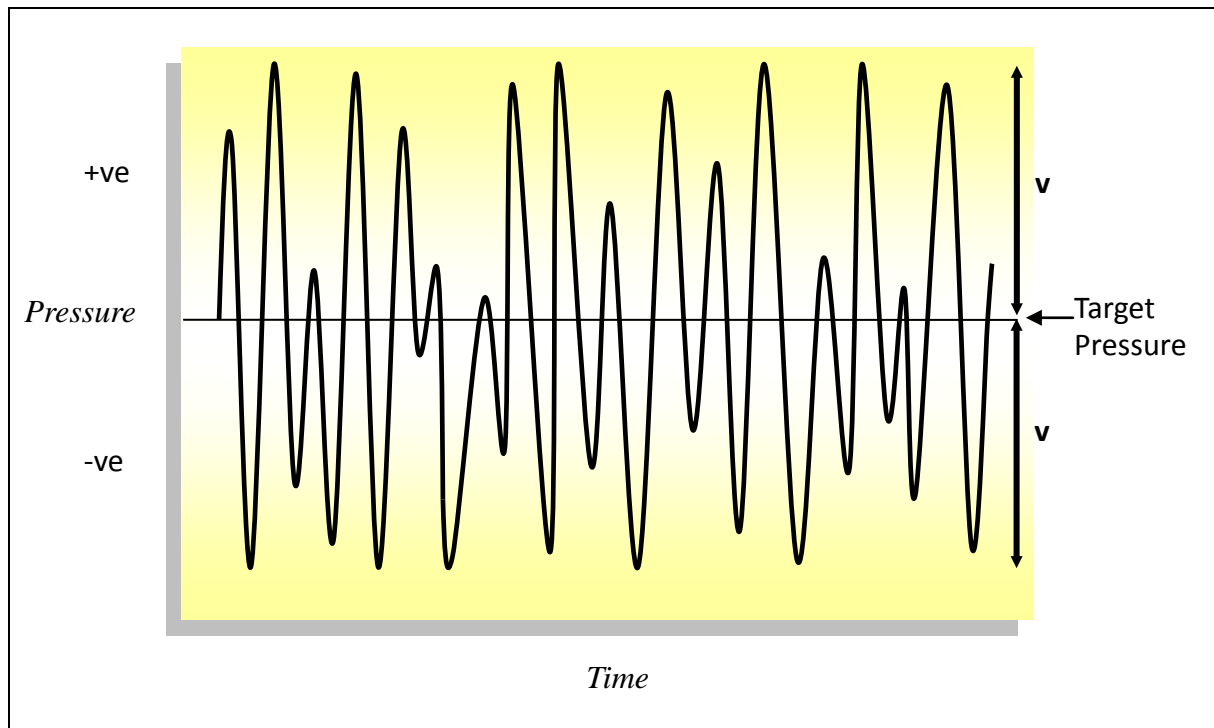
Figure 3.3   Fluctuation of Face Pressure

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

$$P_{Et(S1)} = P_{Et(Crown)} + (Z_{S1} - C)\gamma_E \quad\text{..} \quad (3.4)$$

As the unit weight of the excavation paste 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 face pressure at the sensor required to achieve the calculated pressure at the crown is then assessed.

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

$$P_{E(S1)} = P_{Et(S1)} - v \quad\text{..} \quad (3.5)$$

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 soils in Hong Kong.

The charts in Anagnostou & Kovari are derived from calculations using a limit equilibrium model proposed by Horn (1961). 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 adopt 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 3.1, except that the excavation chamber 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 SPT-N > 30, use $0.25\ \gamma'D$,

    (b) for saprolite or residual soil with SPT-N < 30 but > 10, use $0.4\ \gamma'D$, and

(c) for granular superficial deposits with SPT-N < 10, use 0.55 γD.

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 face pressure is not the only factor that influences the volume loss over the tunnel; other factors include the size of the overcut gap and tail void and the time taken to install the permanent stiff support, i.e. the tunnel lining, the quality of which also requires effective grouting of the tail void (Section 6). 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, mixed ground conditions especially at rock/soil interfaces, which could lead to differential pressures across the tunnel face, or soils with a low shear strength (such as loose sand and soils rich in mica) and variations in the depth of cover.

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 an EPB TBM drive is in a full face of stable rock, then it may be possible for the machine to operate in 'open mode', even to the extent of replacing the screw conveyor with a belt conveyor for the majority of the rock sections of the tunnel, as face pressure is not required for stability. If the screw is retained, but the face is stable with or without the use of compressed air, this is more accurately referred to as the 'semi-EPB mode' (Section 3). There still needs to be sufficient material in the excavation chamber to ensure that the screw conveyor entrance is covered so that the screw can work at the designed capacity. This can increase efficiency and reduce the wear or risk of blockage on the screw conveyor.

Where the top of the stable rock is well established and there is low risk of a change in the geology along the tunnel drive, the EPB TBM can be designed to be modified from closed EPB mode or semi-EPB mode to work in open mode by removing the screw conveyor and using a belt conveyor instead. This approach is particularly recommended if the length in stable rock is considerable. Additional spoil handling plates are required in the excavation chamber during open mode operation to lift and discharge the excavated rock onto the discharge conveyor boot end.

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. Additional effects on the annulus grout injection and groundwater seepage affecting the grout should be considered if planning to operate an EPB in open mode. This should only be considered in stable ground with no hydrostatic head, low permeability soil or in an area where the effects of lowering the hydrostatic head are not a concern.

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 (Section 3.5).

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

Where there is a risk of encountering highly fractured rock, or the consequence of encountering such a feature causing partial face collapse is high, consideration should be given to operating in semi-EPB mode with the excavation chamber only partially filled. This does provide the advantage over open mode in that it allows the excavation chamber to be promptly filled and pressurised with excavated material, or with a bentonite slurry from the reserve tank within only several minutes, thus minimising ground or excessive water loss from the excavation face and its impact. The criteria on when the excavation chamber should be filled should be assessed in design and communicated to the TBM operator for him to follow. The semi-EPB mode without compressed air is however not recommended and should be used only where a risk assessment indicates the consequences of substantial face loss and settlement due to groundwater inflow and drawdown are acceptable.

3.3 Total Stress Calculations of Minimum Face Pressure

For TBM tunnelling, effective stress calculations generally govern the minimum face pressure in coarse materials. However, for EPB TBM tunnelling, where the excavation is often 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. However, it should be noted that unstable granular soils could exist as beds or lenses within a relatively stable cohesive mass (i.e. Young & Dean, 2010). Where it is uncertain as to 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:

$$P_E = (\gamma\, Z_0 + q) - (s_u\, N_{TC}) \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots \text{(3.6)}$$

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 face pressure is then:

$$P_{Et} = P_E + v \quad .... \quad (3.7)$$

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

$$P_{Et(S1)} = P_{Et} - (Z_0 - Z_{S1})\gamma_E \quad .... \quad (3.8)$$

3.3.2   Total Stress SLS Calculations

The potential volume loss at the face and along the body of an EPB shield can be evaluated using the results of the model tunnel tests presented in a chart in Kimura & Mair (1981).   The chart relates volume loss to the Load Factor (*LF*), the ratio of the Face Stability Index (*N*) and the limiting stability index at the point of total collapse (N_{TC}):

$$LF = N\big/N_{TC} \quad .... \quad (3.9)$$

where $\quad N \quad = \quad (\gamma Z_0 + q - P_E) / s_u$

In using the chart, the value of *P* should be taken as 0 and *L* (the length of the tunnel shield) for an EPB TBM without slurry injection and an EPB machine with slurry injection around the shield skin respectively.

The volume loss derived from the charts represents 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).

While an unsupported overcut may stay open long enough in stiff cohesive soils for grouting to be implemented, unsupported overcut will close in unstable ground.   Unless there are ports in the shield for injecting bentonite to support the surrounding ground, the volume loss due to the overcut should be evaluated in the SLS calculations.   It is recommended that all EPB TBMs should be designed with ports in the shield skin for injecting bentonite into the shield annulus to control the ground movement into the overcut gap or to reduce the thrust during TBM advancing.

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 (1981) or calculated from the equation in Dimmock & Mair (2007), and the minimum face pressure required to achieve the allowable *LF* can be calculated from:

$$P_E = (\gamma Z_0 + q) - (s_u N_{TC} LF) \quad .... \quad (3.10)$$

For the SLS case in total stress analysis, it is not necessary to allow for the variation in the face pressure, provided the variation is kept with a limited range.   Then:

Equation 3.7 changes to: $P_{Et} = P_E$

Equation 3.8 applies unchanged.

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 Dimmock & Mair (2007) for an application of this in London Clay).

> (b) Where 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 simpler 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 homogenous 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 the following two methods:

(a) Carrying out calculations based on the model proposed by Horn (1961). The charts in Anagnostou & 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) By numerical analysis. Finite element or finite difference 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. The pressure changes need to be managed such that the higher pressure required to support the weaker unit is applied while there is still a sufficient cover of the stronger unit over the crown of the tunnel. As a rule of thumb, the cover should be at least one tunnel radius. 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 be 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 fullest reasonable variation in ground and groundwater conditions given the anticipated geology.

The location of interfaces between units requiring a significantly different target face pressure must be assessed conservatively, recognising the likely variation in ground conditions. 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.

An EPB TBM may be excavating in generally stable ground conditions, but with local features that require face pressure and operation in a suitable mode 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 and the groundwater pressure may not have been identified in the ground investigation. In this case, it may be appropriate to select the minimum face pressure based on the worst expected ground and groundwater conditions, i.e. for the faulted or weathered zone(s) or soil.

It may be possible to 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.

The selection of an option to give the highest minimum face pressures might appear to be the 'safest' option. However, the wear on TBM cutting tools tends to increase 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, as well as adverse health and safety implications to the workforce where more frequent and higher pressure interventions are required. Excessive wear may also result in other parts of the machine malfunctioning such as excessive wear on the screw conveyor, shield skin and cutter housings. This may lead to an inability to exert the desired face pressure, when weaker soils are encountered. Thus, excavating 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 is caused by advancing the machine with inadequate

muck removal through the screw conveyor. This can be due to careless or poor operation of the TBM. It can cause damage to the TBM (main seals and tail seals) and may result in heave at the ground surface (less common than in Slurry TBMs). If an EPB TBM encounters an open path to the surface (such as an old borehole) there is likely to be an immediate loss of face pressure. This can lead to an eruption of foam, other conditioning agents and/or grout to the surface, but does not necessarily lead to a subsequent face collapse. However, when operating under similar conditions, if there is a significant loss of compressed air during an intervention, this can result in instability of the face and endanger the workers in the chamber through decompression or face collapse.

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 generally not a major concern. The concern is with heave that is sufficient to cause damage, or to rupture the overburden.

The pressure required to cause unacceptable ground heave is effectively the pressure required to lift the block of ground over the tunnel - the reverse of the problem analysed by Anagnostou & Kovari (1996).

This pressure is equal to the insitu 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 in intact ground. For the pressure at sensor 1, this can be achieved if the target face pressure $P_{Et(S1)}$ is checked for:

$$P_{Et(S1)} < C\,\gamma + (Z_{S1} - C)\gamma_E - v \quad .... \quad (3.11)$$

$C\,\gamma$ is the total overburden pressure at tunnel crown. $(Z_{S1} - C)\,\gamma_E$ is the difference in 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 face pressure.

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

$$P_{E(S1)} < C\,\gamma + (Z_{S1} - C)\gamma_E \quad .... \quad (3.12)$$

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 face 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

As discussed above, where there is an open path, foam, other conditioning agents and/or grout can erupt at the surface.   The presence of the material at the surface can cause problems for third parties, such as when it appears on a road, pavement, in a basement or flows into a drain.   Loss of tail void grout to the surface can also cause problems in the tunnel, due to the inadequate tail void grouting that can result from the loss.   It is unlikely that an EPB TBM operated under the full EPB mode will cause any substantial eruption at the surface, because as soon as an open path is encountered the pressure will dissipate rapidly.

The potential impact on the tunnel and third parties has to be managed as part of the risk management for the tunnel.   Common mitigation measures include:

> (a) identifying any likely open paths and grouting them in advance of tunnelling,
>
> (b) maintaining continuous surface watch during TBM tunnelling, and implementing control measures if a loss of material is observed, and
>
> (c) testing whether the compressed air pressure can be maintained before allowing access into the chamber.

All of the above measures are normally implemented in urban areas as it is unlikely that all of the open paths can be identified from existing records.

Because the face pressure in a slurry machine is pro-active whereas in an EPB machine it is reactive, if there is an open path which allows the excavation paste to move up it, face pressure in an EPB machine will be immediately lost until the shield is pushed forward. Unlike slurry it will not continue to flow up the path.   If pressure is regained by moving the shield forward the 'flow' along the path will be restricted by the high viscosity of the paste as well as by its density.   Hence, it is often preferable to continue excavating forward with an EPB TBM until the location of the pressure loss is cleared, after which it can be grouted from within the tunnel or from the surface if access is possible and working space is sufficient.

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

3.7 Pressures for Using Compressed Air for Head Access

Compressed air can be used to allow access into the excavation chamber in unstable ground or where water seepage is unacceptably high, 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 (Figure 3.4). With an exposed face of granular soil, the compressed air will penetrate into the soil pores. The pressure of the compressed air minimises groundwater flow towards the face, but does not provide support to the effective stresses in the soil skeleton. For open face compressed air tunnelling in saprolite, the air pressure would be typically set to balance the water pressure at a level about 1 m above the base of the exposed face. For a full face this would mean that:

$$P_{CA} = [Z_0 + (D/2) - (Z_W + 1)]\gamma_W \quad \text{.......................................} \quad (3.13)$$

This results 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 needs to be provided using timbering or sprayed concrete. At the base of the exposed face the air pressure is usually slightly less than the groundwater pressure, leading to some seepage into the tunnel. CDG is 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 can be observed in the face, and the pressure is adjusted as necessary, based on those observations.

In coarse soil with a low fines content, such as in the beach and marine deposits in Hong Kong, there are a number of issues with the use of compressed air.

(a) Instability of the dried out soil: the sand near the crown is dried out by the effect of the compressed air and becomes running ground.

(b) Instability of the soil at the base of the exposed face: superficial sand deposits are less stable under seepage forces than saprolite. Even the minimal net water pressure can cause some erosion of the sand at the base of the exposed face. This can be exacerbated if the sand is a discrete lens. In this case the water cannot be pushed away from the face by the compressed air. Instead, the compressed air pressure raises the water pressure, causing increased flow and erosion at the base of the lens (Figure 3.5).

(c) Instability following a prolonged stoppage: the material in the face dries out and collapses following a re-start.
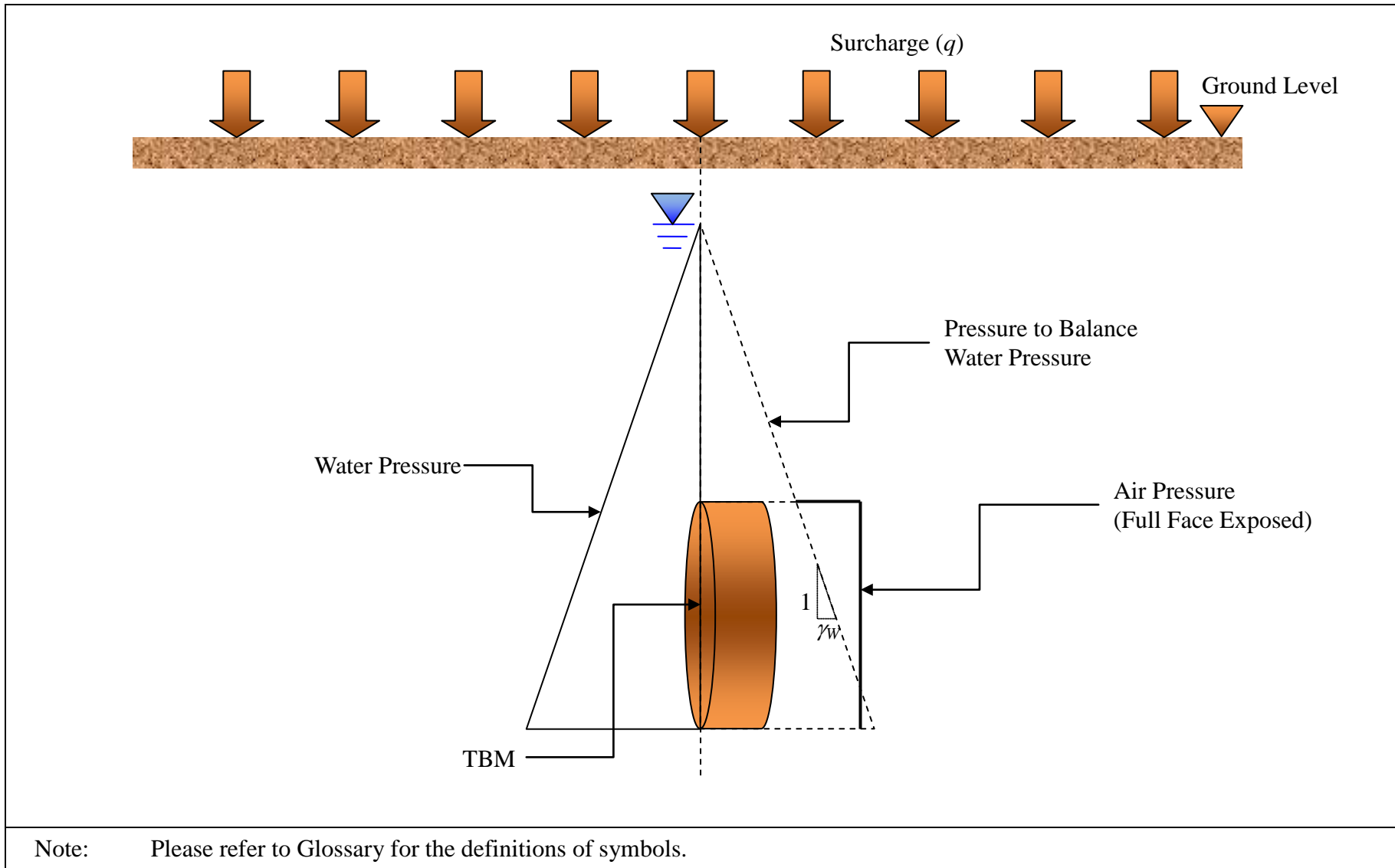
Figure 3.4 Compressed Air Pressure (Balancing Water Pressure 1 m above the Base of the Exposed Face)

Ground Level

Air Enters

Clay

Clay

Sand Lens

TBM

Clay

Water is driven out of base of sand lens, as there is no other escape route.
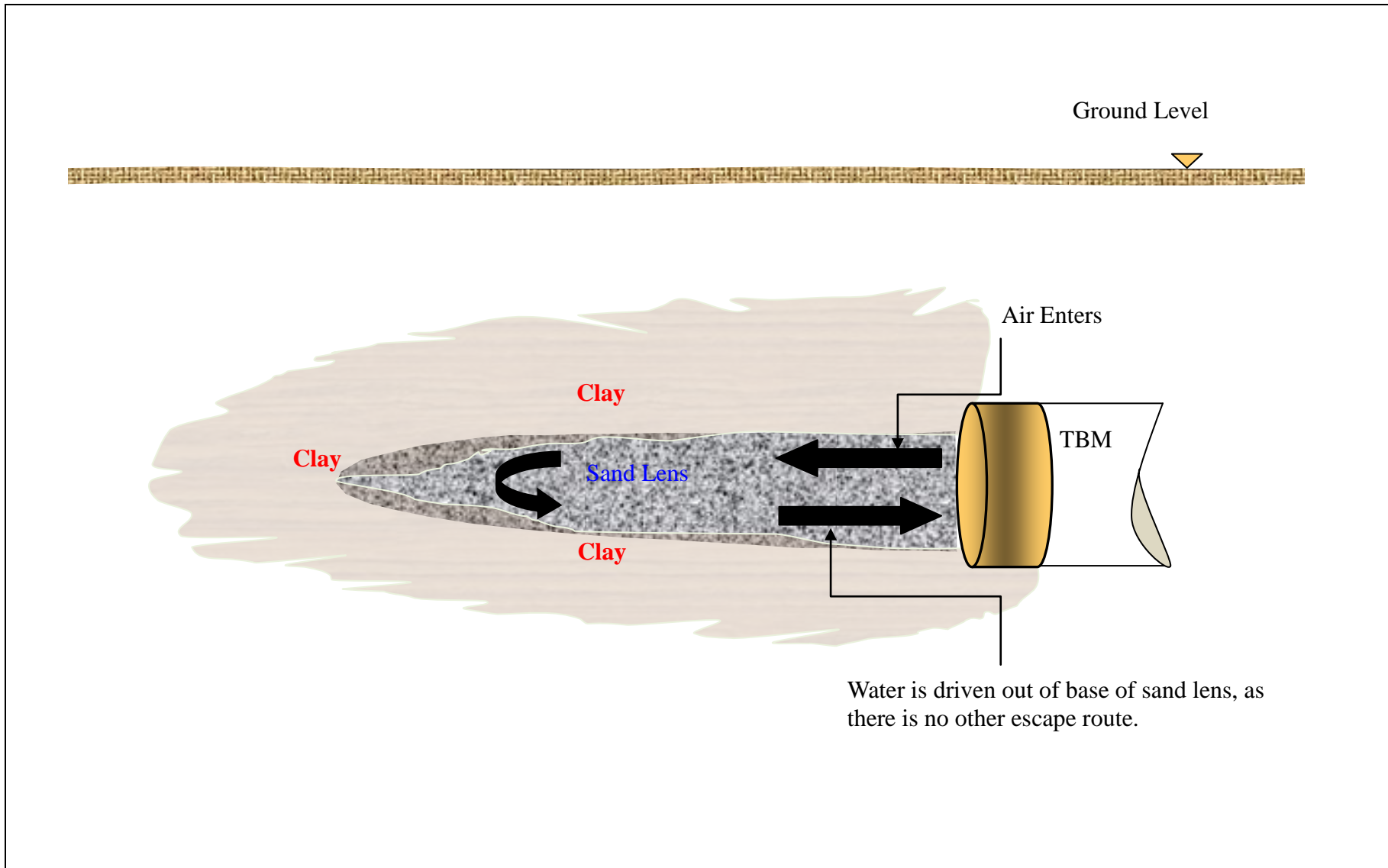
Figure 3.5   Effect of Use of Compressed Air in a Lens or Pocket of Sand

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. For EPB tunnelling a filter cake is not normally formed during the process of tunnelling. Where possible, during manned interventions, it is preferable to only remove the spoil within the excavation chamber down to axis level. This will minimize the size of the compressed air volume and therefore reduce the overpressure at the crown of the tunnel and minimize the risk of face instability as far as practicable. However, prior to a compressed air intervention a partial filter cake can be formed by replacing the muck with clean slurry for the 300 - 500 mm of excavation before the intervention. Any foam soil conditioning is stopped and replaced with reasonably large amounts of bentonite, so that, for this final section of driving, a very sloppy mixture of bentonite and excavated material fills the face. Results of this are:

    (a) this material is easier to draw down with the screw conveyor,

    (b) a layer of the 'sludge' is left on the face which acts as a filter cake, and

    (c) the screw conveyor is left full of softer material which makes it easier to start after a longer stoppage.

Where an emergency face pressure support system is incorporated into the TBM and back-up, this can provide the means to fill the excavation chamber with bentonite slurry. A 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.

The adoption of a bentonite replacement system for interventions on an EPB TBM should be considered in terms of the ground risk, risk to adjacent structures, risk to the intervention team, available space within the machine, the additional cost of the system, the additional complexity of the processes involved, and the time/cost of the replacement process at interventions. It is becoming more common to incorporate an emergency face support system into the larger diameter EPB TBMs, as it provides an additional option for reducing the risk related to excessive settlement due to EPB tunnelling and interventions during the tunnelling.

The filter cake can be maintained during interventions by spraying the face with fresh bentonite.

If an effective filter cake is formed, the compressed air pressure can be increased

above the value given in Equation 3.13. After allowing for the difference in the distribution of face pressure between EPB mode and compressed air mode, the total face support pressure can be increased to match that required in EPB mode, provided that this does not result in an unacceptable factor of safety against blow-out. 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,

(c) increase the overpressure at the crown of the tunnel, reducing the factor of safety against a blow-out, and

(d) increase the health and safety risks to the intervention team.

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 excavated material is drawn down and the nature of the ground conditions in and over the face. For most interventions, the material level in the excavation chamber is not fully drawn down, but is maintained at about axis level. This level of material 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. This should only be undertaken after careful assessment and planning as it adds greatly to the risk if personnel are working below axis within the excavation chamber.

For planning purposes, it is suggested that:

In CDG, Equation 3.13 is used to assess the face pressure for full emptying of the excavation chamber, while Equation 3.14 (below) can be used for drawdown to axis level.

$$P_{CA} = [Z_0 - (Z_W + 1)]\gamma_W \quad \text{...} \quad (3.14)$$

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

Average P_{CA} over the area of the exposed face $\geq$
Average P_{Et} over the area of the exposed face $\quad \text{........................} \quad (3.15)$

The higher of the face pressures from Equations 3.14 and 3.15 should be used in superficial deposits. In Equation 3.15, P_{Et} 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 closed face 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 exposed face or, where used, the filter cake. A dried filter cake will start to peel off the face. Depending on the nature of the ground, the exposed ground can then ravel or run. For such situations, provision should be made to replace the filter cake, or alternative materials/products such as sprayed polymers may be used to provide a seal at the face to prevent the compressed air from drying out the ground. Additional measures should be considered prior to re-starting excavation to avoid dried out ground ahead of the excavation face ravelling immediately after the re-start.

Interventions continue to be a major source of ground loss during closed face TBM tunnelling. The risks associated with interventions, and some possible control measures, are discussed in Section 8 below.

3.8 Use of Compressed Air or Semi-EPB Mode

Semi-EPB mode has been used in mixed faces of rock and saprolite, particularly where there is a high percentage of rock in the face. There are, however, a number of limitations and risks in the use of this mode, which need to be managed. In particular:

(a) Some ground conditions are unsuitable for this mode.

(b) The pressure distribution over the face and the target face pressure required will be different to the closed EPB mode, and the risk of a blow-out has to be checked.

(c) There is a risk of short-term heave during tunnelling; the heave may initially mask the longer term settlement over the tunnel.

(d) The compressed air can lose oxygen as it passes through the ground; there is therefore a risk of de-oxygenated air entering confined spaces, such as other tunnels in the vicinity. In addition the de-oxygenated air can leak back into the excavation chamber and air locks. Therefore, these chambers should be purged before the next manned entry.

(e) There is a risk of sudden loss of pressure if an old borehole, well or other open path to the surface is encountered.

3.8.1 Ground Conditions Unsuitable for Semi-EPB Mode of Operation

The permeability of ground to air is much higher than to water. The compressed air pressure can therefore readily penetrate into the pores in the soil or discontinuities in fractured rock. As a result, an excess compressed air pressure (over the water pressure) applied to the face of the tunnel does not support the soil skeleton or fractured rock, unless there is a low permeability membrane covering the face or the soil mass is of very low permeability and has a low air entry value. In semi-EPB mode, even when bentonite slurry is also injected, any filter cake is constantly being destroyed by the action of the cutterhead. In order to operate in soil in semi-EPB mode, the soil has to have sufficient cohesion for the face to be stable under compressed air. Semi-EPB mode is therefore only possible where the ground is stable under compressed air alone.

There has been extensive experience of compressed air tunnelling in Hong Kong. General observations from this experience are that:

(a) Superficial sand layers, such as alluvial and beach sands, are unstable under compressed air. The use of compressed air dries out the sand, which then runs. Venkta et al (2008) recorded face stability problems when trying to operate in semi-EPB mode in a mixed face of rock and sand in Singapore. If the sand bed is a confined aquifer, the compressed air pressure will tend to enter the top of the bed, pushing the pore water out of the base of the layer into the face. The flow of water at the base will erode the bed of sand.

(b) Compressed air provides an effective support pressure in clays and other very low permeability soils.

(c) Many saprolites are initially stable under compressed air. Over time, the drying effect of the compressed air will cause the saprolite to ravel. This needs to be considered, particularly if the TBM is stopped for a long period of time, as semi-EPB mode may not be suitable during the period of the stoppage.

3.8.2 Pressure Distribution Over the Face under Semi-EPB Mode of Operation

Compressed air pressure is constant over the height of the face that is exposed to the air. The pressure distribution is therefore different to that in closed EPB mode, where the pressure generally increases with depth due to the self-weight of the spoil. Idealised pressure distribution in semi-EPB mode is shown in Figure 3.6. In semi-EPB mode, there is an excess pressure at the crown of the tunnel. This excess pressure creates the risk of a blow-out, particularly if the tunnel is shallow relative to the tunnel diameter.

Face pressure calculations for closed EPB mode are not appropriate for semi-EPB mode. New calculations should be carried out considering the different face pressure distribution, the reduced variability in the face pressure, and to ensure that there is an adequate factor of safety against blow-out. Typically, the face pressure required in semi-EPB mode will be significantly lower than in closed EPB mode at the level of the tunnel axis, but higher at tunnel crown. Setting the target face pressure to the minimum needed for face stability will maximise the factor of safety against blow-out, and minimise the loss of compressed air and the rate of heave.

3.8.3 Risk of Short-term Heave

Some loss of compressed air during semi-EPB mode is inevitable, although the rate of loss depends on the permeability of the ground in the face, the area of the face exposed and the magnitude of the excess pressure (over water pressure) at tunnel crown. The 'lost' compressed air will migrate from the tunnel; during compressed air tunnelling it is common, after heavy rainfall, to see a mass of bubbles in the ponded rainwater due to the escaping air.

If the compressed air encounters a low permeability soil above the tunnel, the compressed air will build up under the cap of the low permeability soil. Ultimately, the pressure under the capping layer can build up to the pressure applied at tunnel level. This can lead to heave, or rupture, of the capping layer. The rate at which the heave develops depends on many factors, including the relative permeability between the soil at the face and the capping layer.

The heave at the ground surface will dissipate eventually, as the compressed air escapes, and the underlying settlement due to the tunnelling becomes evident. Until this occurs, the heave will mask the magnitude of the underlying settlement.
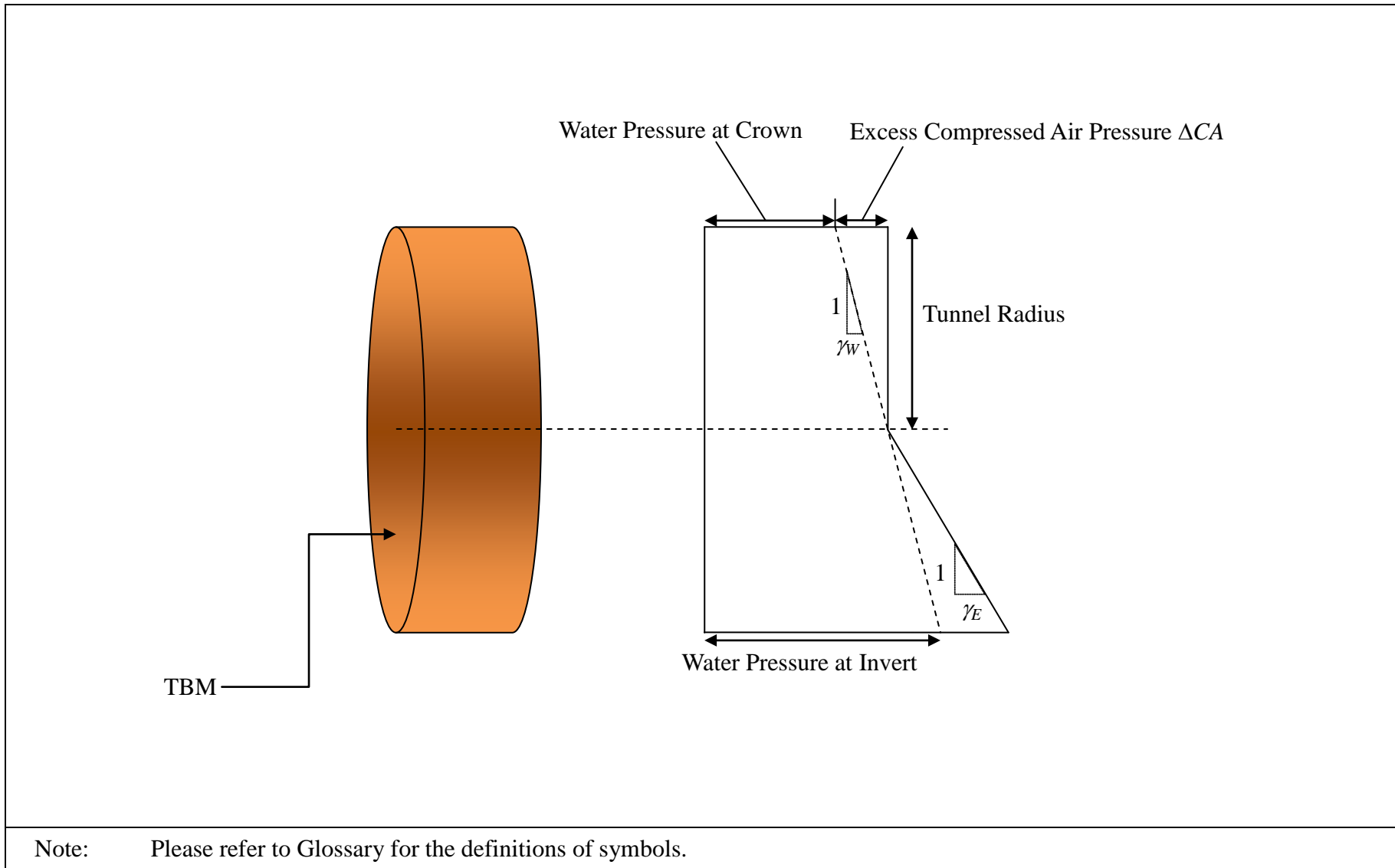
Figure 3.6   Idealised Pressure Distribution in Semi-EPB Mode, Based on the Spoil Level at Tunnel Axis Level

Note: Please refer to Glossary for the definitions of symbols.

3.8.4 De-oxygenated Air

As the compressed air passes through the ground it can become de-oxygenated. This is a safety risk if the compressed air enters a confined space, such as another tunnel or a basement. It is important to check the oxygen levels of such confined spaces before entry after tunnelling.

3.8.5 Risk of Encountering an Open Path

If the EPB TBM operates in semi-EPB mode, and an open path is encountered, the compressed air will escape much more rapidly than normal, in the same manner as an airlift. If the rate of loss of air is greater than the available rate of supply, this will lead to a loss of face pressure and, potentially, a major loss of ground. Most open paths are man-made, such as unsealed boreholes, instrumentation (such as standpipe piezometers or inclinometers), wells, some types of pile, or where piles or sheetpiles have been removed. It is important, before starting tunnelling, to carry out a detailed desk study and site reconnaissance of the locations of such items on or near the tunnel route, and to take measures to seal the paths before the TBM approaches. It is also important to remove any of the installations that could obstruct the tunnelling.

3.9 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 procedures 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. Based on this data the target face pressures for the following week can be issued.

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. All this data should be reviewed in conjunction with the corresponding instrumentation and monitoring data from the surrounding ground, surface and structures.

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

3.9.1 Adjustment of Target Face Pressures

A regular review of the target face pressures should be carried out daily.   The site and supervisory staff directly involved in the tunnelling should meet 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 review meeting to assess whether the basis for the target face pressures needs to be adjusted.   This 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 system records, monitoring or observation,
>
> (c) any deviation from the planned ranges of grout volume and pressure,
>
> (d) any new information on ground or groundwater conditions,
>
> (e) construction or excavation works in the vicinity of the tunnel alignment,
>
> (f) instrumentation readings,
>
> (g) observed ground behaviour in the face,
>
> (h) measured ground movements and the settlement of buildings, structures and utilities, and
>
> (i) the actual variations in the pressure applied.

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

3.9.2 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 EMC system,

(b) loss of face pressure during ring building, when the TBM is stationary, and

(c) adverse monitoring data, such as significant change in piezometric levels near the areas of tunnelling and excessive ground settlement.

The face pressure will need to be adjusted if there is evidence of face instability, over-excavation, piezometric pressures outside the range assumed, excessive settlement/heave or increasing/reducing pressure in the excavation chamber.   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 authorised persons for modifying the adjustment of the confinement pressure must be clearly defined in the work procedures for TBM operation.   The work procedures should identify critical observations and the decision making process for responding to those observations.   If an AFS system is installed, its use and trigger pressures should be included in the work procedures.

Equipment designed to inject bentonite slurry from a reserve into the crown of the cutterhead, if face pressure falls below a set value, is commonly specified.   Concurrent with this, the discharge gate closes and the advance stops - all to maintain face stability and prevent blow outs or sink holes.   This equipment is also used for supplying bentonite during a TBM stoppage.   During weekend stoppages (if any), foam is degrading and bentonite slurry has to be injected into the chamber to compensate properly for the drop in level and maintain the proper face pressure.

3.10   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 work procedures for the tunnelling.   Examples of work procedures are given below for reference, but they need to be adapted to the particular circumstances on each project.

3.10.1   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 may be deemed necessary:

  (a) where the tunnel is close to or at a major interface between different geological units,

  (b) where there is shallow ground cover,

  (c) where the ground level changes suddenly or where the piezometric levels fluctuate significantly,

  (d) where there are major underground structures (i.e. bored tunnels, underpasses or culverts) over or under the tunnel,

  (e) where there are piles over or adjacent to the tunnel,

  (f) where there is a significant (e.g. > 0.1 bar) difference in the calculated pressure between adjacent sets of calculation,

  (g) at the break-in and break-out, and

  (h) 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.

3.10.2 Presentation of Target Face Pressures for Regular Review during Tunnelling

The focus of the regular design and management review of target face pressures is typically on the next one to two weeks of production. The operators and other staff in the tunnel are typically provided with this information along with a daily briefing for at least the next one to two days of production noting any adjustments required following the data from the recent excavation. The tunnel manager, or a key staff member designated by the tunnel manager, provides to them a simple summary of the target face pressures together with the other key target operating parameters, such as the screw operating speed, pressure for the sensors along the screw conveyor casing, maximum torque, grout volume and pressure, etc. 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. Target face pressures, as well as the compressed air pressures to be used at intervention locations should be included in the daily briefing sheets as well as the ring by ring target data for the TBM operator to work to, based on the results of the daily meetings reviewing the TBM performance and settlement figures, as well as the expected ground conditions ahead. This daily information should include as a minimum:

(a) anticipated ground conditions, including piezometric levels,

(b) any other expected obstacle or structures,

(c) target face pressures,

(d) minimum face pressures,

(e) proposed intervention locations and pressures,

(f) conditioning details,

(g) grouting pressures,

(h) grouting volumes,

(i) other key TBM operating parameters,

(j) anticipated location of key interfaces,

(k) the location of buildings, structures, tunnels, roads and utilities over or close to the tunnel,

(l) the location of subsurface instrumentation and known boreholes and piezometers, and

(m) the location of piles, known/suspected obstructions or wells on or close to the line of the tunnel.

The information can provide a ready reference for planning of the work and when assessing the appropriate response to any problems during the tunnelling. The target face pressures summarised in the sheet should be reviewed (Section 3.9.1), and adjusted as necessary, based on the experience gained during tunnelling.

3.10.3 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 one to two days of production. The tunnel manager, or a key staff member designated by the tunnel manager, provides to them a simple summary of the target face pressures together with other key operating parameters, pressure sensors along the screw, maximum torque, grout volume and pressure, etc.

3.11 Why the Target Face 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 errors 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 face pressures are misread,

    (c) the operator deliberately reduces the pressure below the target face pressure, for example, to improve the speed of tunnel advance,

    (d) the operator is not sufficiently experienced to maintain the face pressure within the minimum/maximum planned pressure range (the target face pressure plus or minus the variation in pressure used in the design calculations), and

    (e) the pressure sensors are not correctly set or maintained (sensors must be checked and cleaned during each intervention).

Mechanical and other problems could result in inadequate (or excessive) face/compressed air pressure, for example:

    (a) The discharge gate should close automatically if the face pressure drops below a preset figure. This should be built into the programmable logic controller (plc) controls and logic of the TBM controls. If there is a boulder or such preventing closure, the gate will need to be momentarily released to clear the block and then closed again immediately.

    (b) There is a failure/breach of the main pressure bulkhead due to excessive abrasion.

    (c) There is an excessive wear on the screw conveyor.

    (d) There is a power failure on the machine not allowing the discharge gate to close. It is essential to specify that discharge gates are fitted with a fail-safe closure mechanism (accumulator) to prevent this.

    (e) The pressure sensors are malfunctioning, leading to incorrect information on the face pressure. Most machines are fitted with multiple sensors and hence a single sensor failure should be easily identified and corrected.

The risk that the target face pressures are not achieved 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, organisation, 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.

3.12   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 EPB 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.

The number and locations of pressure sensors in the excavation chamber and the screw conveyor need to be considered and agreed during the design stage of the TBM. A minimum number should be specified by the client. Consideration should be given to their positions to monitor the pressure distribution of the excavation paste at the face and along the extrusion path, bearing in mind the possible effect of heterogeneity of the paste on the face pressure applied and the need to have sufficient reliable data for the control of the risk against variable face pressure. A level of redundancy and the ability to replace these sensors should be considered in the TBM specification and design.

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 100, the face at this point will not be at the location of ring 100, but at the location where, say, ring 105 to 108 (depending on the relative length of the ring and the TBM) will later be built (Figure 3.7). 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.

TBM

Direction of
Tunnelling

Ring 108

Rings Built

Ring Being Built –
Ring 100

Rings Built
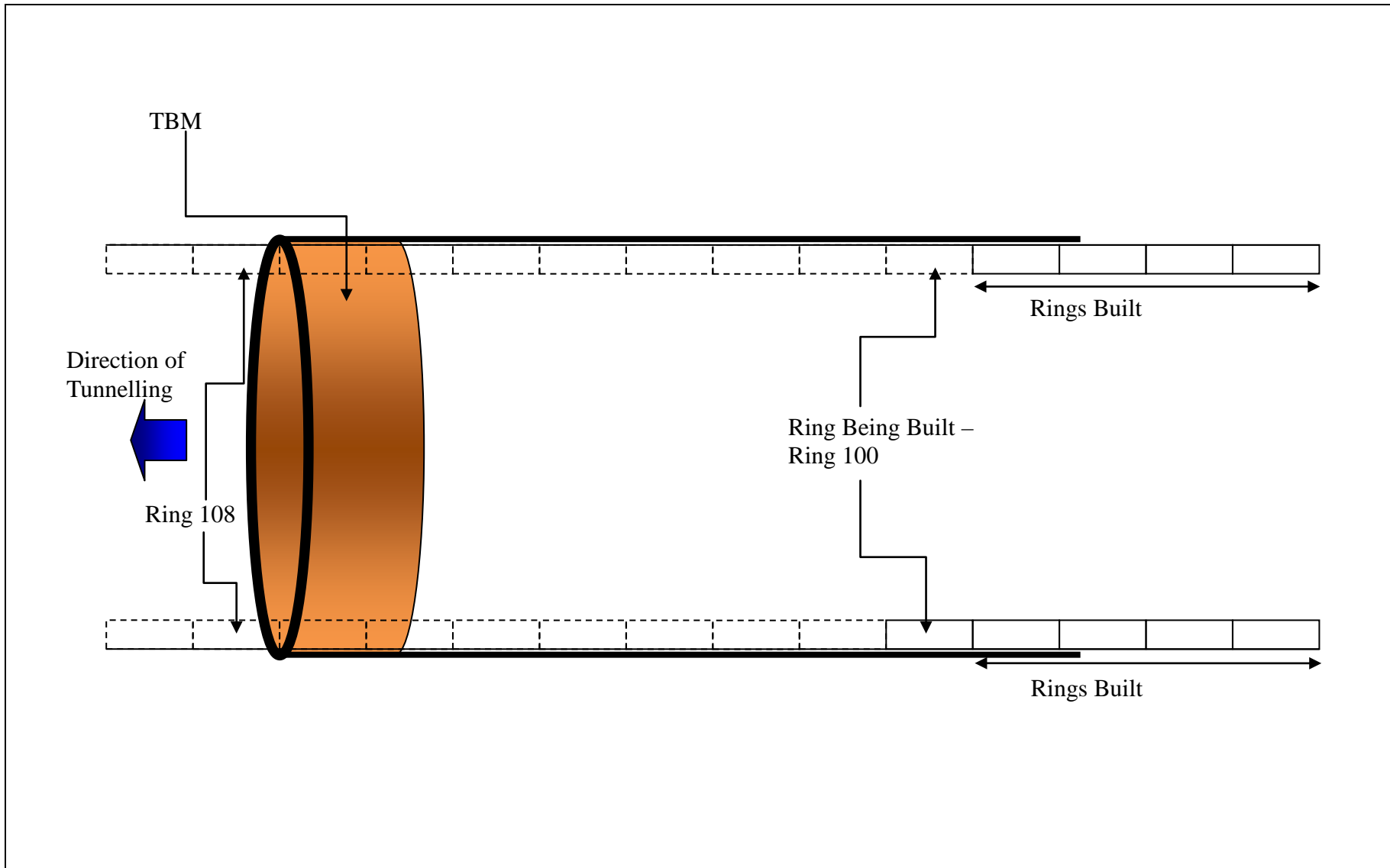
Figure 3.7   Section View of a Tunnel Showing Ring Numbers

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 the geological interpretation accurate and has there been enough desk study and 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? Is there enough permeability test wells carried out to allow the assessment of groundwater flow?

(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 those due to the removal of temporary works along the tunnel alignment 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, do the piezometric pressures rise as the face is driven past, reflecting the higher than hydrostatic pressures calculated for the target face pressures?

(f) Is there any performance data on EPB 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 EMC system consistent with those for the face pressures used and settlements recorded?

(h) Are the results of the settlement monitoring above the tunnel within the predicted range?

4 Screw Conveyor

The screw conveyor of an EPB TBM is the key component to regulate and maintain the confinement pressure and to achieve the controlled pressure drop along its length. It

controls the discharge of the excavated material and uses the shear resistance of the excavated material to counter the earth pressure at the bottom of the screw.   The excavated material then drops onto a belt conveyor running the length of the TBM back-up where it is removed by further conveyors or muck wagons.

The discharge gate is a guillotine gate, the opening of which can be varied for achieving the required face pressure during excavation.   Discharge gates must be fitted with a fail safe automatic closure mechanism in case of a power failure or a blow-out through the screw conveyor which should also be designed to close automatically when the face pressure drops below a pre-set value which is changed as the tunnel drive progresses according to face pressure requirements.   The use of a double discharge gate reduces the risk of it failing to close due to being blocked and is an option which should be considered in the TBM specification and design.

It is common to utilise wear protection plates on the screw conveyor.   These can either be welded into place or of a bolt-on type to allow them to be replaced more easily.

By varying the rotational speed of the screw conveyor the face pressure can be maintained.   A number of variations of the screw conveyor have been developed.   These are described below and several of them are shown in Figures 4.1 to 4.5.

The face pressure that a particular TBM can exert depends largely on the design of the TBM, although the nature of the (conditioned) spoil also has an effect.   The practical design of screw conveyors is a key component in determining the limiting pressure under which EPB TBMs can operate.   Other components of the TBM have to be designed to ensure that the machine can be operated at the maximum planned face pressure, plus a margin for safety/unforeseen conditions, including:

(a) the pressure bulkhead,

(b) the seals for the main bearing,

(c) the soil conditioning system, and

(d) the thrust rams, for which the face pressure is one of several resistances that have to be overcome.

The screw conveyor(s) have to be designed to maintain the difference between the pressure in the excavation chamber, at the inlet to the conveyor, and the pressure at the outlet from the system (the discharge gate).   Generally, the earth pressure reduces along the flight of the screw conveyor until it effectively balances with atmospheric pressure at the discharge gate.   However, a few machines have incorporated systems which allow controlled discharge at a pressure greater than atmospheric, thus reducing the difference in pressure.   The pressure drop that can be achieved by the screw conveyor is largely determined by the number of flights along the screw, which in turn is approximately related to the length of the screw conveyor.   However, the pressure difference is also affected by the type of screw (centre shaft or ribbon), the number and arrangement of the screw conveyors used, whether there is excessive wear, etc.

4.1 Types of Screw Conveyor

4.1.1 Centre Shaft Screw Conveyor

This is the most common type of screw conveyor, used on the majority of EPB TBM. It is effectively an Archimedean screw inside a tube with a drive at the upper end.   This type of screw is the preferred design for all soils except where the size of boulders (or rock fragments) anticipated is too large to pass up the flights (Figure 4.1).

Figure 4.1   Centre Shaft Screw Conveyor (Courtesy of Herrenknecht AG)

4.1.2 Ribbon Screw Conveyor

Where there is a risk of obstructions such as cobbles and boulders that would not be able to pass up the flights of a centre shaft screw conveyor, a ribbon type screw can be adopted.   The larger openings in the ribbon screw allow boulders up to 60% of the screw diameter to pass through the conveyor.   However, ribbon screws have limitations in the amount of torque which can be applied to them.   They require a peripheral drive and can be susceptible to blow-out through the centre of the screw.   Ribbon screw are less effective at dissipating the face pressure along the length of the screw and so are less suitable where there is a high hydrostatic/piezometric head.   They have torque limitations due to the torsional strength of the ribbon (Figure 4.2).   To cope with large boulders which may arrive at the discharge gate, the gate has to have sufficiently large openings to discharge the boulders.

Figure 4.2   Ribbon Screw Conveyor (Courtesy of Richard Lewis)

4.1.3   Twin Screw Conveyor

On some larger diameter EPB TBMs a pair of side by side screw conveyors have been used to effectively double the excavation rate while maintaining the same face pressure control.   Such options are only suitable in ground conditions where large boulders or other obstructions are unlikely to be encountered.   It is more common to increase the diameter of a screw conveyor to suit the ground conditions as the control systems of twin screw conveyors are complicated (Figure 4.3).
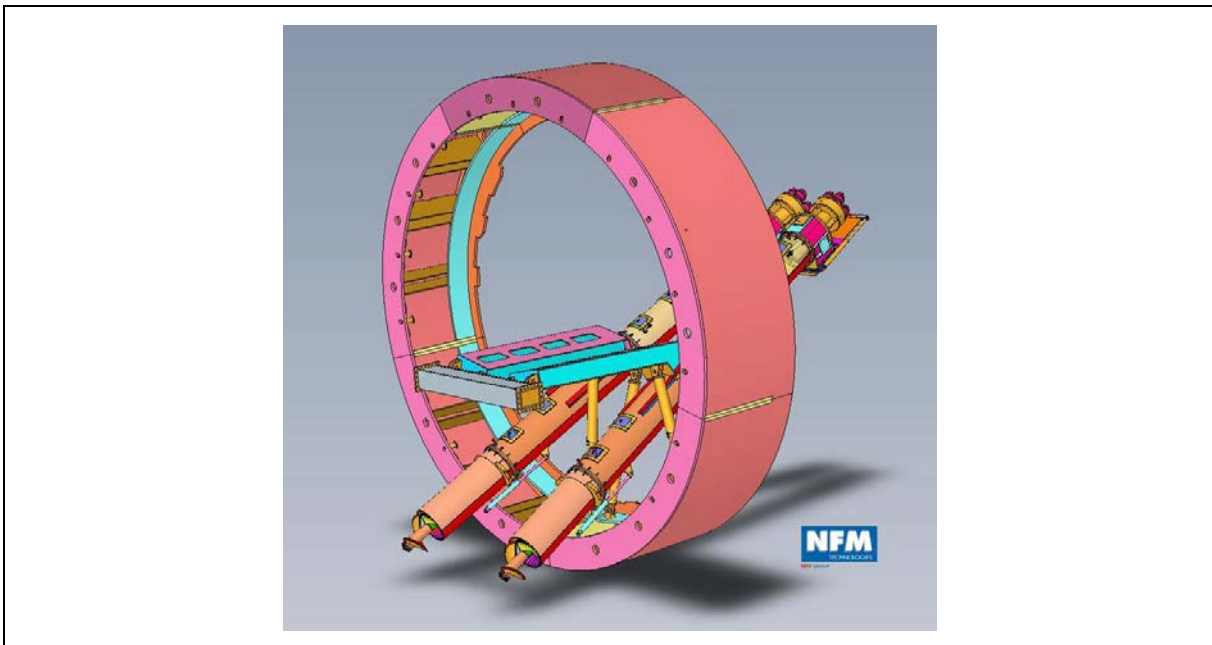
Figure 4.3   Twin Screw Conveyor (Courtesy of NFM Technologies)

4.1.4   Articulated Screw Conveyor

Some machines are fitted with a centre shaft or ribbon screw conveyor inclined up to the crown of the tunnel, which allow boulders to travel up the shaft, where they are disposed of through a boulder collecting gate (removing boulders without losing pressure is not simple and requires enough space).   This is followed by an articulation joint beyond which a second centre shaft screw conveyor is mounted horizontally in the crown of the tunnel.   This second screw can be of any length required, allowing greater earth pressure to be dissipated smoothly (Figure 4.4).   Articulated screw conveyors that have both lengths as centre shaft screws are more common.   These are used to increase the length of the screw conveyor for pressure reasons, or to lift the screw conveyor higher through the build area.   Forward and rear sections of the auger can be joined by a universal joint to allow a single drive, or they can have separate drives in which case they become a two-stage screw conveyor.   Some screw conveyors are articulated at the forward end to allow the rear discharge points to be maintained directly above the belt conveyor on the first gantry.   This feature is common where the TBM has to negotiate tight curves.
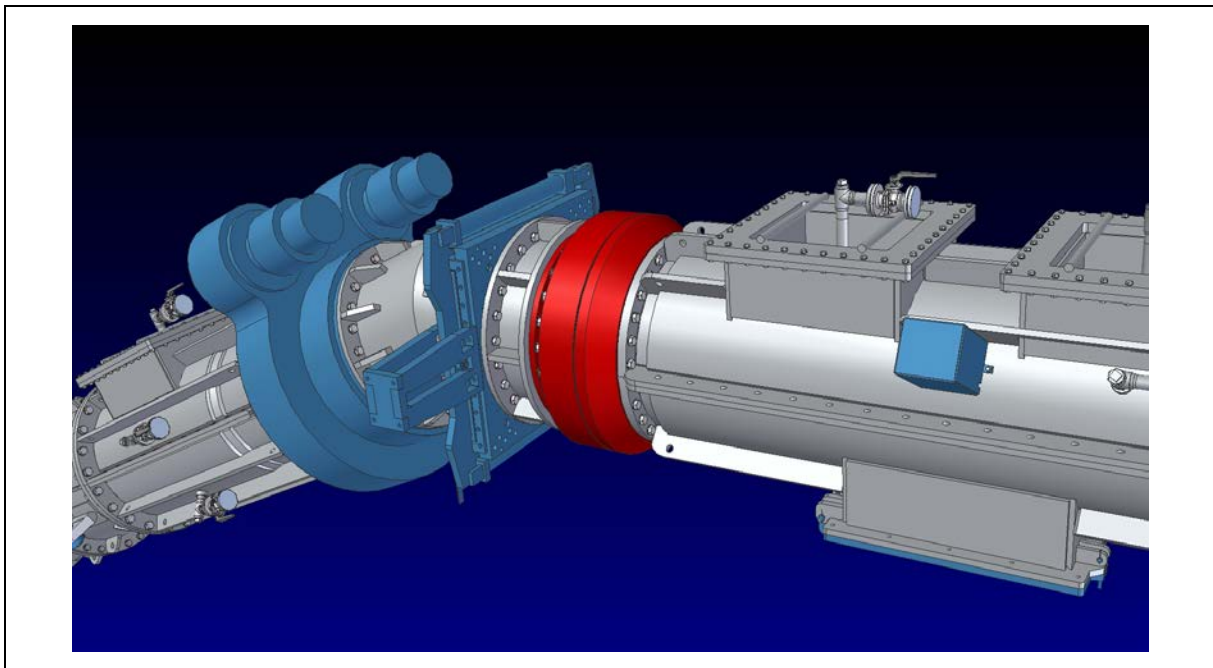
Figure 4.4   Articulated Screw Conveyor (Courtesy of The Robbins Company)

4.1.5   Two-stage Screw Conveyor

A number of different configurations of two-stage screw conveyor systems have been used to provide greater resistance to earth pressure and greater control over the effective face pressure by combining a second, shorter, counter-rotating screw in series to the primary screw. Peripheral drives are required for the forward screw.   Japanese manufacturers have experimented with two-stage screw conveyors which vary the rotation rate automatically with the variation in face pressure.   This type of screw conveyor has been used successfully for both soft cohesive clays, in sands and gravels, and for deeper tunnels in cohesive soils

(Figure 4.5).   One of the advantages of a two-stage screw conveyor is that by varying the speed of the second screw compared to the first screw, a sand plug of compressed material can be created between the two screws, thus improving the overall resistance of the double screw to pressure.
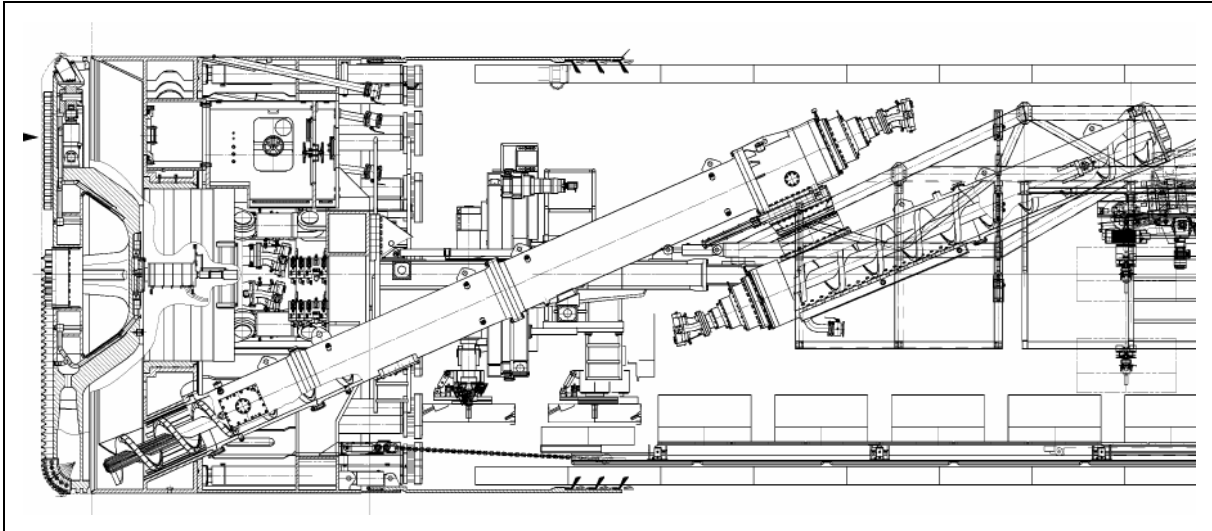
Figure 4.5   Two-stage Screw Conveyor (Courtesy of Herrenknecht AG)

All screw conveyors should include access ports for unblocking, ports for injection of conditioning agents at various points along the length, pressure sensors at each end of each stage as a minimum, and preferably in the centre as well.   There should be wear protection along the length, the first 5 - 6 flights being the most important.

Excessive wear of the screw conveyor will reduce the effectiveness of the screw conveyor as a critical part of the face pressure control system (Figure 4.6).   The design of the screw conveyor needs to give due consideration to the abrasiveness of the ground.   Items to be considered include, inter alia:

> (a) additional wear protection,
>
> (b) facility for inspecting the screw conveyor for wear without removing it from the casing, and
>
> (c) facility to remove and maintain/replace the screw conveyor without risk of loss of ground.

The ability to withdraw a screw conveyor and close a guillotine gate at the entrance to the screw in the excavation chamber bulkhead should be included in the TBM specification and design to allow maintenance of the screw conveyor while maintaining face support pressure.   Careful consideration should be given to the maintenance method if this feature is not included in the TBM design.

Figure 4.6   Example of Excessive Wear on Screw Conveyor

5   Conditioning Agents

In EPB TBM technology, the correct use of soil conditioning agents is essential for a successful and efficient TBM drive.   It is important to form a paste of a suitable and consistent density and viscosity.   Jancsecz et al (1999) and Thewes (2007) outline the range of soil grading that can be conditioned for EPB TBM tunnelling.   With the future development of EPB TBM technology and conditioning agents, the soil grading range of application of EPB TBM tunnelling will not be limited to soft clays, clayey-silts and silty-sands.   The aims of providing conditioning agents are:

(a) to enable the excavated material retained within the cutterhead to be workable by having properties (homogeneous, viscous, flow, permeability) that would allow it to pass easily through the cutterhead to the screw conveyor,

(b) to support the excavated soil and any rock fragments in the paste mix without segregation and settlement of the larger rocks to the bottom of the chamber,

(c) to make the paste effectively impermeable when it enters the screw conveyor,

(d) to reduce friction and therefore reduce the required torque and wear on the cutterhead and the screw conveyor, and

(e) to make the paste able to transmit the pressure from the bulkhead onto the face to support it.

There are many products that have been used and are still useful.

Plain water is commonly used. However, for many soil conditions, a pure bentonite slurry is used. These were the only additives in the early days of EPB machines, following which polymers were used. The main disadvantage of water-based fluids is that if the quality control is not maintained properly the material can become too fluid when it emerges from the screw conveyor. It can be too wet for handling easily on the belts and for disposal. Foams were developed because they impart fluidity to the material, without making it wet. Thus, as the bubbles collapse on the belt conveyor and the air escapes from the spoil, the remaining material is relatively dry. However, there are still applications where water, bentonite or polymer is more effective than foam.

Polymer is added to foaming solutions to give strength and stability to the bubbles in the foam, i.e. as a stabiliser. Polymer is also useful to reduce the friction or adhesion between the paste and the side walls of the excavation chamber, especially if there is a lot of clay in the spoil.

Conditioning agents can be broken down into a number of major types, as described in the following subsections.

5.1 Foaming Agents

The main demand of foam as a conditioning additive is to create a paste, to build up and maintain the necessary face support pressure in the excavation chamber, and to minimise pressure variations. It creates small bubbles (foam) in the excavation paste, and is also used to obtain a suitable rheology. The reduction in torque and abrasion is a very important additional effect. Foam can be created out of a turbulent mixing of a surfactant solution and air. The main surfactant properties are:

(a) fluidising effect on soils because of the decrease in surface tension, and as such the soil particles are no longer bound to each other by linked water, and

(b) electrostatic repulsion effect which can separate two particles attracting each other by electrostatic forces.

Surfactants are a combination of a hydrophobic chain and a hydrophilic head. Both parameters can be varied. Each soil type, from stiff clay to sandy gravel, requires its own type of foam to reach its best effectiveness. The type of surfactant which should be used for a special site has to be determined by laboratory tests with the insitu soil. Three important foam parameters are:

(a) concentration of surfactant agent in the foaming solution,

(b) ratio of mixing foaming solution with air (to create a foam), and

(c) ratio of mixing foam with soil.

5.2 Polymers

In addition to their foam stabilising effect, there are two main functional types of polymers:

(a) water binding polymers to dry out (liquid) soils, and

(b) soil structuring polymers which are useful in loose, coarse soils to change the excavation paste properties and which prevent sedimentation.

Some polymer developments are based on hydrocarbon chains and are produced by bacterial fermentation. These polymers are water soluble, biodegradable and compatible with the foam surfactants. They are safe for the foaming generator. In consequence, they can be mixed with the foaming solution and passed through the foam generator. Polymers can induce a more stable support pressure in the excavation chamber during the tunnel drive and when stopping the machine for a short time. All polymers should preferably be in liquid form to avoid dosing problems.

5.3 Anti-clogging Agents

EPB TBMs were originally designed for soils with at least 30% of fines (Thewes, 2007). However, many clays have a tendency to stick to the cutterhead and affect the cutting efficiency of the TBM. Anti-clay polymers carry a high charge density which separates the soil particles, and creates an electro-chemical barrier which avoids binding/sticking effects. Surfactants can also fulfil these functions.

5.4 Abrasion Reducing Agents

Anti-abrasion additives are used in highly abrasive soils or rock excavation. They are designed to protect the cutterhead, the tools mounted on the cutterhead and the screw conveyor. Anti-abrasion additives are generally introduced directly into the cutterhead, in the mixing chamber or in the screw conveyor.

Other conditioning agents, such as high-density slurries, have been used to deal with specific ground conditions. Specialist advice and local geotechnical knowledge should be sought when planning a project to ensure that the design is as comprehensive as possible.

Conditioning agents from different suppliers should be tested for each project, as the effectiveness of the chemical will be affected and altered in each situation by the natural minerals occurring in the ground and in the local groundwater. Site and laboratory testing is essential to ensure that the desired paste properties can be achieved and therefore should be specified. Confinement pressure can have a significant effect on the performance of foaming agent and therefore insitu results should be compared to the site and laboratory testing at an early stage of the excavation, at any changes in geology, and periodically along the drive to ensure the expected performance is being achieved.

Important parameters and considerations include:

(a) the concentration of the conditioning agent as purchased,

(b) the dilution ratio with water to achieve the desired quantity of usable chemical,

(c) the injection ratio as a percentage of solid ground excavated, for foams, this is referred to as the FIR (Foam Injection Ratio),

(d) for foams, the FER (Foam Expansion Ratio), and

(e) some polymers are also anti-clogging agents, which are useful when sticky clays are excavated.

The FIR/FER will need to be estimated very early in the project, as these will determine the design capacity of the air compressors and receivers required on the TBM backup.

Although primarily a means of adjusting the properties of the excavation paste to make it suitable for EPB tunnelling, the use of soil conditioning also has other beneficial effects. These include a reduction of torque to both the cutterhead drive and the screw conveyor drive, and a reduction on abrasive wear from the material to the steel components in contact with it (cutterhead, shield, cutter tools and screw).

The best place for injection of foam, polymer or other conditioning agents is through nozzles in the cutterhead face fed through the cutterhead arms. This enables the conditioning agent to be mixed with the excavated soil from the very moment it is excavated by the cutterhead. Injection ports are also fitted to the pressure bulkhead but these are less effective. It is also important to have some injection ports on the screw conveyor so that if the screw torque increases the material can be softened. In this respect, it is better to introduce the conditioning agent near the entrance to the screw conveyor.

Nozzles on the cutterhead should be spaced so that each nozzle covers an approximately equal swept area, thus radially the spacing is slightly closer together near the outside of the face. However, the central area also needs an extra injection port as the mixing action is reduced toward the centre of the head and clogging is more likely here. Spacing of nozzles on the cutterhead should be approximately 1 m apart radially.

Injecting agents through the rotating cutterhead arms requires a rotary joint on the TBM. The port through this joint should be large enough to allow passage of the foam. Each port going to each injection port should have its own pump. If nozzles are piped together and fed from a single pump all the foam or polymer will pass through the port with the least resistance and the other ports will receive nothing. Moreover, they will become blocked very quickly. It is important to have a rotary joint with several injection lines in order to have the ability to control the injection conditioning differently for each port and to provide redundancy should one port/line become blocked or damaged.

The injection of the conditioning agent should be monitored carefully and the volume injected has to be considered when the excavated volumes and weights are being measured for comparison with theoretical and for assessment of over excavation.

6 Tail Void and Shield Skin Grouting, Limiting Pressures and Volumes

Tail void grouting is essential, both to minimise 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 TBM 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.   EPB 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.   Providing grout pipes, which are laid within the tail skin and behind the tail seals, can do this.   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 affected by losses into the surrounding ground by permeation or over excavation and by bleeding of the grout.   It is therefore normal to inject more grout than the minimum theoretical volume of the tail void (annulus).   Typically the injected grout volume will be at least 20% above the theoretical annulus volume.

It is normal to define, for the tail void grouting: a minimum volume, a minimum pressure and a maximum pressure.   The work procedures 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.   The effective over pressure will also be affected by the number of grout ports available around the shield perimeter.   It is preferable for the injection pressure to be slightly greater than the face pressure in the head of the machine, as the excavated material can connect to the tail void by passing around the machine through the overcut.   This effect is less likely in EPB machines as the overcut tends to be smaller and the excavated material is less liquid than in a slurry TBM.   Typically, the minimum grout pressure is set at 1 to 2 bars over the target face pressure, due to the location of the grout pressure sensors.   It is useful to carry out a trial for measuring the pressure loss along the grout lines.   This can also be checked with an instrumented section (trial plot).

For shallow tunnels, or for tunnels in soft clay, this may equal or exceed the overburden pressure at the crown.   In addition, for shallow tunnels (with a ground cover less than two tunnel diameters), the pressure should be assessed very carefully and in most cases should be much less than 1 bar over the target face pressure.

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 a key 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 of compressible soils.   Effective tail void grouting is also essential to maintain the build quality of the completed segmental lining which also reduces long term settlement due to squatting of the rings.

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.   Tail seal grease is generally measured by volume used as the TBM progresses.   The pumps inject 'shots' on a timed basis.   Pressure is measured in the pump line, not at the tail seal.   Whilst the pressure in the grease should be above the grout pressure, it is not something which can practically be adjusted.   Measured pressures (in the supply line) typically vary throughout a shove between 2 bars and 40 bars depending on the timing of the shots and the pressure measurement.   The main objective is to keep the tail seal full of grease, as a thin layer, typically 0.8 - 1.0 mm thick depending on the roughness of the segment extrados, deposited on the outside of the lining as the TBM advances.

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 is a key factor in the development of consolidation settlements over the tunnel (Shirlaw et al, 1994).

Reverse spring plates on the extrados of the tail shield are recommended to prevent grout migration along the extrados towards the cutterhead.

6.1 Types of Grout

Types of grout can be sub-divided into three categories:

(a) Active - Grouts containing high percentages of Portland cement causing full hydration of the grout material.

(b) Semi-inert - Mainly composed of the same elements as found in the Inert type with the addition of a small percentage of a material that will cause some degree of hydration to occur, e.g. < 100 kg/m^3 cement. The grout may take a considerable period of time to harden although stiffening will occur more quickly.

(c) Inert - The system contains no Portland cement, mainly hydraulic lime and/or flyash and therefore only stiffens slightly over time. It provides initial stiffness due to a mechanical lock up of the mix. It is important to maintain the mix and avoid any segregation during handling, as this can lead to no mechanical lock up and float of the tunnel. The rheology of this grout (shear strength and yield value) has to be defined in order for the grout to be able to support the encountered load.

The selection of the type of grout will depend on the particular tunnel requirements, such as the risk of settlement, the allowance for future deformation, effective down time of the machine (dayshift only operation) and the soil conditions.

A second sub-division of grouting types is single or two component grouts.

6.1.1 Single Component

A single component grout is a mixture of sand, a binder (usually cement or cement and lime), water, and usually a workability additive. The material must be capable of being workable for a long enough period from mixing to injection and be capable of being pumped into the annulus while at the same time providing properties to avoid the segmental lining moving, distorting or floating as the TBM advances. This can be difficult to achieve and depends heavily on the types of sand available locally to the project.

6.1.2 Two or More Components

Two component grouts are generally described as A/B grouts where Component A is a stabilised mortar filler and Component B is an activating liquid (typically sodium silicate) which is mixed with Component A at the point of injection.

Two component grouting can give better results so far as filling of the annular void is concerned, but requires more sophisticated equipment. Pipe blockages can occur at the injection nozzles if the equipment is not operated and maintained correctly. The risk of blocked grout pipes can result in inadequate grouting, which can lead to increased settlement, ring deformation and cracking due to unbalanced loading. Site trials should be undertaken to demonstrate that the mixed two-part grout can penetrate throughout the tail void before gelling occurs.

6.2 Shield Skin Grouting

To control ground movements into the overcut gap around an EPB shield, in particular for EPB TBM with a step shield configuration where the volume of the annular gap increases as the TBM advances, a system to allow injection of bentonite around the shield skin should be provided. Injection of bentonite may not be required in loose or soft ground if the paste formed from the excavated material can continuously fill the annular gap during the advance of the TBM. However, for EPB TBM tunnelling in granular or unstable ground, the excavation paste may not continuously fill the annular gap. Therefore, injection of bentonite may be required in granular or unstable ground to minimise the ground movement.

7 Excavation Management Control System

The net volume of the ground removed during excavation can be calculated by measuring the volume of excavated material mucked out, and subtracting from it the volume due to bulking and the volume of soil conditioning agents added. This can be compared with the theoretical excavated volume calculated from the TBM cutterhead diameter and the stroke length. The net weight of the ground removed can be assessed by measuring the weight of excavated material mucked out and subtracting from it the weight of soil conditioning agents added. This can be compared with the theoretical weight of excavated material calculated from the theoretical excavated volume and the insitu bulk density of the excavated material. Such calculations and comparisons provide the basis for assessing whether there has been significant over-excavation at the face of the machine. While, in principle, this is simple, there are a number of practical issues that must be considered, including the accuracy of the volume or weight measurement and the accuracy of the bulking factor and insitu bulk density of the excavated material used. Due to variability of materials along the tunnel drive, the effects of bulking of the material excavated and the difficulty in accurately measuring the insitu density of the excavated material, it is difficult to measure the actual volume or weight of material excavated to within 10% - 15%.

7.1   Accuracy of the Volume and Weight Measurements

Laser scanners are the direct way of continuous monitoring of the excavated volume while belt weighers are the direct way of continuous monitoring of the excavated weight.

Even with regular calibration, the accuracy of the calculations can only be within about 10% - 15%. This can introduce significant variation in the 'measured' net volume or net weight of material mucked out. The longer the excavation cycle, the greater the potential error becomes.

The measurement of volume or weight can also be affected by gains and losses to the system. Gains can come from groundwater entering the excavation chamber (i.e. inward seepage). This should only happen if the face pressure is lower than the groundwater pressure. Errors can also occur due to loss of additives into the ground or to the surface, and during maintenance and head access where bentonite slurry is used to prepare the excavation face for the maintenance interventions.

7.1.1   Volume of Additives

It is essential when calculating the volume of material discharged that account is taken of the effects of any additive used. The potential loss of additives into the surrounding ground which does not come back through the screw conveyor should also be considered in the overall volumetric or weight calculations.

7.1.2   Belt Weighers and Belt Scanners

Belt weighers and laser scanners/profilers can provide continuous measurement of spoil quantity (BTS/ICE, 2005).

Belt weighers can suddenly malfunction, giving a spurious reading and causing unnecessary concern over apparent over-excavation. It is becoming more common to specify that two belt weighers are used simultaneously, on the basis that it is unlikely that both will malfunction at the same time.

The use of belt weighers to measure the weight of material excavated and laser scanners to measure the volume and hence deduce the density of material on the belt conveyor is the best method of monitoring the spoil quantity. Regular cross-checking via sampling and calibration of the belt weighers and laser scanners should be undertaken.

7.2   Assessment of the Volume and Weight of the Material Excavated

There is a significant difference in the weight of 1 m^3 of soil and 1 m^3 of excavated rock.   This is a function of the way in which the TBM cutterhead has been tooled to excavate the ground and an accurate assessment is essential for the design of the tunnel mucking system. It 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 metre 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 and the moisture content will be constantly varying, and so the insitu bulk density of the excavated material per ring will also vary significantly.

7.3   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 EPB TBM 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 EMC system is a primary warning that significant over-excavation may be occurring, and is an essential part of EPB TBM tunnelling, particularly in an urban environment.   By plotting the cumulative net volume (or net weight) of material excavated against the theoretical volume (or 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 system 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 materials discharged

> from the screw conveyor, results of cross checking using the
> belt weigher and laser scanner data, and the instrumentation
> and monitoring results (including measured volume losses)
> and other observations, before deciding whether significant
> over-excavation has in fact taken place.

Systems are available for real-time remote measurement of the TBM parameters such as torque, thrust, RPM, advance rate, face pressures and grout volumes. Continual assessment of these parameters, together with the plots of deviations from the theoretical volume (or weight) of material excavated, can indicate when over-excavation is occurring, and allow the appropriate control measures to be taken expeditiously.

The results of the settlement monitoring on the surface are important and should be used in the daily and weekly meetings referred to in Section 3.9 when assessing TBM performance and face pressures. Many urban projects require the use of real-time monitoring using remote sensing instrumentation to provide continuous data which is processed instantly and provide very timely and useful data to detect trends or provide early warnings of unpredicted ground movement. With traditional manual survey monitoring, by the time the recorded data has been processed, there is often little or no time to take action to mitigate the effects. Major over-excavation typically appears at the 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.

8 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.

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 8.1, which lists the high risk activities and a number of possible mitigation measures. The table is based on Shirlaw et al (2005). The mitigation measures

listed in Table 8.1 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.

Table 8.1   High Risk Activities and Some Mitigation Measures That Can Be Considered for Reducing the Likelihood of Loss of Ground during EPB TBM Tunnelling (Sheet 1 of 4)

High Risk Activity	Mitigation Measures
Shield launch	• Where possible, select site location with low risk for the launch. • Provide a seal between shield and wall of launch shaft (Figure 8.1). • Test the seal pressure prior to launch beyond stable ground. • Provide minimum length of ground treatment - typically length of shield plus the length of at least 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 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, provide a contingency plan for blockages in the screw conveyor 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 screw conveyor. TaM tubes should be fully grouted to avoid creating a flow path to the surface.
Shield recovery	• Where possible, select site location with low risk for the recovery. • 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 without a support pressure. • Provide a seal between the shield and the wall of recovery shaft.

Table 8.1 High Risk Activities and Some Mitigation Measures That Can Be Considered for Reducing the Likelihood of Loss of Ground during EPB TBM Tunnelling (Sheet 2 of 4)

High Risk Activity	Mitigation Measures
Shield recovery (Cont'd)	• 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 screw conveyor. TaM tubes should be fully grouted to avoid creating a flow path to the surface.
Interfaces between stable & unstable geological units	• Provide detailed planning of face 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.
Mixed face conditions	• Provide detailed planning of face 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 or screw 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.

Table 8.1   High Risk Activities and Some Mitigation Measures That Can Be Considered for Reducing the Likelihood of Loss of Ground during EPB TBM Tunnelling (Sheet 3 of 4)

High Risk Activity	Mitigation Measures
Mixed face conditions (Cont'd)	• 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 damage. • Provide shrouds to protect the bearings of the discs from impact damage or clogging with soil.
Mechanical problems	• 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. • Provide discharge gates from the screw conveyor with a fail-safe closure mechanism that operates automatically in the event of a power failure. • Provide inspection ports along the casing of the screw conveyor, so that the screw can be inspected without removing it. • Provide facility so that the screw conveyor can be partially retracted, then hydraulically operated gates closed and a blanking flange inserted to seal off the opening in the main pressure bulkhead, so that the screw conveyor can be safely removed and replaced, if necessary. • Provide a wear detection system that includes selected: cutting tools, muck collecting buckets and the main cutterhead. • Mixing arms should be of substantial construction and provided with wear protection.
Chamber/head access (intervention)	• 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 face pressure will be replaced by compressed air pressure, and vice versa. • Provide compressed air locks (double air locks where the size of the TBM allows). Compressed air access to be available within 72 hours at all times.

Table 8.1 High Risk Activities and Some Mitigation Measures That Can Be Considered for Reducing the Likelihood of Loss of Ground during EPB TBM Tunnelling (Sheet 4 of 4)

High Risk Activity	Mitigation Measures
Chamber/head access (intervention) (Cont'd)	• Provide remote camera facilities in the cutterhead to allow the face stability to be checked before entering the excavation chamber, and also to monitor the team during the intervention. • 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, drainage, communications) 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 close head openings during intervention to minimise water inflow and provide ground support (not possible on most TBMs). • Carry out inspection of the exposed face at the start of the intervention, and at regular intervals during the intervention. • Minimise the need for long interventions. Generally, it is better to have more numerous, short, interventions to inspect and replace (as necessary) the cutting tools than to risk the much longer intervention(s) than may be needed if there is major damage to the cutterhead due to operating with worn or damaged cutting tools. For long drives in abrasive ground, a combination of relatively frequent interventions to inspect/replace cutting tools with occasional long interventions at intermediate shafts or at 'safe havens' (see above) can reduce the risk associated with the interventions.

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 (Figures 8.2 and 8.3), the part projecting ahead of the TBM will disturb the excavated face. Whether a boulder or 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.

Figure 8.1   Typical Launch Seal (Courtesy of Richard Lewis)

Figure 8.2   Example of Rock Fragment Trapped in the Cutterhead

Figure 8.3   Example of Rock Fragment Trapped in the Opening of the Cutterhead

Similarly, rock fragments, which are too large to pass through the EPB screw conveyor will get trapped and/or block the conveyor.   This may stall the screw conveyor, lead to excessive wear, damage or create a void in the screw conveyor, which results in deterioration of the pressure plug formed by the screw conveyor.   Ultimately this can lead to a blow-out through the screw conveyor.   To reduce the possibility of large fragments entering the screw conveyor, bars can be installed across the cutterhead openings to limit the size of fragments passing through into the excavation chamber.   Screw conveyors should be able to allow the operator to reverse their normal rotation to assist in freeing any blockage.   Work procedures to undertake this safely must be provided.

As discussed in Section 3.7, 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.   This type of machine with closure doors or plates is not common due to the constraints such systems place on the overall design of the cutterhead.

Prior to a compressed air intervention, part (usually) or all (occasionally) of the paste in the excavation chamber has to be removed, and the support pressure replaced, where necessary, by compressed air pressure.   When it is necessary or possible to form a filter cake on the face there are intermediate steps of introducing the bentonite slurry, over-pressurising the slurry, and then replacing it with compressed air.   At the end of the intervention the chamber has to be refilled, ready for the start of tunnelling in EPB pressure.   The excavation paste is only formed as the excavation re-commences.   The face pressure has to be

maintained above the minimum pressure required for stability, at all times, during these stages; this 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, investigated for 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 tunnelling, so that there is a default value available if an intervention is required.   However, assumptions regarding pore pressures should be reassessed prior to entry if other works have affected them (e.g. dewatering), emphasising the requirement for sufficient piezometers and on-going pore pressure measurement.   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 havens', to allow major maintenance and repair of the TBM.   The 'safe havens' 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 haven' for a major intervention.   As a general rule in abrasive ground, manned interventions should be carried out at fairly frequent intervals such as every 24 or 48 hours.   This is to confirm that no undue wear or breakage is taking place with the cuttertools.

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.   Face pressures and intervention pressures should be calculated for every ring based on the daily reviews and included in the daily excavation instruction sheet, sometimes called a permit to excavate.   Safe havens should also be considered where intervention pressures higher than 3.45 bars may be required.[2]   However, even if safe havens are prepared and used, there should always be a contingency plan for interventions in the case of emergency situations outside the safe haven.   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

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[2] A limit is specified under the Factories and Industrial Undertakings (Work in Compressed Air) Regulation 12 that no person shall be employed in compressed air at a pressure exceeding 50 pounds per square inch, which is equivalent to 3.45 bars, without permission from the Commissioner for Labour except in the case of an emergency.

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 8.1 provides examples of possible mitigation measures, which can be considered, to reduce the likelihood of loss of ground during TBM tunnelling. Another means to reduce risk is to mitigate the consequences in the event of a loss of ground. In Table 8.2, a number of possible mitigation measures for the consequences of ground loss are listed. Most of these measures have been used in practice, either on their own or in combination with the measures given in Table 8.1. The measures in Table 8.2 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 8.2 is not exhaustive, and other mitigation measures not listed in the table may be appropriate in particular circumstances.

Table 8.2 Some Mitigation Measures That May Be Considered for Controlling the Consequences of Excessive Loss of Ground during EPB TBM Tunnelling (Sheet 1 of 2)

Purpose	Mitigation Measures
Intercepting the ground loss	• 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. • Purging the trapped air with a dedicated system to prevent possible small collapses of ground at the crown caused by foam degradation (which could result in formation of air bubbles trapped on top of the excavation chamber).
Mitigating the effects - buildings	• 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	• 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. When a thickened road slab has been constructed, take positive steps to look for cavities below the road slab, which may not be noticed for some time after the TBM has passed.

Table 8.2 Some Mitigation Measures That May Be Considered for Controlling the Consequences of Excessive Loss of Ground during EPB TBM Tunnelling (Sheet 2 of 2)

Purpose	Mitigation Measures
Mitigating the effects - utilities	• 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. • Grout the ground beneath the utility.

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Glossary of Terms

Glossary of Terms

(A) General terms related to tunnelling, used in this report:

Extrados The outside face of a structural element, e.g. tunnel lining extrados.

Invert The lowest surface of a tunnel.

Overcut gap The annulus around the shield skin created by the difference between the excavated diameter and the outside diameter of the shield skin. The size of the annulus varies due to different diameters of different parts of the shield skin. It is also affected by steering of the shield, as well as by the degree of overcutting.

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.

Tail void The annulus around the extrados of the tunnel, due to the difference between the outside diameter of the lining and the excavated diameter. Also known as the tail gap or annular space.

(B) Terms relating to EPB TBM tunnelling, used in this report:

Additive Additives are materials that are added at the cutterhead, into the excavation chamber and/or the screw conveyor to improve the properties of the excavated material. The purpose is to alter the properties of the excavated material to form a suitable paste for EPB operation.

Auxiliary Face Support (AFS) system A system based around a tank of bentonite slurry within the TBM backup. The tank is linked to the excavation chamber, such that if the face pressure drops below a pre-set value, the slurry is automatically injected into the chamber to maintain the face pressure.

Bentonite annular injection TBM-mounted system to inject bentonite slurry around the shield in squeezing ground. Also to maintain the support pressure between the face and the annular grouting zone.

Conditioning agent Conditioning agents include foams, polymers, binders and bentonite slurry, either alone or in combination. Synonymous with additive.

Discharge gate The opening at the upper end of the screw conveyor where the excavated material discharges at atmospheric pressure.

Earth Pressure Balance (EPB) TBM An EPB TBM is a closed face tunnel boring machine. This type of TBM was originally developed primarily for cohesive soils, but has since been adapted for use in a wide range of soil and weathered rock. The excavated material is mixed, usually with injected additive(s), in the excavation chamber to form

a paste, which must completely fill the excavation chamber and is then discharged via a screw conveyor. The excavation paste is pressurised against the pressure bulkhead by the propulsion thrust exerted by the TBM thus providing active face support. The face support pressure can be altered by changing the relative rate at which material enters the excavation chamber (defined by the rate of TBM advance) compared with the rate at which excavation paste is removed (through the screw conveyor). To maintain this pressure, the paste must form a plug in the screw conveyor that supports the pressure difference between the inlet and discharge ends of the screw conveyor (Figure 1.1).

Foaming agent   Surfactant chemical used as an additive to aerate the excavation paste and provide confinement, aid excavation, modify the density of the paste and reduce wear of the TBM components.

Intervention   Access into the excavation chamber of the machine.

Polymer   Additive often used to enhance the viscosity of the excavation paste or to stabilise the foam bubbles. Usually applied to homogenise excavated granular materials. In clay soils, some polymers act as a dispersant to reduce stickiness.

Pressure bulkhead   The pressure bulkhead separates the excavation chamber of the TBM from the rest of the TBM, separating the pressurised area from the non-pressurised area (Figure 1.1).

Ribbon conveyor   A type of screw conveyor fabricated without a central shaft to allow larger solid objects to pass along the screw.

Screw conveyor   A screw conveyor is used to transport excavation paste from the excavation chamber, and also forms part of the regulation system for the face pressure. The screw conveyor is largely housed in a casing, except for the section forward of the pressure bulkhead. Refer to Figure 4.1 for a screw conveyor and discharge point.

Shove   Advance of the machine to allow one ring to be installed.

Soil conditioning   Soil conditioning techniques are used to alter the properties of the excavated materials to make them more suitable for excavation by the TBM. This technology can also expand the range of soil conditions suitable for EPB TBM tunnelling.

Glossary of Symbols

Glossary of Symbols

(A) Dimensions

C Ground cover over the tunnel (depth from ground level to the highest excavated point of the tunnel)

D Excavated diameter of the tunnel

D_1 Internal diameter of the lining

D_0 External diameter of the lining

P Unsupported length of the tunnel heading

Z_0 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]

(B) 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_E Unit weight of the excavated material (including additives)

γ_W Unit weight of water

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[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.

(C) Pressure and external loads

P_{CA} Compressed air pressure

P_E Pressure in the excavation chamber

$P_{E(S1)}$ Pressure measured at sensor 1 (or (S2) for sensor 2, etc.)

P_{Et} Target face pressure in the excavation chamber

$P_{Et(Crown)}$ Target face pressure at the crown of the tunnel

$P_{Et(S1)}$ Target face pressure at sensor 1 (or (S2) for sensor 2, etc.)

q Average surcharge pressure at the ground surface

v Maximum variation in applied face pressure, due to control accuracy, with v being the change in pressure above OR below the target face pressure (Figure 3.3)

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