Canada Line Transit Tunnels (Vancouver, BC): ground stability when excavating in Earth Pressure Balance mode; how changes to the conditioning of the excavated soil cope with varying challenges presented by a highly permeable layer.

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1. Summary

Glacial till section of the Canada Line TBM tunnels experienced some disruption during the excavation. Unexpected not homogeneous soil conditions and layers of loose high permeable material with hydraulic connectivity have been successfully faced though. TBM was always below water table level with peak of 20 metres head.
The control of stability pressure and muck conditioning were key elements of a challenging accomplishment. Specifically, field tests on the spoils and frequent adaptations to the conditioning agents' mixture allowed regular boring cycles. Pulpy and homogeneous material inside the TBM excavation chamber granted a more uniform distribution of the earth stability pressures.
This paper investigates the influence of keeping a proper not permeable muck in respect of the TBM face stability. Analyses show the effects of a homogeneous muck with a coefficient of permeability at least equal to the lowest in situ value. Under such conditions, the possibility of considering the undrained behaviour of the soil is one of the major finding of the present study.
Permeability's influence with non cohesive soil will be studied in further developments.

2. Canada Line: introduction

The Canada Line is a 19km long automated light rapid transit system with 16 stations in Vancouver, BC, Canada. The Concession was awarded to InTransitBC to design, build, partially finance, operate, and maintain the system for 35 years. The project must be complete by November 2009 to be ready for the 2010 Winter Olympics to be held in Vancouver and Whistler, BC. The bored tunnel section consists of 2.45km of twin bored tunnel and 3 stations. The station work is limited to the excavations and temporary support necessary to facilitate the bored tunnel works. A joint venture between SNC-Lavalin Constructors Pacific and SELI (SSJV) has recently completed the design and construction of this section. Work for the bored tunnel commenced in November 2005 with shoring and excavation works for the tunnelling operations pit that will become Olympic Village Station and will be completed in the spring of 2008.
The tunnels were driven with a 6m diameter Earth Pressure Balance Tunnel Boring Machine (EPBM) and lined with 250mm thick, 5.3m diameter precast concrete segments. Seven hundred and fifty meters of each tunnel were driven through glacial and interglacial deposits of sand, silt, and clay with granitic boulders. The paper discusses the challenges faced in negotiating these soils which contained unforeseen geological conditions.

3. Stretch considered in this study: False Creek – Granville Street

The bored tunnels pass under the False Creek and the Downtown area.

The first stretch is beneath the residential area at South False Creek (Stamps Landing). Sloping down (5.5%) the alignment reaches its first lower spot roughly in the middle of False Creek, which here spans about 370m. Hence, it commences to rise at 5.5% again, reaching False Creek's North shore and then smoothly arrives in Yaletown Roundhouse Station (located along Davie street between Pacific Boulevard and Mainland street).
The tunnels continue along Davie Street on a northwesterly bearing to sta. 413+250, at which point they curve (minimum radius 200m on inbound drive) right through 90 degrees onto a northeasterly bearing below the centre of Granville Street.
Beginning the curve the tunnels rise from the second and deeper lower point 5.5% grade to sta. 424+051.68, just south of Vancouver City Centre Station. Afterwards they remain at relatively shallow depth, with cover varying between one and two tunnel diameters, for the remaining part of the drives.

The break through is at the extraction shaft located North crossing between Pender and Granville Street (sta 424+561.51).

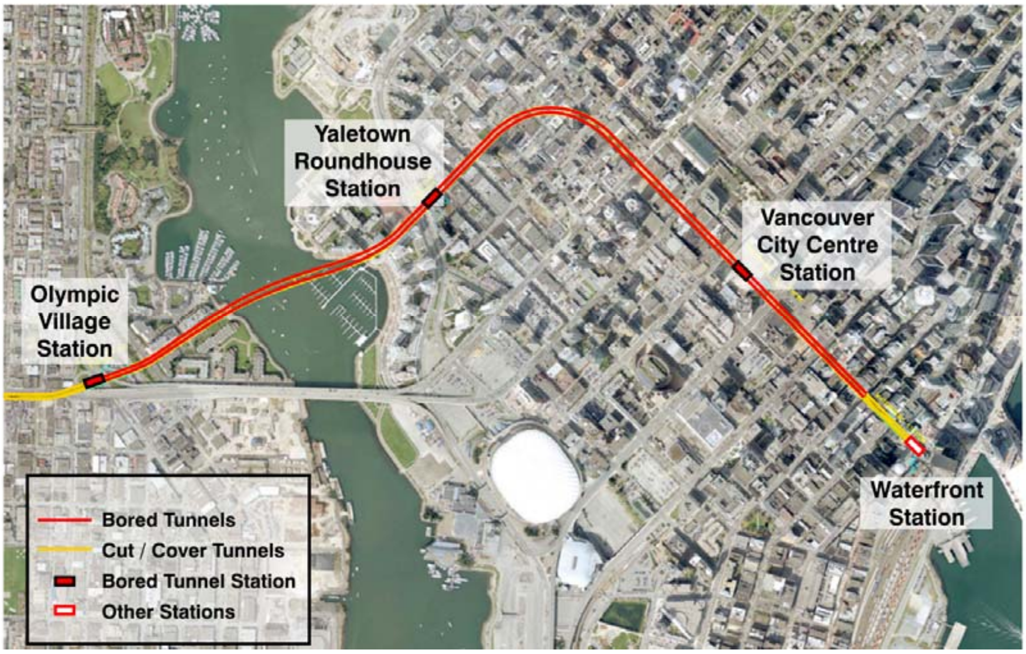

Figure 1: Canada Line TBM tunnels alignment.

Approximately in the middle of the False Creek, the geology changes turning from sandstone into the overlying till (a glacial deposit of silt and sands). The transition is foreseen through a discontinuity having a sub – vertical immersion.

Midway through the curve on Brava Towers, the tunnels turn back into sandstone bedrock – the invert first encounters bedrock at sta. 423+435.

The Early Tertiary sandstones and siltstones are massive and jointing is poorly developed or absent.

The till consists of a clay-silt-sand matrix with gravel and very strong cobbles and boulders up to several meters in diameter.

During the Quaternary Period, within the last 1 million years, the Vancouver area was intermittently covered by thick ice of glaciers. At least three major periods of glaciation are believed to have taken place during the Pleistocene Epoch in this region, the earliest (known as the Semiahmoo and Westlynn glaciations) occurring more than 60,000 years ago, and the most recent (known as the Fraser Glaciation) retreating some 11,000 years ago. The ice sculpted the landscape and deposited a variety of glacial and non-glacial sediments. These sediments are expected to be encountered during tunnelling alignment (Downtown and False Creek); they include thick complex units of glacial till and stratified drift deposited beneath, and at the margins of, the ice.

4. *Geological situation and geotechnical investigations*

The geomechanical conditions considered in the analyses correspond to a situation frequently faced while boring the stretch of twin tunnels from False Creek to Granville Street, which was characterized by the presence of significantly continuous layers of high permeability loose till.

Evidence of the wide spread and continuity of such a soft inclusion with abundant water recharge was obtained during the drillings for the pile sheet of Yaletown Station. Sonic drillings, vertical profiles of grain size distribution and borehole tests (pressuremeter, SPT and hydraulic conductivity testing) gave further indications about the geotechnical properties and the typical thickness of the soft inclusions, which could thereafter soundly represented as a continuous layer. Where such a layer intersects the core of the tunnel to be excavated, a typical case of mixed face conditions is obtained.

Although it is likely that more than a single layer exists, possibly with additional minor inclusions and lenses of limited extent, to keep the model as simple as possible a unique layer was considered. The thickness of the soft layer was fixed equal to 1.5 m, that is D/4 (where D is the tunnel diameter, equal to 6 m), while its position within the face was varied: i.e., four cases were considered, corresponding to a relative elevation z of the base of the layer, with respect to the tunnel axis, of -3, -1.5, 0 and 1.5 m. The set of performed analyses include also two situations of entirely homogeneous soil, i.e., without a soft layer and with a soil mass formed only of soft material, respectively.

On the basis of geotechnical parameters estimated from the site report and the investigations carried out during tunnel construction (Table 1), the two materials which compose the calculation model were given the parameters listed in Table 2. The parameters of the till formation represent the average properties of an ideal medium formed by randomly alternated strata of weak and strong till.

The material model assumed in the following analyses is the conventional linear elastic perfectly plastic model with Mohr-Coulomb strength criterion. The behaviour of the water-saturated porous medium during excavation advance was represented by a two-phase approach. Only the drained elastic moduli and the drained strength parameters are therefore required, while the undrained behaviour is governed by the water bulk modulus (K_w = 2 GPa), porosity and the skeleton bulk modulus.

Table 1: Geomechanical properties of soils from site report and additional investigations

		Strong till	**Weak till**	**Silt**	**Weak, loose till**
Bulk unit weight, γ	(kN/m^3)	21.5	21.5		
Porosity, n	(-)	0.3	0.3		
Shear modulus, G	(MPa)	170	60		
Elastic modulus, E'	(MPa)	425	150	65	
Cohesion, c'	(kPa)	100	25		30
Friction angle, φ'	(°)	40	38		32
Undrained cohesion, c_u	(kPa)	800	200		
Permeability, k	(m/s)	10^{-8}, 10^{-7}	10^{-8}, 10^{-7}		10^{-5}

Table 2: Geomechanical properties assumed in the numerical models

		1) Typical till formation	**2) Layer of high permeability**
Bulk unit weight, γ	(kN/m^3)	21.5	21.5
Porosity, n	(-)	0.3	0.3
Shear modulus, G	(MPa)	120	24
Elastic modulus, E'	(MPa)	300	60
Poisson ratio, ν'	(-)	0.25	0.25
Cohesion, c'	(kPa)	20	0
Friction angle, φ'	(°)	40	30
Dilation angle, ψ	(°)	0	0
Permeability, k	(m/s)	10^{-8}	10^{-5}

The initial stress conditions in the soil mass were obtained by assuming a unit weight of the rock mass of γ = 21.5 kN/m^3 and a coefficient of lateral pressure at rest K_0=0.5. Pore pressures have a hydrostatic distribution.

The tunnel reaches its section of minimum overburden (about 15 m) at the chainage of Yaletown Station, whilst the maximum overburden is (about 30 m) correspond to the under passing of Brava Tower, near the end of the stretch of tunnel to be analyzed. Figure 2, as an example, refer to the situation of minimum overburden. For the same case, the undisturbed water table is at 6 m depth from the ground surface.

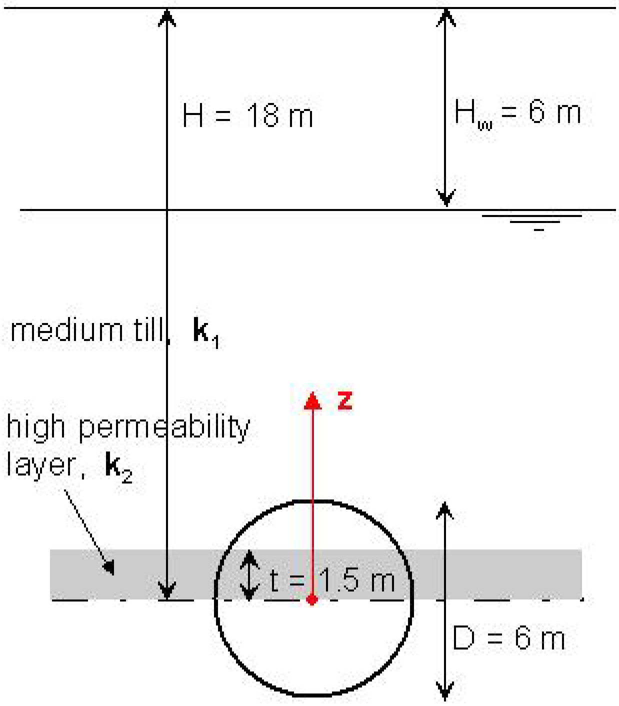

Figure 2: Typical section of analysis.

5. *Problems during excavation and adopted solutions*

Drive 1

The transition from sandstone to till went smoothly without any geological anomalies becoming apparent. Once the machine was wholly in the till face inspections, under atmospheric pressure, showed the face to be comprised of strong impermeable till.

These favourable ground conditions lasted for approximately 300m, to just beyond Yaletown-Roundhouse Station, and regular cutting head inspections and tool changes could be made in normal (atmospheric) conditions. Surface and building settlements were limited to 1 or 2mm with minimum cover of 12m to the crown of the tunnel and water table approximately 8m below surface. A face support pressure of approximately 1 bar was maintained throughout.

After Yaletown, the alignment drops at 5% to implement the mitigation measure of increasing the cover under Brava Tower, while ground level elevation increases at about 4%; ground water level following ground level. Normal boring conditions continued for approximately 120m with good advance rates, small surface and building settlements (2 or 3mm) and face support pressure increased from 1 to 1.2 bar. However, the muck was noted to contain higher proportions of wet sand and silt. The next intervention revealed a layer of wet sand within a stronger till matrix that could not be supported in normal atmospheric conditions due to the mobilization of groundwater.

From this point on the total thrust of the TBM and the cutting head torque began to steadily increase with a steady decrease in penetration rate. However, extracted muck weights and settlement stayed at, or slightly above, normal so the TBM was advanced to find suitable ground for a face intervention but the muck exhibited higher percentages of cohesionless sand showing the waterlogged permeably material was not confined to a pocket, but was a layer with recharging water. In this area, there was more than 20m head of water above the crown. At this stage the face support pressure was increased to prevent the possibility of over excavation. Pressing forward, the penetration rate continued to decrease with increased thrust and it became clear that the cutting tools were at the end of their useful life.

Face interventions were still not possible and sensor observations made during lowering the pressure in the excavation chamber showed that even with compressed air inside the chamber the pore water pressure would increase. With the tunnel crown over 30m deep the only option was to continue boring in the hope that the ground conditions would return to the predicted “till” or become more favorable. After approximately twenty advances in extremely poor ground conditions of silt and sand the ground conditions became better with the volume of water decreasing significantly. Throughout the soil section, the ground was conditioned with foam to work the muck into a paste to properly transfer face support pressure and to form an impermeable plug in the screw conveyor to avoid loss of ground from the face.

The increasing coarseness of the material combined with the high water pressure meant the foaming agent was not effective, or was less effective due to the high water pressure and flow of water breaking down the foam bubbles. In these instances, the muck was permeable and would allow water to flow through the screw conveyor, bringing particles with it (flowing ground). Even in non-plastic silt under the same water pressure the operator had managed to condition the muck to prevent flow of water through the material.

However, this conditioning was less than ideal and the material would flow through the screw conveyor if not mechanically managed by partial closure of the guillotine door.

With the cutting tools in such poor condition, there was significant concern that the machine could become immobilized as it entered the sandstone under Brava Tower. Also, it was expected that the ground conditions at the soil/sandstone interface could include anomalies such as nested boulders or inclusions of poor ground. These mixed face conditions would make the already problematic ground conditioning and face support pressure transfer even more difficult.

The decision on how to continue the drive under the highest risk section with a machine of reduced capacity due to excessive wear and probable damage was based on a number of factors. The settlement readings in all buildings along this section had remained very low (less than 4mm) and surface settlements had remained less than 3mm in all but one section (where the road surface displayed significant previous disturbance). Analysis of the Brava Tower structure showed that with a settlement of 9mm of an individual footing (more than twice that previously experienced), the stresses in the structure would increase only by 10% locally.

Experience had shown that adequate ground control (low loss of ground) could be achieved even with worn cutting tools and very poor ground conditions unsuited to normal EPBM operations. In addition, as the building was designed to be fully drained, the head of water above the TBM was somewhat reduced to approximately 15m instead of 25m. Considering all facts, a calculated risk was taken to continue the drive. The TBM under passed the building (34 advances) in less than 5 days with the penetration rates reducing from 50 to 30mm/min. The maximum settlement experienced at any footing was 6mm with 3 or 4mm being normal for other footings directly over the alignment.

For the entire section of till, the cuttinghead was dressed with disc cutters that were required for two reasons; to bore within the rock at each end of the soil section and to bore through the boulders that are confined in the stiff/hard till matrix. Through the wet sands and silts, the scraper teeth at the cutting head openings will have done most of the “cutting”.

<u>Before Drive 2</u>

The overall positive results achieved during Drive 1 provided the basis for improvements for Drive 2, in particular eliminating or minimizing the risk due to the previously “unknown and unforeseen” conditions.

Following a specific risk evaluation report prepared by Prof. Kovari, an additional package of improvements, refinements, and modifications was introduced as risk mitigation measures to enhance the operational and functional efficiency of the TBM in the reassessed geology and to reduce risks to acceptable levels. Most of the proposed improvements were implemented in the section after the station where the most difficult conditions (wet sand) were encountered.

The risk assessment for the second drive did not identify any explicit hazard in the first 200m of the till section, but focused primarily on the stretch beyond Yaletown-Roundhouse Station, and specifically on the critical passage close to and underneath two high-rise buildings (1155 Homer and the Brava Towers).

Due to the unfavorable geological conditions and the dense urban environment in this section several mitigation measures have been taken, such as:

- Tunnel separation increased by 1 diameter (from 12m to 18m) underneath the Brava Towers to minimize their interaction
- Use of long-life and high wear resistance cutting tools with a mixed-face configuration (ripper teeth and cutters)
- Automatic grout line cleaning at the end of each stroke by a shot of hydraulic oil
- Polymer injection: the addition of an automatic integrated Polymer System pump to the original conditioning foaming system
- Additional monitoring instrumentation and 24-hour monitoring (surface, buildings, and extensometers) during the most critical sections.
- Locating maintenance areas based on the Drive 1 reassessed geological mapping and records, where safe and “open face” cutting head maintenance might be possible.
- Hyperbaric crew: specialized crews for hyperbaric intervention available on site (until the tunnel is fully into the sandstone formation).
- Operations and boring parameter control: phases and values checked daily based on shift reports and PLC automatic data logger. Daily toolbox meeting for coordination and planning.
- More frequent calibration of weight scales as the belt scales are adversely affected by very wet soils and the vertical and horizontal curves.

The mitigation measures and the scheduled maintenance areas were instrumental in the efficacy of the excavation through the entire till section, and the reduction of risks from Drive 1 greatly enhanced the performance of the TBM, particularly the ability to keep the proper face support pressure and assure the right conditioning reducing the soil permeability. These two aspects were thoroughly studied before approaching the sand layer during Drive 2.

Having encountered no real problems in the stretch before Yaletown-Roundhouse Station during the first drive, the design face support pressures were maintained the same. Beyond the station the principal criterion used for the EPB pressure calculation was to counter balance the water table pressure and add 20kPa as a safety buffer. The sand section was split into three subsections where the pressure was raised from 150kPa to 200kPa, reaching a maximum of 220kPa going toward the Brava Towers and the sandstone transition.

In addition, an experimental section was successfully driven with high face support pressures (up to 2 bar) before approaching the station to check the TBM performance and possible problems under high operating pressures. Two similar test sections were made after the station at which several tests were conducted to determine the appropriate soil conditioning parameters including Foam Expansion Ratio, Foam Injection Rate and injection pressures. Variations of water flow, foaming agent, and polymer concentration were investigated through slump tests, permeability tests, and visual inspections of the muck. As a result of these tests the set of selected parameters produced a pulpy, dense, and water-tight paste suitable to stop the water flow, and with the right workability. The overall aim was to operate at the design face support pressure and properly conditions the soils using foam agents and if necessary add the polymer to stabilize the foam and reduce overall permeability.

Drive 2

As said in the previous chapter the design of the excavation between the sandstone/till transition and Yaletown-Roundhouse Station did not fundamentally change for Drive 2. The ground control confirmed by the surface, building, and soil monitoring instruments were very similar to Drive 1.

With respect to the maintenance of the TBM, passing through the Yaletown-Roundhouse Station box provided the opportunity to fully refurbish the cuttinghead and switch to a mixed configuration of cutting tools as prescribed in the mitigation measures (the first drive passed through the station box prior to excavation).

Beyond the station, three maintenance areas had been planned, the location of which was carefully analyzed from the experiences of the Drive 1. At the first maintenance area 70m from the station, the sand layer was expected to be approximately 2m deep and half way up the face with hard till (with boulders) above. The chamber was emptied for the first intervention and this assumption was confirmed by visual inspection.

Most of the rippers, having been damaged by boulders or worn by the ground, were replaced with disc cutters. Only long life carbide insert rippers remained. The gage disc cutters were replaced with long life carbide insert disc cutters as the cutters in this position are subject to more wear.

The cutting head fully refurbished, excavation continued as planned past the first critical building section (1155 Homer Street) with 15m of cover to the deep foundation and into a full face of sand. With the cutting head in good condition, the design face support pressures could be maintained and the ground conditioning parameters were tweaked slightly depending on the observations of the engineers and TBM manager. Unlike the first drive, the high operating pressures could be maintained and the ground was properly conditioned using only foam.

The second maintenance area was planned at a short section of “good” ground (a section of only several advances) experienced during Drive 1, which lay just before a section of ground where the control of excavation was extremely difficult on Drive 1. The assumptions about the good ground proved to be correct and the cutting head was fully maintained under normal atmospheric conditions. At this stage, the TBM was 50% through the sand layer and the cutting head had been refurbished twice. By comparison, at this location on Drive 1, only a few cutting tools had been changed due to the interventions coinciding with very bad ground and the cutting tools were already worn.

The third and final planned maintenance area was 75% through the bad ground and shortly before the alignment entered the footprint of the Brava Towers. The maintenance area was again planned adjacent to a very short section of “good” ground experienced in Drive 1.

The 2 hour intervention at this location began with a steady flow of water coming from the sand layer that changed into gushing water flow that started to cause sloughing of the ground above the crown of the machine. The intervention was abandoned but was long enough to change 4 disc cutters and do a good inspection of all cutting tools revealing that they were in relatively good condition. General maintenance was carried out on the entire TBM and back up system. Based on the good condition of the cutting head and maintained TBM, the construction team made the decision to complete the drive

through the bad ground without further planned stops, the emphasis now being on continuous boring until the TBM was fully into the sandstone, through the soil/rock interface under Brava Tower.
The machine advanced at an average of 12 rings per day (17m), at 2.2 bar face support pressure at the top of the cutting chamber. The penetration rates, total thrust, torque, grout injection pressures and volumes all remained within expected limits with the torque and thrust increasing and penetration rate dropping as the machine entered the sandstone. The ability to maintain the design face support pressure (not having to drop the pressure to allow forward advance), thus balancing the hydrostatic pressure (therefore not mobilizing any ground water through the sand) allowed the soil to be conditioned only by the injection of foam. Conditioning of granular material, to make the muck into the required impermeable consistency, was therefore achieved at chamber pressures of 2.7 bar (at inlet to screw conveyor).

Tests on conditioned soil

Conditioning system had been checked and continuously updated along a 200 m trial section of Drive 2.
Several tests had been performed on spoil material removed from the screw conveyor screening:

- Slump
- Density
- Permeability
- Moisture content

All the parameters related to every TBM stroke had been filed and analyzed.
On the trial stretch, face foam flow, surfactant percentage (TA%) and water flow in the chamber have been modified to verify the TBM behaviour under different conditions (high or low excavation pressure, till matrix with or without sandy inclusions, etc).
Slump tests did not depend on excavation pressure while muck density recorded 13 to 15 kN/m^3 values.
High EPB pressure required a different conditioning set, increasing the surfactant percentage. The best results have been achieved with a low to very low water flow in the excavation chamber and high F.I.R. values (110-140).
Besides laboratory tests on till mixed with sand had been made. Addition of polymer agents had been studied too.
Eventually the most effective conditioning parameters granted reaching a permeability of the excavation spoil ranging from 10^{-7} to 10^{-8} m/s.

6. *Preliminary analyses of face stability*

The gradual increase of ground deformation and loads on the EPBM shield during tunnel excavation is associated with spatial stress redistribution taking place in the vicinity of the advancing face and with consolidation processes in the ground surrounding the tunnel. Time-dependent consolidation processes are particularly relevant for tunnelling through water-bearing, low-permeability ground.

Plastic deformation and remoulding of the saturated ground around the EPBM shield leads in general to an increase in water content, which occurs more or less rapidly depending on the permeability of the ground. In a low-permeability ground, the water content cannot change immediately after excavation. Instead, excess pore pressures develop, which dissipate over the course of time.

In case of non-hydrostatic in situ stress ($K_0 \neq 1$), volume deformations due to excavation can assume different sign along the tunnel wall and, as a consequence, the pore pressure change be negative as well positive. In any case, tunnel excavation causes a transient seepage flow process. The short-term behaviour is characterized by a constant water content ("undrained conditions"), while the long-term behaviour is governed by the steady-state pore pressure field ("drained conditions"). The undrained conditions are generally more favourable because short-term negative pore pressure changes are prevailing in the ground surrounding the face.

The ratio of excavation rate v_a to ground permeability k is the key parameter governing the deformation process at the tunnel face as well as the inception of instability. If the excavation proceeds slowly or the permeability is high, the conditions can also be practically drained since the short term. In the case of mixed ground, as that to be analyzed herein, the situation becomes even more complex, because the characteristic times for pore pressure equalization can be very different in zones of markedly different permeability.

The proposed modelling approach is based on the application of three-dimensional (3D) Finite Difference Models (FDM), which can achieve a realistic prediction of ground deformations near the excavation face as well as of the rock loads acting on the TBM shield and the support system.

A preliminary set of stress analyses focused on the stability conditions of the face, taking the seepage process towards the excavation chamber into account. The following assumptions were made in order to simplify the analysis and ease the comparison with the results of conventional limit equilibrium models (e.g. Kovari's approach): i) lining is perfectly rigid and installed before any deformation of the tunnel walls, ii) both lining and EPBM shield represent impermeable boundaries so as flow is directed only towards the face, iii) the excavation camber is not included in the model, thus the boundary conditions applied to the face consist of fixed distributions of pore pressure as well as of effective stress, with given vertical gradients.

Two limit cases were considered for groundwater flow conditions: A) represents the situation of maximum drainage at the tunnel face (pore pressure p = 0), corresponding to very high permeability of the spoil-water mixture inside the excavation chamber (i.e., totally unsatisfactory conditioning); B) represents the optimal situation, where the undisturbed hydrostatic pore pressure distribution is fully preserved, thanks to a sufficiently low permeability of the conditioned soil inside the excavation chamber, which therefore acts as an ideally impermeable barrier.

Also possible are intermediate situations, which correspond to different grades of partial drainage at the face: i.e., pore pressure distributions reduced with respect to the undisturbed groundwater conditions but still greater than zero. Such a situation will be investigated in the following set of analyses.

For the limit case p=0, the transient flow is calculated for increasing elapsed time. For the typical values of ground permeability considered herein, the time required to attain steady-state flow was found to be about 3 hours. Hence, taking the average excavation rate into account, it seems reasonable to assume that the EPB excavation process develops in drained conditions, at least for what concerns the behaviour of the soft layer of higher permeability.

Further assumptions were required to define the shape of the total pressure profile applied to the face. In a first case 1) a linear distribution characterized by a gradient γ_f = 10 kN/m^3 was assumed; then, a case 2) characterized by a uniform distribution was also experimented. In either the case the control parameter is the value of total pressure (P0) applied at the centre of the tunnel face. As known, there is no simple relation between the vertical gradient γ_f and the density of the muck inside the excavation chamber (about 1500 kg/m^3): the assumed value can be seen as an estimate based on average readings from pressure measurements at different positions inside the face. Many factors, such as the water content, the foaming additives and the rotation or rest of the cutting head can affect the vertical pressure gradient.

The stability factor of the face was therefore calculated by the method of progressive reduction in the effective stress applied to the face. The stability factor can be represented as the ratio between the total pressure actually applied to the face and that corresponding to the triggering of uncontrolled displacements.

Table 3:- Preliminary analyses without EPB.

Case	**Permeability k_1 (m/s)**	**Permeability k_2 (m/s)**	**Total pressure at face**	**Pore pressure at face**	**Analysis**
VNA1	undrained	10^{-6}	linear (grad=10 kN/m^3)	0	Flow+Mech
VA1	10^{-8}	10^{-6}	linear (grad=10 kN/m^3)	0	Flow+Mech
VA2	10^{-8}	10^{-6}	uniform	0	Flow+Mech
VB1	10^{-8}	10^{-6}	linear (grad=10 kN/m^3)	hydrostatic, undisturbed	Mech only
VB2	10^{-8}	10^{-6}	uniform	hydrostatic, undisturbed	Mech only

A sensitivity analysis was aimed at evaluating the influence of the following factors: the position of the high permeability layer inside the face, the shape of the total pressure distribution upon the face and the amount of pore pressure reduction inside the chamber with respect to the undisturbed ground. The last parameter is particularly important and the crucial question arises of how its appropriate value can be estimated on the base of the relevant properties of the conditioned soil. A link to the findings of the experimental testing program on the variably conditioned till should be established.

A further analysis was devoted to the study of a simplifying modelling approach in which the flow domain is limited to the high permeability layer, while the medium till portion of the model is assumed to display a fully undrained response during excavation advance.

A set of figures showing some results of the preliminary analyses is thereafter included: most of figures refer to the situation where the high permeability layer is located in the upper half of the face ($0 < z < 1.5$ m), chosen as the reference case. The main findings of the study are hereafter summarized.

- In the limit case of fully drainage (p=0), the distance of influence of the face on flow conditions is about 6D, i.e., at greater distances the reduction in pore pressure is less than 5% (Figure 3). This situation represents the worst case scenario for the assessment of the impact of the excavation process (excessive drawdown of water table, risk of large settlements). On the other hand, this case corresponds to the demand of total pressure to stabilize the face (P0 = 20 kPa, Figure 4).
- In case of no disturbance of the initial hydrostatic pore pressures, the minimum total pressure necessary for face stability corresponds to P0 = 120 kPa, which indicates that almost the whole amount of pressure is required to counterbalance the water pressure.

- The influence of the vertical gradient γ_f of face pressure seems modest, at least within the limits posed by the accuracy (±5 kPa) in the determination of the collapse load (Figure 5). This result can be explained considering the small thickness of the low strength layer with respect to the tunnel diameter: the response of the face to unloading of original stress is therefore markedly different, and more advantageous, from the case of homogeneous soil.
- The position of the soft layer within the tunnel face (4 different situations were analyzed and compared each other) has a limited but yet significant influence on the minimum pressure necessary to avoid instability. Figure 6 show the situation in terms of average pressure P0 as a function of layer position: the maximum variation in P0 is of about 20 kPa. A less important variation would be obtained if the pressure value at the middle of the soft layer were considered instead of the pressure P0 at the centre of the tunnel face.

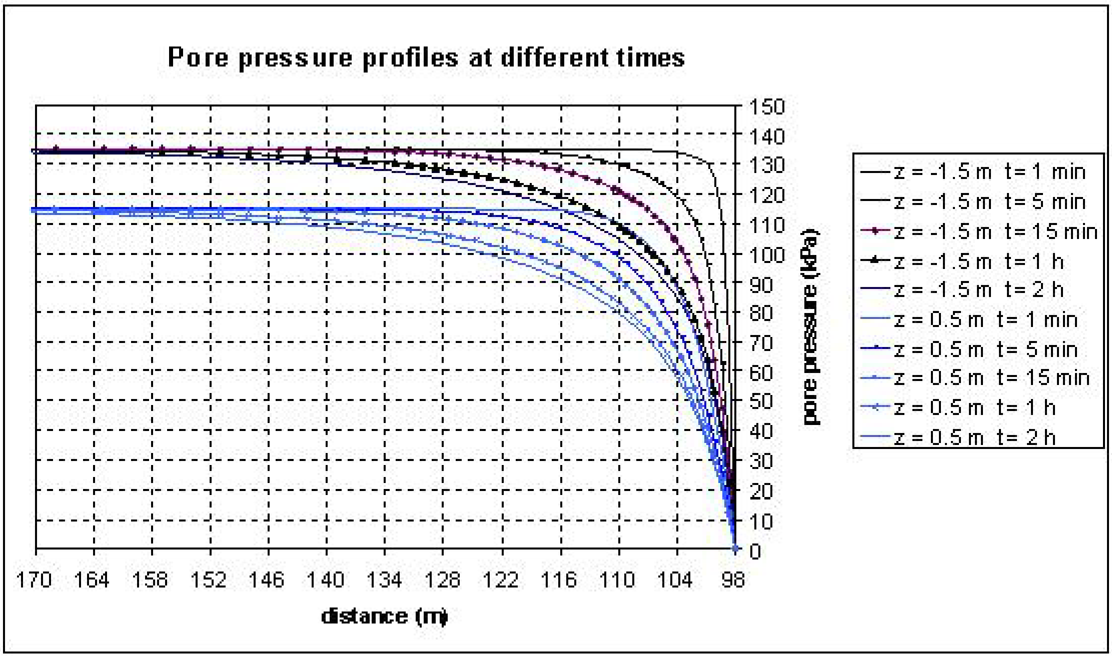

Figure3

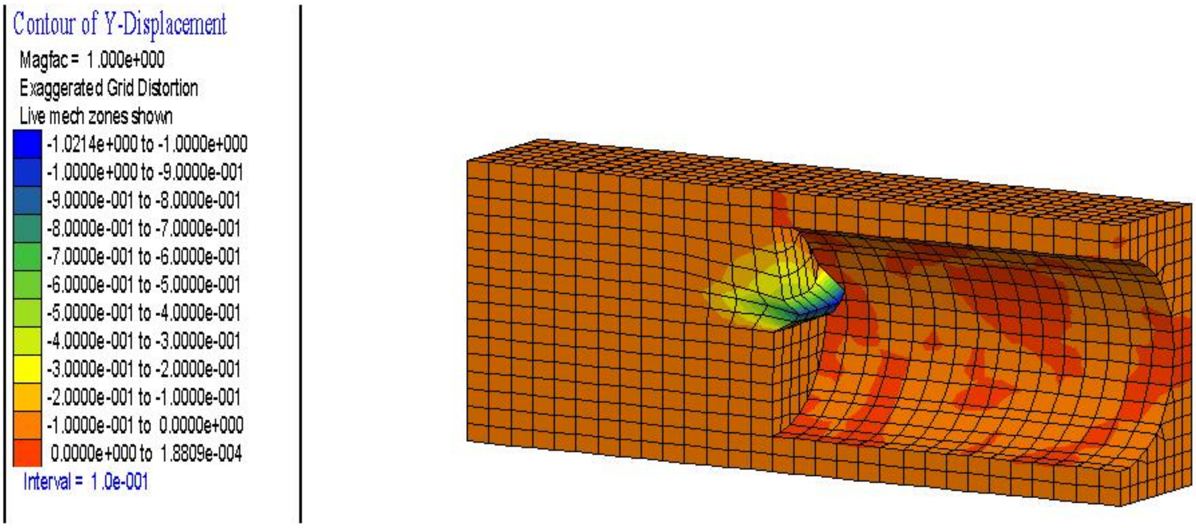

Figure4

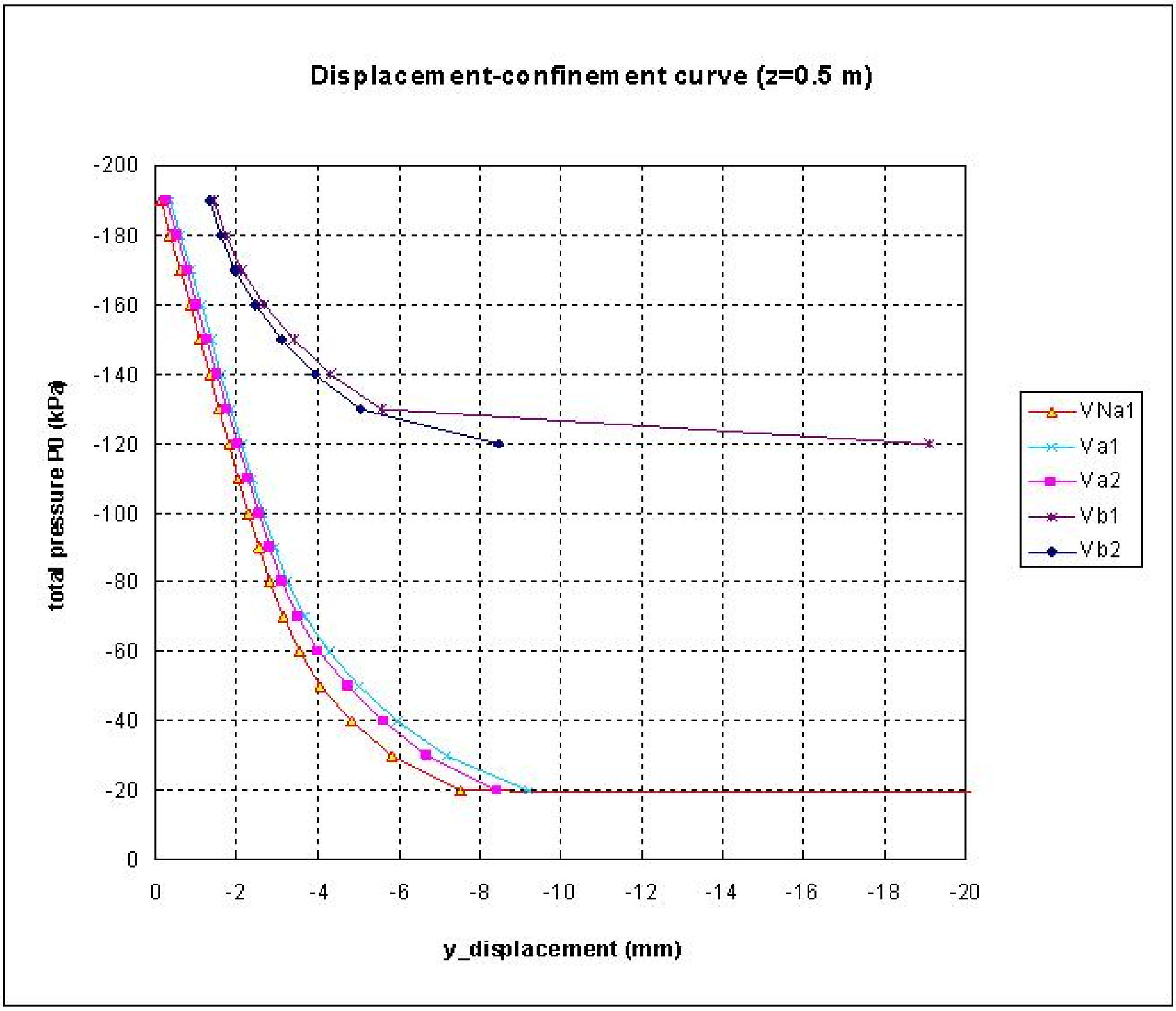

Figure5

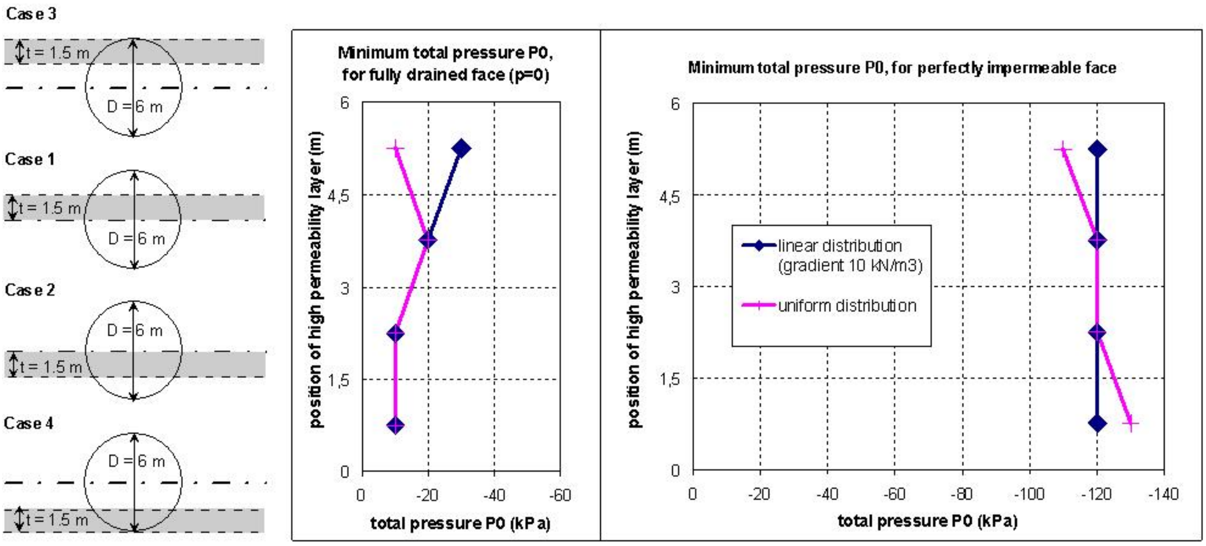

Figure6

7. *Analyses with explicit modelling of EPBM*

A further refinement introduced in the model is the possibility of directly representing the conditioned soil inside the excavation chamber and the screw conveyor, again by adopting a two-phase model for the conditioned soil as well as for the natural soil surrounding the tunnel. This approach eliminates the need of an a-priori assumed distribution of pore pressure over the tunnel face but require an appropriate assessment of the permeability of conditioned and fully remoulded soil inside the machine. In fact the flow domain also includes, although with an idealized geometry, the aforementioned parts of the EPBM.

Following the same approach of the "without EPB" analyses, the flow process and thus the pore pressure distribution has been studied for increasing times (transient flow) up to obtain the steady state flow conditions. The key parameter is the permeability (K_{EPB}) of the conditioned soil inside the EPB, which determines the time required to reach stationary flow and, moreover, the relative head loss inside the EPB (mainly along the screw conveyor) with respect to the decrease in hydraulic head in the ground in front of the face.

Again, the analysis has been split down in two phases: a first "flow" only calculation, up to steady state, and then a "mechanical" phase, in which a progressive decrease in total stress applied to the face has been performed.

Direct measurements of muck permeability, as it flows off from the screw conveyor, have indicated typical values of about 10^{-8} m/s, when the material exhibits a well homogenised texture and a medium water content (i.e., optimum conditioning); on the contrary, some sporadic much higher values of permeability (practically out of the range of the available in situ measurement device) were obtained for disaggregated muck and discontinuous flow from the conveyor. Several data reported in the technical literature, mainly coming from EPB excavations in fine sand and silt, indicates muck permeability in the range 10^{-6} - 10^{-5} m/s for water and foaming agents percentage so as to reduce the effective (grain-to-grain) stress to very low values.

In the present analyses, valuable and explicatory results have been obtained carrying out a first set of calculations for $K_{EPB} = 10^{-6}$ m/s and for $K_{EPB} = 10^{-5}$ m/s.

The main findings of this last set of analyses are:

- It is sufficient to guarantee a muck permeability K_{EPB} as low as 10^{-6} m/s, that is, equal to the assumed in-situ permeability of the soft ground layer, in order to virtually eliminate the drawdown effect associated to the excavation advance. In fact, in this case, groundwater conditions stay undisturbed and most of the head loss develops inside the screw conveyor. From a mechanical point of view, the total pressure required for face stability equals that obtained in the previous analysis with "no drainage" hypothesis (P0 = 120 kPa, Figure 7). The "characteristic curve" of the face (i.e., extrusion displacement vs total pressure) shows that under the ordinary support pressure of 150 kPa the displacement at the centre point of the soft layer is as low as 3 mm. Similar results, in terms of pore pressure as well as of face displacement, are obtained also by a more fast calculation in which only the high permeability layer is represented by a two-phase approach whilst the surrounding low permeability till behaves as an "undrained" medium (analysis VN2).

- If the permeability of the conditioned muck is increased to 10^{-5} m/s, steady-state pore pressure at face decrease significantly below the original undisturbed value (say, 90 instead of 120 kPa at the centre). Correspondingly, the "characteristic curve" of the face indicates a stiffer behaviour than the previous case and a limit pressure of 90 kPa instead of 120 kPa. Finally, observation of the longitudinal stress σ_y contour as well as of the extent and magnitude of plastic strains in front of the face reveals a deformation mechanism whose features generally resemble the "wedge collapse" mechanism often utilized (e.g. Kovari) to asses the limit equilibrium pressure. Yet, the plastic zone (area of shear- and tensile-plastic strains) appears more limited in extent: most of plastic shear strains affect the soft layer and while some tensile strains develop in the surrounding stiffer material as a consequence of the "squeezing" deformation undergone by soft layer.

Table 4: Analyses including the conditioned soil inside the EPB

Case	Permeability k_1 (m/s)	Permeability k_2 (m/s)	Permeability k_{EPB} (m/s)	Total pressure at face	Pore pressure at face	Analysis
VN2	undrained	10^{-6}	10^{-6}	linear (grad=10 kN/m^3)	Not imposed	Flow+Mech
V2	10^{-8}	10^{-6}	10^{-6}	linear (grad=10 kN/m^3)	Not imposed	Flow+Mech
V3	10^{-8}	10^{-6}	10^{-5}	linear (grad=10 kN/m^3)	Not imposed	Flow+Mech

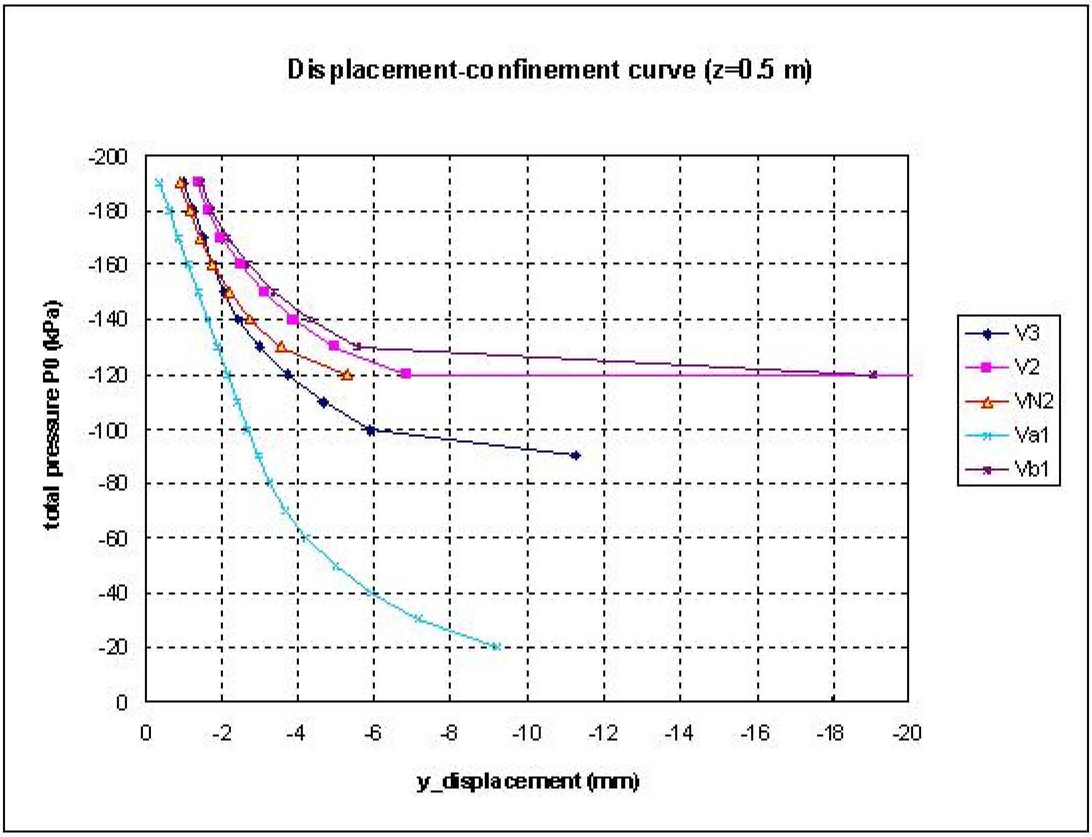

Figure7

8. *Future analyses with simulation of EPBM advance*

A more thorough analysis of the construction process could be achieved by applying a step-by-step approach, in which each round of excavation (i.e., removal of tunnel core elements) is followed by the installation of a new slice of lining (i.e., activation of a new set of liner elements).

This procedure allows detailed simulations of TBM tunnels and a better prediction of surface settlements due to the excavation of shallow tunnels in urban areas. Being the focus on ground deformation, less attention was generally devoted to the structural behaviour of the lining. In many situations of soft soil tunnelling by closed face TBMs, the segmental pre-cast lining represents a relatively "rigid" support and the excavation-induced settlements depend on different factors, mainly related to the control of bulkhead pressures, steering problems of the TBM, tail "gap" and relative effectiveness of gap-grouting.

In this study the lining is modelled by shell elements, which may represent a satisfactory compromise between accuracy of results and computational efficiency and can indeed provide essential information about the ground-support interaction mechanism. The distance from the face to the section where the segmented rings are installed is nearly equal to 10 m.

A more comprehensive approach is tentatively applied, in which the grout pressure at the extrados of the shield is included in the 3D modelling, in order to represent the progressive loading of the shield as well as of the lining, if deformations of the tunnel walls are that large that the annular gap around the shield will be closed. The large-displacement approach as well as interface elements should be adopted in order to simulate the progressive closure of the gap. Yet this kind of approach is heavily time-consuming so as an optimal compromise between accuracy and calculation time must be pursued.