The Eglinton Crosstown Light Rail Transit

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ABSTRACT
This paper provides an overview of the regional geology and followed by a discussion of the preliminary subsurface investigations carried out for the proposed Eglinton Crosstown Light Rail Transit (ECLRT) and the subsurface conditions revealed in the preliminary investigations. Challenges and constraints on the design and construction of the proposed ECLRT project from a geotechnical point of view are discussed. Various alternatives to overcome these constraints are presented including the application of earth pressure balance (EPB) tunnel boring machines (TBM) equipped with cutter head capable of dealing with boulders excavation; ground movement controls by soil conditioning and compensation grouting; groundwater control by secant caisson or slurry walls; and temporary shoring and head walls for underground station construction.

RÉSUMÉ
Ce document donne un aperçu de la géologie régionale, suivie d'une discussion des enquêtes sous-sol effectuées pour le projet et les conditions du sous-sol révélées dans les enquêtes. Les défis et les contraintes sur la conception et la construction du projet ECLRT proposé à partir d'un point de vue géotechnique sont discutées. Diverses solutions pour surmonter ces contraintes sont présentés, y compris l'application des tunneliers (TBM) à pression de terre équilibrée (EPB), équipées d'une tête de coupe capable de faire face à l'excavation des rochers, les contrôles des mouvements au sol par injection de coulis de compensation et amélioration des sols, le contrôle des eaux souterraines durant la construction; système d'étayage temporaire et des murs de soutènement pour la construction de la station de métro.

1 INTRODUCTION

The proposed ECLRT is a 33km long corridor that would link the Pearson International Airport in the west and the existing Kennedy subway station in the east upon completion. Due to the surficial congestion in the middle section of the existing Eglinton Avenue, the proposed LRT route will include an underground section with twin tunnels to start from a portal in the west near the Black Creek Drive, cross a well developed urban area and end at a portal near Brentcliffe Drive in the east. The total alignment length of the twin tunnels is about 10.5 km.

The underground section of the ECLRT will consist of constructing proposed twin tunnels with an internal diameter about 5.75 m and a total of twelve underground stations along the tunnel alignment. While an expanded tunnel length for the ECLRT project is under consideration, this paper focuses on the geotechnical constraints of the present project originally conceived in the Transit City network.

From a geotechnical engineering point of view, the proposed ECLRT twin tunnels can be very challenging and demanding due to the urban setting with respect to ground loss and ground movement control as any excessive ground settlements would cause potential damages or even collapse of existing adjacent structures at or near the ground surface. It is therefore critical to have a thorough appreciation of the regional geology and to obtain adequate information regarding the subsurface soil and groundwater conditions and geotechnical properties of the subsurface medium by implementing a well commissioned subsurface exploration programs to facilitate engineering design and selection of appropriate construction methodology.

2 GEOLOGICAL SETTING

2.1 Physiography and Drainage

On a regional scale, the topography of Toronto slopes gently southward towards Lake Ontario, which is approximately 7 km south of the project site. However, the local terrain generally varies from east to west where hilltops, formed during various glacier and interstadial activities, have been eroded by streams draining southward into Lake Ontario. The river erosion of the till deposits has developed into valleys that are relatively steep sided. It is of interest to note that the bedrock contours suggest that present river system formed along pre-glacial streams, which eroded deep into the shale. The topography along the tunnel section of the alignment slopes downhill in the vicinity of the intersection of Old Forest Hill Road and Eglinton Avenue where a hillcrest is located around El. 185 m. At the base of this hill, West Don River forms a valley to the east where the terrain is about El. 105 m; and Black Creek forms a valley to the west where the terrain is about El. 110 m. The alignment of the ECLRT crosses the following rivers and streams sorted from west to east: Mimico Creek, Humber River, Black Creek, Don River West and Don River East. Loose alluvial deposits and soft ground may be found in these locations.

The groundwater at the site along the proposed tunnel alignment within the quaternary overburden drains westerly to Black Creek, easterly to West Don River, and southerly to Lake Ontario. Water levels ranges from several meters of artesian pressure along the west hill slope to a few meters below the proposed tunnel invert near the hilltop and along the east hillside. Existing piezometric levels in the tunnel horizon are expected to range from approximately El. 110 m at the western end of the tunnelled portion near Black Creek Drive rising to approximately El.158 m between Dufferin Street and Bathurst Street and falling to approximately El.140 m at Bayview Avenue. Groundwater flow occurs towards Lake Ontario to the south and laterally toward lower lying areas in the Humber River Valley and the Don River Valley. Perched groundwater can be expected to occur within shallow layers of sandy native soils and within fill. Downward hydraulic gradients are apparent in piezometric records in the higher ground between Dufferin and Bathurst Streets indicating a zone of recharge to the groundwater system while upward gradients and slight artesian conditions are noted in lower lying areas near Black Creek indicating groundwater discharge.

2.2 Quaternary Deposits

The general geology of Toronto area consists of Quaternary deposits overlying shale bedrock. The Quaternary overburden in Toronto typically consists of recent alluvial deposits; glacial tills; glaciolacustrine silt and clay deposits; granular beach deposits; and glaciofluvial silt and sand deposited during and after the various advances and retreats of glaciers.

The recent natural alluvial deposits are found within the flood plains of existing rivers and streams, such as the Humber River, Black Creek, and Don River. Generally these sediments vary from bouldery gravels in stream beds to organic-rich sands and silts along the flood plains. In addition to naturally formed deposits, emplaced fills can be found in the area.

From the published geological data (Karrow 1967 and Sharpe 1980), the Toronto area experienced at least three glacial and two interglacial periods, during which time a sequence of glacial and interglacial depositions took place. Towards the end of the last ice age, when Wisconsinan glacier withdrew from the Lake Ontario basin to the north and to the east, Lake Iroquois, the forerunner of the present Lake Ontario, was established. The entire sequence of these glacial, interglacial and lacustrine deposits is, however, seldom found intact and usually one or more of these units are absent at any one location.

The oldest Quaternary deposits are the Illinoian age represented by the York Till which is overlain by Sangamonian-aged interglacial deposits (sands, silts, and clays) of the Don Formation. The Wisconsinan age is represented by deposits formed during several glacier advances and retreats. These deposits are represented by the Scarborough, Pottery Road, and Thorncliffe Formations set down during glacial retreats, the Sunnybrook Till from the Early Wisconsinan time and Leaside Till (Newmarket Till and Halton Till) from the Late Wisconsinan period. These tills were deposited during periods of ice advances into the Ontario basin. Typically, the tills consist of a heterogeneous mixture of gravel, sand, silt and clay size particles in varying proportions. Cobbles and boulders are common in these deposits.

The soils below the upper till are generally pre-consolidated, primarily due to the height of ice which covered most of the area. As such they provide good foundation materials and can stand above water table at relatively steep side slopes in temporary excavations. There are however exceptions to this where the tills have been reworked due to water action at the frontal lobe locations or other natural phenomena, such as ice inclusion when first transported and subsequent melting which caused weak zones in the till. These weaker tills are locally referred to as 'ablasion tills'. The last major glacial event during late Wisconsinan age was the occupation of the current Lake Ontario basin by former glacial Lake Iroquois. This resulted in the deposition of glaciolacustrine sediments mostly silts, sands and gravels along its shoreline and gradually finer-grained sands, silts and clays further off-shore in deeper water.

2.3 Bedrock

The Bedrock underlying Toronto area consists of three shale-dominated units of Late Ordovician age, namely the Blue Mountain, the Georgian Bay and the Queenston formations. The bedrock underlying the ECLRT tunnel section is considered to be the Georgian Bay Formation which is bluish and grey shale with interbeds of sandstone, siltstone, limestone and dolostone. The rock is heavily fractured and sheared, and may contain significant locked-in horizontal stresses which can impose significant loads on tunnel liner or excavation wall in a time-dependent manner.

The shale bedrock surface is recorded at elevations ranging from 99 m to 105 m between Yarrow Road and Richardson Avenue near the western limit of the propose tunnel alignment. The bedrock levels are falling steeply to the west into the Humber River valley. The interpreted bedrock contours rise to approximate El. 110 m beneath Dufferin Street and gradually fall to El. 60m near Brentcliffe Road at the eastern limit of the proposed tunnel alignment.

Based on the available geological information, the results of the preliminary investigations and the current proposed vertical tunnel alignment, bedrocks will not likely be encountered during the tunnelling drive.

3 INVESTIGATION PROGRAMS

Geotechnical information is needed from the very moment of project planning start as site geology plays a dominant role in many of the major decisions that must be made in planning, designing and constructing of a tunnel. Geology dominates the feasibility, behaviour and the cost of the tunnel project. The investigation programs and the extent of the investigations depend on the complexity of the site geology, the complexity of the proposed project and the amount of information available.

3.1 Previous Investigations

The initial investigations along the west part of the existing Eglinton Avenue corridor between Keele Street and Allen Road were started in 1990's. The investigations were carried out by means of borehole drilling, in-situ testing and laboratory testing. Data and information from previous investigations were made available by TTC before planning and implementing the current investigation programs.

3.2 Current Investigations

The purposes of the current investigation undertaken by Coffey are to assess the subsurface conditions along the tunnel alignment and provide information and data for the preliminary design of the project. The preliminary investigation program consists of borehole drilling, in-situ testing and laboratory testing. The field work of the investigations was carried out using the latest TTC Geotechnical Standards. A total of 150 boreholes were advanced to the depth of 1.5D to 3D (proposed tunnel diameter) below the proposed track level (El. 110 m to 155 m) at a maximum borehole spacing of 120 to 150 m. At proposed underground station sites, 9 boreholes were typically advanced to 15 to 20m below proposed track level. In-situ testing was carried out in conjunction of soil samplings. In-situ testing conducted during the investigation includes standard penetration tests (SPT), field vane shear tests (FVST), pressuremeter (PMT) and field hydraulic conductivity testing (K-test).

3.3 Results of Investigations

3.3.1 Subsurface Stratigraphy

The stratigraphy encountered along the twin tunnel sections generally consists of fill, native glacial till deposits of silty clay to clayey silt and silty sand to sandy silt, and interglacial deposits of silty sand to sand, silty clay to clayey silt. A generalized profile of subsurface stratigraphy relative to the proposed vertical profile of the tunnels is shown in Figure 1. It has been noted that non-cohesive lenses and layers of sand and silt were generally encountered in the glacial till. The interglacial deposits predominantly consisted of sand to sand and silt with interlayered silty clay to clayey silt deposits.

Boulders and cobbles were inferred by auger grinding, obstruction to auger advancing and spoon refusals during drilling. The existence of boulders and cobbles in glacially derived deposits has been confirmed by previous construction experiences of the existing Toronto subway tunnels.

Groundwater levels measured in monitoring wells installed during the current investigation ranged from approximate El. 117 m to 172 m. The twin tunnels are anticipated to be excavated under the groundwater level based on the current proposed vertical alignment. The presence of artesian and sub-artesian condition was noted in the vicinity of the west portal area (Black Creek Drive to Keele Street area). Based on the inferred bedrock elevations and the current proposed tunnel alignment, bedrock is unlikely to be encountered during the twin tunnel TBM drive.

3.3.2 Field and Laboratory Testing Results

Conventional geotechnical laboratory tests were performed on the recovered soil samples for purposes of classification and assessment of the engineering properties of the subsurface soils along the tunnel alignment. The results of In-situ and laboratory testing show that the glacial tills typically have a firm to hard consistency or in dense to very dense condition except in west portal area where soft to firm silty clay to clayey silt till was encountered. The interglacial cohesive deposit of silty clay to clayey silt has firm to hard consistency and the non-cohesive deposits are in compact to very dense condition.

Pressuremeter testing and laboratory triaxial test results from the previous and current investigations were collected and evaluated. To facilitate tunnel liner design analysis using a non-linear elastic constitutive model such as the Duncan-Chang model (Duncan & Chang 1970), the model parameters were assessed from the available test results and are presented in the Figure 2.

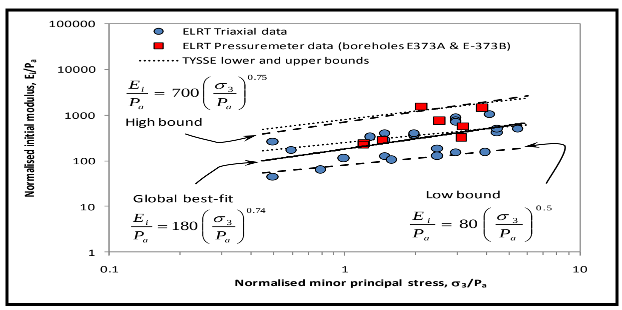

Figure 2. Initial elastic modulus-stress dependent relationship (after F. Badelow and D. Oliveira 2010)

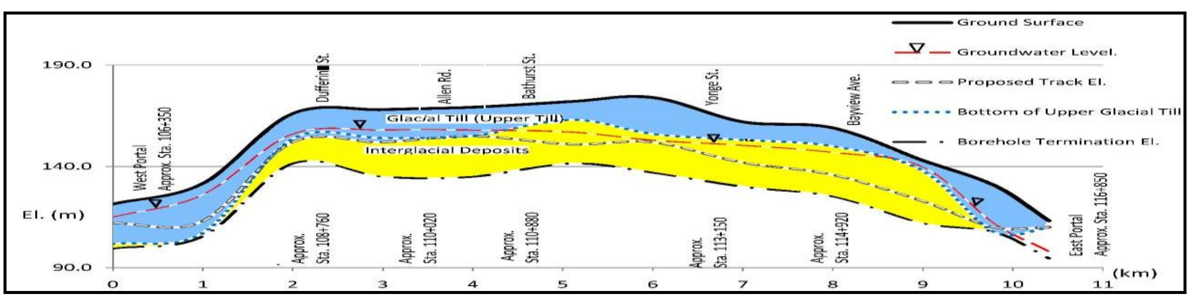

Figure 1. Generalized subsurface profile along tunnel alignment

4 CHALLENGES AND CONSTRAINTS OF TRANSIT TUNNELS IN URBAN ENVIRONMENT

Transit tunnels in urban environment such as the proposed ECLRT tunnels can be very challenging from a geotechnical engineering point of view. Some common features exist in Transit tunnels constructed in urban environment due to their relatively shallow overburden.

Firstly, in most cases most recent weak geological formations are encountered near the ground surface and their conditions are very frequently varied due to the presence of soil lenses, layers and boulders. Secondly, tunnelling may have to be carried out under groundwater table, under drainage courses that cross the alignment. Lastly, tunnelling in urban setting is demanding with respect to ground movement control as excessive settlements would cause potential damages or even collapse of existing adjacent surface structures or underground utilities.

Sources that contribute predominantly to ground movements during the ECLRT tunnel drive may include ground loss at tunnel face due to unstable face condition; ground loss along the TBM due to over excavation during the drive; radial convergence of the excavated openings due to the change of effective stress in the ground; and consolidation or volume changes in ground due to stress state changes and ground disturbance.

Given the tunnel alignment constraints, including adjacent buildings, major utilities, existing subway stations, a right-of-way on the order to 23m, and other significant surface constraints, the 6m diameter ECLRT twin tunnels will be bored at center-to-center spacing of about 11m only. It is considered to be of prime importance to control the above sources of ground movement to an acceptable level from both design and construction perspectives.

5 DESIGN CONSIDERATIONS AND DISCUSSIONS

5.1 Tunnelling Conditions and Face Stability

Based on the subsurface stratigraphy along the tunnel alignment, the proposed ECLRT twin tunnels are anticipated to be excavated in mixed face conditions of cohesive and granular soils and in full face of granular soils that are typically water bearing and saturated. The presence of artesian and sub-artesian condition was noted in the vicinity of the west portal area (Black Creek Drive to Keele Street area).

The face stability of tunnels is dependent on the relative strength of the ground, the depth of the tunnel, and any internal pressure applied to the tunnel face. The most common approach to assessing the stability of tunnel faces in soil drives uses the stability number N_s, which can be defined in Eq. 1 for cohesive soils:

$$N_s = (\gamma H - P_i)/S_u \qquad [1]$$

Where

γ = total unit weight (kN/m^3)
H= depth to tunnel axis (m)
P_i=supporting pressure at tunnel face (kPa)
S_u=undrained shear strength of soil (kPa)

Based on references (Clough & Schmidt, 1981) and experience, it is generally considered:

- $N_s < 2$; small ground movement and shield tunnelling not required;
- $2 < N_s < 4$; shield generally used to restrain ground movements;
- $4 < N_s < 6$; increasing ground movement even with shield tunnelling;
- $6 < N_s$; face may be unstable and face support is required.

The proposed ECLRT twin tunnels are anticipated to be excavated predominantly through non-cohesive deposits below the groundwater level. Face stability issues are anticipated during tunnelling. Tunnel boring machines such as earth pressure balance (EPB) machines that can provide positive pressure to counter balance the earth pressure and the groundwater pressure at the tunnel face will be used for the tunnelling drive.

5.2 EPB TBM and Soil Conditioning

The essence of an EPB machine is the provision of substantial support to the excavated tunnel face at all the time. A true EPB drive requires that the working chamber be fully filled with excavated soil or muck so that adequate pressure can be provided to ensure face and ground stability. Furthermore, a fully filled working chamber can also prevent groundwater from entering the chamber and minimise the risk of over-excavation. Proper management of face support pressure, prevention of water entering the TBM and management of drive torque can have a considerable influence on the performance of the TBM such as the advance rate and downtime.

Certain properties of the muck in the working chamber are critical to maintaining the chamber pressure and to keep the stability of tunnel face. Ideal properties of the excavated soil generally include low permeability, low inner friction, good plasticity and proper consistency to prevent the groundwater entering the TBM and to reduce the TBM drive torque of the TBM and abrasive wear. While natural soils rarely posses the ideal properties as mentioned above, soil conditioning is often used to modify the soil properties to improve the operation of EPB machine. Soil conditioning is an integral part of the excavation process in EPB drives. Soil conditioning is achieved by injecting conditioning agents, most commonly foams and polymers into the front of the cutter head, into the head chamber and if necessary into the screw conveyer. Depending on the ground conditions and soil type, different soil conditioning technologies can be applied in order to get the best results.

Based on the results of the subsurface investigation, it is considered that foams and or polymers can be used as the conditioning agents to non-cohesive granular soils in order to increase the plasticity and reduce the permeability of the muck as granular soils are generally highly permeable and have very poor plastic deformation behaviour. The cohesive deposits along the alignment typically have water contents close to or below their plastic limit and a soft to hard consistency. The application of certain type of conditioning agents is considered necessary for maintaining consistent face pressure and reducing the clogging effect.

The variety of soil conditioning agents and their combinations available are extensive and their selection should be based on the ground conditions. In order to ensure full flexibility of applying soil conditioning according to site needs, it is essential that an EPB TBM is equipped with an efficient foam generating and delivery system. Such system should include at least two additive dosing pumps so that the dedicated lines can be used for the foaming chemicals and for the polymer addictives.

5.3 Anticipated Ground Behaviour to Tunnelling

To describe the potential behaviour of the ground exposed without support, Terzaghi's "Tunnelman's Ground Classification System" (1950) has been used. Using this system, the clayey silt/silty clay and the tills with similar composition may be considered as "slowly ravelling" soil type with estimated stand-up times at the face ranging between less than half an hour and two hours. The cohesionless sand, silty sand, sand and silt, sand and gravel and gravelly sand strata below the groundwater table fall into the "rapidly ravelling" to "flowing" soil types and will require immediate and full support. When stabilized by dewatering, these soils at and exposed and unsupported face would behave like "slowly ravelling" and "rapidly ravelling". The estimated stand-up times at the tunnel face in "rapidly ravelling" and "slowly ravelling" soil types are about 2 to 10 minutes and between less than half an hour to two hours respectively. At the crown of the tunnel these soil types should be assumed not to have any stand-up time and thus, they would require immediate and full support. The deposits of silt, sandy silt and glacial till of similar composition above the water table could be classified as "slowly ravelling" ground with estimated stand-up time of about 10 to 20 minutes at an unsupported exposed tunnel face. Below the water table at low water pressures these soils are anticipated to be "rapidly ravelling". At water pressures larger than about 3m "flowing" ground conditions can be expected. Stand-up time at the tunnel face in the "rapidly ravelling" soils is estimated to be 5 minutes or less (depending on the percentage of clay size particles). The "flowing ground" will have no stand-up time and must be immediately supported.

The tunnel will be advanced through soft to firm clayey soils in the vicinity of west portal , where the stability number N_s at the tunnel face may approach or exceed the critical value for unstable "squeezing ground" conditions at the face. Under these circumstances a positive balancing pressure should be applied to support the face of the tunnel to avoid over-excavation and inward squeezing ground.

5.4 Ground Movements Inducing by Tunnelling

Sources that contribute predominantly to ground movements in tunnelling projects include the following: ground loss at tunnel face due to unstable face condition; ground loss along the TBM due to over excavation during the drive; radial convergence of the excavated openings due to the change of effective stress in the ground and consolidation of cohesive soils.

Extensive field measurements have shown that the settlement trough can be well characterised by the Gaussian distribution (Peck, Schmidt 1969; Mair and Taylor 1997), with the settlement given by Eq. 2:

$$S = S_{\max} \exp(-x^2/2i^2) \qquad [2]$$

The maximum settlement $S_{\max}$ is related to the volume loss $V_L$by Eq. 3:

$$V_L = 2.5 i S_{\max} / \pi (D/2)^2 \qquad [3]$$

Where:

S and $S_{\max}$ = ground surface settlement at any point from the tunnel centreline and settlement (maximum) above the centreline (m)
x = horizontal distance from the tunnel centreline (m)
i = center distance to the point of inflection of the settlement trough (m)
D = Tunnel diameter (m)

The volume loss which is expressed as a portion (percentage) of the theoretically excavated tunnel volume is a parameter of major importance in soft ground tunnelling. Recent research and tunnelling practice (Mair, 2008) have shown that the volume loss less than 0.5 to 1.0% are now achievable with the application of modern designed EPB machines. It is considered practical to control volume loss to the level of 0.5 to 1% for the ECLRT twin tunnels that are to be constructed with EPB machines, provided that reasonably good and consistent workmanship is applied by the contractor.

5.5 Modelling for Tunnel Liner Design Analyses

On the ECLRT project, a non-linear elastic model will better represent the stress-strain behaviour of the subsurface soil. The hyperbolic Duncan-Chang model (Duncan & Chang 1970) which can capture non-linear elastic behaviour of the soil, stress dependency and different soil response to loading and unloading conditions is recommended for the analyses. Further more, it is suggested that this model coupled with Mohr-Coulomb failure criterion in application is advantageous in that it uses parameters that can be well known by conventional tests and measurable with a simple failure criterion. The coupled model is not readily available in FLAC software and coding was implemented by Coffey Geotechnics (F. Badelow and D. Oliveira 2010) as a user defined model to be used by TTC.

5.6 Lateral Support for Deep Excavation

The construction of the proposed station requires deep excavation which is anticipated to be about a maximum of 28m below existing grade. Based on the inferred subsurface stratigraphy and condition, the excavation will be extended into different soil stratum including existing fill, localized silty clay till, interlayeyed water bearing non-cohesive and cohesive deposits. The bottom of excavation will be below the groundwater table. It is considered water-tight lateral support systems such as secant concrete caisson walls or slurry walls will be required for the excavation.

The installations of secant concrete caisson walls or slurry walls prior to tunnelling construction have several advantages compared with the conventional soldier pile and lagging system, including minimizing the potential ground loss and groundwater dewatering ingress during the excavation, higher wall stiffness and the potential to be part of the permanent wall. The contiguous concrete walls could be constructed as cut-off wall to stop the potential seepage of groundwater.

5.7 Head Walls

At the proposed stations on both sides where the box structure will connect the running tunnels, head walls are required. As the headwalls will later be bored through during the twin tunnelling, considerations should be given to construction of walls that incorporate "soft tunnel eyes" to facilitate the TBM breaking into the station.

Secant caisson and/or slurry walls discussed in the previous section can be designed as the head walls. Lean concrete or fibreglass reinforced concrete could be used as headwalls materials for easier TBM break-ins.

As an alternative, Jet grouting may be adopted in the headwall construction by which cemented or stabilized soil cement wall as overlapping jet-grouted columns are constructed. The jet-grouting design should determine the following aspects of stabilized zones including the minimum overlap thickness, the residual permeability, the minimum unconfined compressive strength (UCS) and the vertical and horizontal tolerances. Test or trial grouting is required to facilitate design and construction control.

6 DICUSSIONS ON TUNNELLING CONSTRUCTION

6.1 EPB TBM Drive

To control the ground loss at tunnel face, the face pressure must be maintained consistently within specific range suitable to the ground conditions. The working chamber pressure supporting the tunnel face should be regulated by controlling the rate of spoil discharge in relation to the advancing rate of the TBM.

Properly implemented soil conditioning is very important in maintaining pressurized head chamber and preventing groundwater from entering the chamber. Ground water conditions and their influence should be considered in determination of required face pressure.

It is important that the extraction of the spoil or muck is well controlled and synchronised with the speed of excavation. The excavation speed should match the liner erection speed as well to keep the excavated tunnel supported. The tail void and the annulus spacing should be grouted immediately or as early as possible.

6.2 Groundwater Control

Based on the available information most of the granular soils within the tunnelling zone are water bearing and saturated. As groundwater inflows into the tunnel face may cause loss of ground leading to excessive ground movements which may cause potential damages to surface structures and interruptions of surface traffic.

Recent developments in TBM such as EPB machines have made tunnelling in a wide variety of ground conditions with controllable ground loss achievable including tunnelling below groundwater table. In such case, hydrostatic pressures at tunnel face should be considered in determining the face pressure required for TBM drive.

6.3 Settlement Controls

One of the main tasks in settlement control is to control the ground loss during tunnel drive. Adequate face pressure must be maintained consistently during tunnel drive as fluctuation of chamber pressure may have significant effects on settlements for shallow tunnels.

In addition to underpinning of structures within zone of influence, compensation grouting as a very promising technique has been increasingly used for the mitigation of ground settlements, particularly in soft ground tunnelling through settlement sensitive zones. The principles, method and effect of compensation grouting were presented in the listed references (Mair & Hight 1994) and illustrated in Figures 3 and 4. This technique was previously adopted by TTC in tunnelling for a sewer relocation project below the existing Spadina Subway line (Boone, S. J., B. Pennington & et al. 1996).

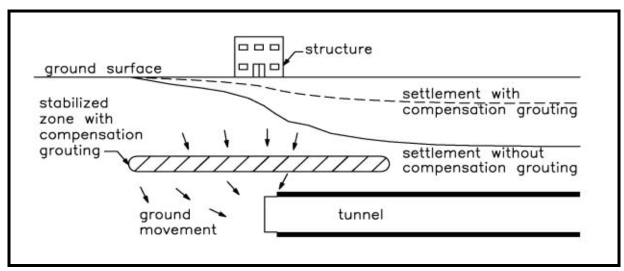

Figure 3. Effcet of compensation grouting on ground settlement

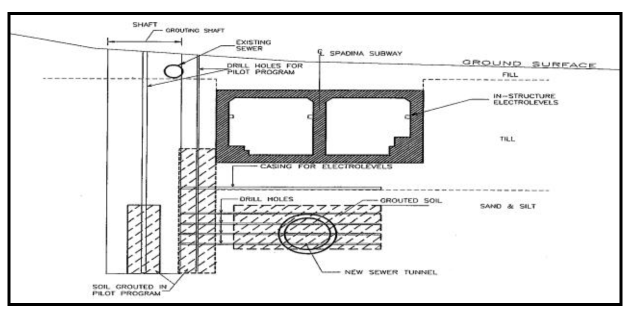

Figure 4. Scheme of compensation grouting for tunneling beneath the existing Spadina Subway (after S.J. Boone and et al 1996)

6.4 Boulders and Other Obstruction

Boulders and cobbles are commonly encountered in Quaternary deposits in Toronto based on local geology and our previous project experiences and their existence will hinder the tunnelling drive.

The recorded maximum boulder size founded in Toronto so far has been about 3 m in the maximum dimension. Boulder volume ratios (total boulder volume per volume of excavated earth material) BVR of 0.12% and 0.17% for interglacial deposits and glacial tills respectively have been recommended for TTC Subway projects such as the Sheppard Subway (S. J. Boone & J. N. Shirlaw 1996) and the Toronto-York Spadina Subway Extension (S.J. Boone & J. Westland 2008).

It is considered that EPB tunnel boring machines equipped with cutter head that are capable of dealing with mixed face conditions and boulders or rock cutting should be designed and used for the ECLRT twin tunnel construction. The TBM's should also provide protection devices to prevent large boulders from entering the machine.

7 SUMMARY

Transit tunnel projects in urban environment can be challenging from a geotechnical perspective. A thorough understanding of the subsurface conditions by implementing a comprehensive and appropriately scoped subsurface investigation is of prime importance. The geotechnical challenges and constraints on the design and construction of the proposed ECLRT project can be overcome by the application of various techniques including the earth pressure balance (EPB) tunnel boring machines (TBM) equipped with cutter head capable of dealing with boulders excavation; ground movement controls by proper soil conditioning and compensation grouting; groundwater control with contiguous or secant caisson wall or slurry wall as temporary shoring system and fibreglass concrete head walls for underground station construction.

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