# Challenges with tunnelling in saturated cohesionless soils

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ABSTRACT
This paper presents two case studies involving tunnelling in saturated cohesionless soils in the Greater Toronto Area. The first case discusses the installation of the 3.2 m diameter Langstaff Road trunk sewer using an earth pressure balance tunnel boring machine (EPB TBM). While tunnelling in saturated sand to silt, a significant amount of soil and water entered into the tunnel through the tailbrush seals between the tunnel segments and the tail can of the TBM. The TBM was flooded and had to be abandoned. The second case study examines the installation of a 1.5 m diameter feedermain in saturated sandy silt to gravel and sand crossing the Credit River by means of pipe ramming. During piping ramming, erosion of the leading edge soil plug in the pipe occurred, causing inundation of the launching shaft. The possible reasons for soil loss for the two cases are investigated and recommendations for tunnelling within saturated cohesionless soils are provided.

RÉSUMÉ
Cet article présente deux études de cas portant sur l’effet tunnel dans les sols pulvérulents saturés dans la région du grand Toronto. Le premier cas décrit l’installation de l’égout de tronc de Langstaff Road diamètre de 3,2 m en utilisant une terre pression équilibre tunnelier (EPB TBM). Alors que les tunnels en saturé sable, de limon, une quantité importante de sol et l’eau est entrée dans le tunnel à travers les joints de tailbrush entre les segments de tunnel et de la queue peut du Tunnelier. Le Tunnelier a été inondé et a dû être abandonné. La deuxième étude de cas porte sur l’installation d’une conduite de diamètre de 1,5 m en limon saturé à gravier et le sable traversant la rivière Credit par tuyau de refoulement. Au cours de la tuyauterie de refoulement, l’érosion du bord d’attaque sol bouchon dans le tuyau s’est produite, causant l’inondation de l’arbre de lancement. Les raisons possibles de la perte de sol dans les deux cas sont l’objet d’une enquête et des recommandations pour le creusement de tunnels dans les sols pulvérulents saturés sont fournies.

## 1 INTRODUCTION

Saturated cohesionless soils including gravel, sand and silt are widely distributed in the Greater Toronto Area (GTA), Canada. While tunnelling in saturated coehsionless soils, quick conditions can occur, manifesting as to loss of ground and short stand-up time. Additional serious consequences may occur if the temporary tunnelling support is not sufficient to resist the static groundwater pressure or seepage pressures. The soil loss occurring at the tunnel face results from loss of confinement, dilation of the soil mass, coupled with water seepage forces generated by the water flow toward the free face.

Ensuring tunnel face stability is directly related to the safe and successful construction of a tunnel. The face stability depends on various factors, such as soil strength parameters (friction and cohesion), groundwater pressure and soil permeability, tunnel diameter, tunnelling and support methods. Analysis methods to evaluate the face stability have been suggested for cohesive soils (Broms and Bennmark, 1967; Davis et al. 1977) and cohesionless soils (Atkinson and Potts, 1977; Leca and Dormieux, 1990). Effect of seepage forces on the tunnel face stability has been studied using numerical analysis methods by Pellet et al. (1993) and Lee et al. (2003). However, there is a gap which exists between the research and engineering practice.

The load on the temporary support segments and joints is dependent on the tunnelling ground and groundwater conditions and is assumed to be the lesser of full overburden or 1 to 2 times the tunnel diameter for a circular tunnel in soil (FHWA, 2009). The groundwater pressure imposed on the joints for tunnelling below the groundwater table could be much higher than the effective earth pressure.

This paper presents two case studies of tunnelling in saturated cohesionless soils in the GTA: (1) Langstaff Road trunk sewer tunnel; and (2) Heritage Road feedermain crossing the Credit River. During tunnelling in saturated sand to silt for the Langstaff Road trunk sewer, a significant amount of soil and water entered into the tunnel through the tail brush seals between the tunnel segments or at the tail of the tunnel boring machine. During pipe ramming for the Heritage Road feedermain crossing the Credit River, soil and water flowed into the launching shaft.

The possible reasons for the soil loss in the two cases are investigated. Lessons from the two cases are summarized and recommendations for tunnelling within saturated cohesionless soils are provided.

## 2 LANGSTAFF ROAD TRUNK SEWER TUNNEL

The Langstaff Road trunk sewer was proposed to relieve pressure on existing sewer lines in the City of Vaughan, Ontario. The sewer consists of 3.2 m diameter concrete segment pipes installed at a depth of about 22 m below the existing ground surface. Tunneling was by means of an earth pressure balance tunnel boring machine (EPB TBM). The concrete segments were used as both temporary and permanent tunnel support. During the tunnelling in saturated sand to silt east of Dufferin Street and north of

Highway 401 in 2008, a significant amount of soil and water entered into the tunnel through tail brush seals between the tunnel liner segments or at the tail can of the TBM, filling a reported 300 m length of the tunnel. The TBM had to be abandoned and a 25m by 30m surface expression of a sinkhole formed at ground surface as shown in Figure 1.

Figure 1. A sinkhole above the TBM

The sinkhole was backfilled with approximately 267m$^3$ of unshrinkable fill, followed by an additional 600 to 800 m$^3$ of sand on the same day and following weekend. A double bulkhead within the tunnel, approximately 300m west of the sinkhole location, was constructed at the same time. A rescue shaft was constructed to recover the buried TBM and to complete the installation of the trunk sewer in the disturbed section (Cao, et al. 2010).

The possible reason for soil and water leakage through seals between segments or at the tail of the TBM was investigated and a discussion on the remedial work is provided in the following sections.

## 2.1 Soil Characteristics

Subsurface investigations by means of boreholes were carried out from the existing ground surface prior to the shaft construction as well as with borings advanced inside the shaft after the excavation reached the bottom of the shaft.

Two boreholes were drilled just beyond the edge of the sinkhole and one borehole inside the sinkhole prior to shaft construction. All of the boreholes were sampled in association with the ASTM D1586 Standard Penetration Test (SPT) method using the conventional 50mm diameter split spoon sampler at depth intervals of 0.75 to 1.5 m.

All of the boreholes encountered a variable thickness of surficial pavement structure/fill deposits overlying a predominantly sandy silt to silty sand glacial till deposit extending approximately 5 to 6m below grade. Underlying the upper glacial till are predominantly cohesionless deposits ranging in texture from silt, sand and silt to gravelly sand. The SPT 'N' values measured in the cohesionless deposits in the two boreholes drilled outside the sinkhole were in excess of 50 blows per 300mm penetration and thus, considered to be in a very dense state of packing. No voids or zones of apparent loosening were encountered or inferred in these two boreholes drilled outside the sinkhole. At a depth of approximately 4.3m in the borehole advanced inside the sinkhole, the augers dropped approximately 0.76m, suggesting a possible zone of loosening or void at this level. Groundwater was encountered within all of the boreholes at a depth of approximately 5.5 to 6 m below the existing ground surface.

Figure 2 shows the simplified soil profile and variation of SPT 'N' values with depth. The groundwater level (GWL) is also shown in Figure 2.

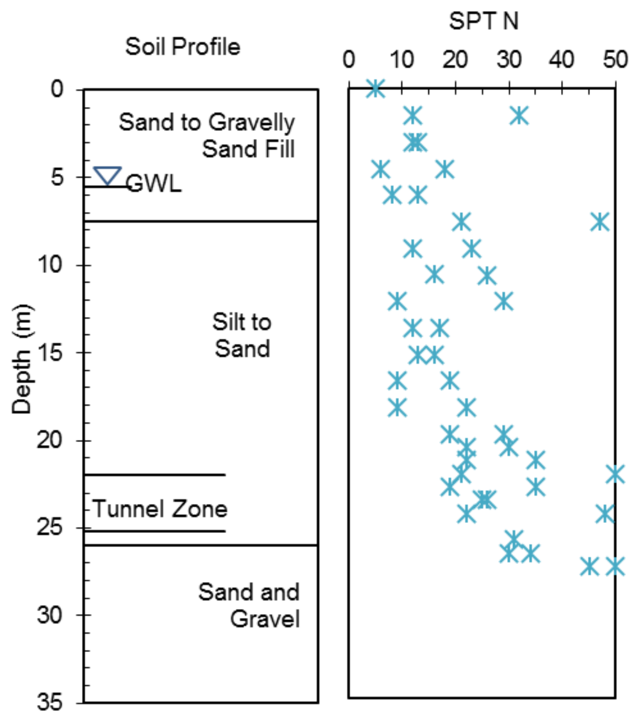


Figure 2. Soil profile and SPT 'N' values at the Langstaff Road trunk sewer site

## 2.2 Joint Seal Failure and Discussions

At the tunnel face, the lower (safe) and upper (unsafe) bound support pressures, $\sigma_T$, recommended by Atkinson and Potts (1977) are as follows, respectively,

$$\sigma_T = 2\gamma' R K_p / (K_p^2 - 1) + u \quad [1]$$

$$\sigma_T = 0.5\gamma' R(1/\tan\phi + \phi - 0.5\pi)/\cos\phi + u \quad [2]$$

where $\gamma'$ is the effective unit weight of soil; R is the radius of tunnel lining; $K_p = (1 + \sin\phi)/(1-\sin\phi)$, the coefficient of passive earth pressure; $\phi$ is the maximum angle of shearing resistance; and u is the pore water pressure or seepage pressure, which is between 22% and 28% of the hydrostatic pressure for the drainage type and waterproof type of tunnel face, respectively based on the study of Lee et al. (2003).

Based on the borehole information, the soil consists of saturated compact to very dense silt to sand within the tunnel zone. The tunnel centroid is about 23.6 m below the existing ground surface and approximately 18.1 m below the groundwater table. Assuming $\phi$ is 32º, $\gamma$' is 11 kN/m$^3$ for the compact to very dense silt to sand and a waterproof type of tunnel face, the required lower (safe) and upper (unsafe) bound support pressures for the tunnel face is 62 kPa and 56 kPa, respectively based on Eqs. 1 and 2 and assuming that the seepage pressure is 28% of the hydrostatic pressure.

For tunnelling in the saturated silt to sand below the groundwater table, the ground can be classified as a potentially "flowing ground condition" (FHWA, 2009). The load on the tunnel support is the lesser of full overburden pressure or 2 times the tunnel diameter for tunnelling in potentially flowing ground conditions. The total vertical overburden at the tunnel crown located at a depth of 22 m below the existing ground surface is 462 kPa. The effective vertical load of 2 times tunnel diameter is 70 kPa and the groundwater pressure at the tunnel crown is 178 kPa, making the total vertical pressure of 248 kPa, which is much greater than the support pressure for the tunnel face of 62 kPa.

The joint seals at the tail of the boring machine need to be designed at least for a pressure of 248 kPa, which is about four times the support pressure for the tunnel face. The joint seal between the tunnel segments or at the tail of the boring machine would be fail if only designed for the groundwater pressure of 178 kPa or the support pressure for the tunnel face of 62 kPa.

### 2.3 Remedial Work

In early 2009, a 5 m by 30 m recovery shaft was constructed at the location of the sinkhole to remove the buried TBM. The shaft consisted of contiguous 1 m diameter caissons. The caisson walls were toed at 25.5 m below the existing ground surface. The buried TBM was found and removed when the excavation inside the shaft reached about 21.7 m below ground level and then an approximately 0.6 m thick concrete slab was cast on the bottom of the excavation. It was noted that the collapsed tunnel segments, cables and some equipment were not removed and remained in the soils beneath the shaft base. Construction dewatering was commenced to draw down the groundwater levels to depths of 14.5 to 20.5 m below the existing ground surface.

Shortly after the casting of the concrete base slab inside the shaft in May 2009, permeation grouting using ordinary Portland cement was carried out to improve the soils below the concrete slab inside the shaft since these soils would have to support the trunk sewer and shaft backfill. The proposed grouting program called for 80 grout holes at a spacing of 1.5 m by 0.88 m. The grouting was proposed to extend to 8m below the base slab surface Unfortunately, grouting was unsuccessful due to insufficient soil information below the base slab. Of forty primary grouting holes, nineteen holes were grouted to depths ranging 5 to 8 m below the concrete slab, seventeen holes were not grouted but simply backfilled due to encountering obstructions at depths ranging from 1 to 3.6 m below the concrete slab, and four holes were not drilled. Of the forty secondary holes, three were grouted to a depth of 8m below the concrete slab, one hole was not grouted but backfilled due to an obstruction found at a depth of 1m below the concrete slab, and the remaining thirty six holes were not grouted.

The area in which obstructions were found within 3.5m below the concrete slab coincided with the area where equipment and the precast concrete segmental tunnel liners were buried. The soil conditions below and within the buried equipment and segments could not be explored.

In order to establish soil conditions under the base slab, two boreholes were drilled from the concrete basal slab to depths of 3.5 to 9.7m below the slab, from within the shaft. The ground investigation showed that it was unlikely that voids existed below the buried liner segments and equipment. However, the soils below the base of shaft to 5m below the base were considered to be disturbed relative to the original condition of the ground.

Based on the investigation results, the existing ground conditions below the base slab of the shaft were not deemed to be sufficiently competent to support the weight of the pipe and over 20m of backfill without the risk of unacceptably large settlements (greater than 25 mm). This finding turned over the initial design assumption that soils below the base slab improved by grouting could support the pipe and backfill.

Two options were proposed to solve this problem. The first option was to support the pipe and backfill by the shoring caisson walls which were socketed into dense to very dense sandy soils and were considered to have sufficient capacities to support the backfill. However, this approach would require specialized connections between the pipe or grade beams and the walls and was therefore deemed uneconomical.

The second approach was to consider the pipe supported by both base slab and the caisson walls. A 30 MPa concrete mass to form the cast-in-place sewer within the shaft would replace the tunnel. The thickness of the concrete mass was approximately 4.6 m including an overlying, 1.4m thick concrete cover. Then the remainder of the shaft would be backfilled with 0.2 MPa unshrinkable fill. The soil below the base slab was considered to have a bearing capacity of 150 to 200 kPa under the serviceability limit state. The structural connection between the base slab/mass concrete and walls through the welding of steel struts to the king piles was sufficient to transfer the remaining loading to the caisson walls. The estimated settlement under the load of backfill was less than 25 mm. This approach was simple and easily constructed and thus adopted. In order to verify the approach, settlement monitoring was carried out through the whole process of pipe casting and shaft backfilling

A comprehensive ground movement monitoring program was carried out prior to and during the shaft construction and after backfilling. The monitoring results confirmed the successful remedial work. During backfilling the shaft, the maximum settlement at the shaft base was not more than 7mm and the maximum settlement at the cast-in-place sewer pipe was not more than 6 mm. After backfilling, the maximum settlement at the cast-in-place sewer pipe was not more than 3 mm movement.

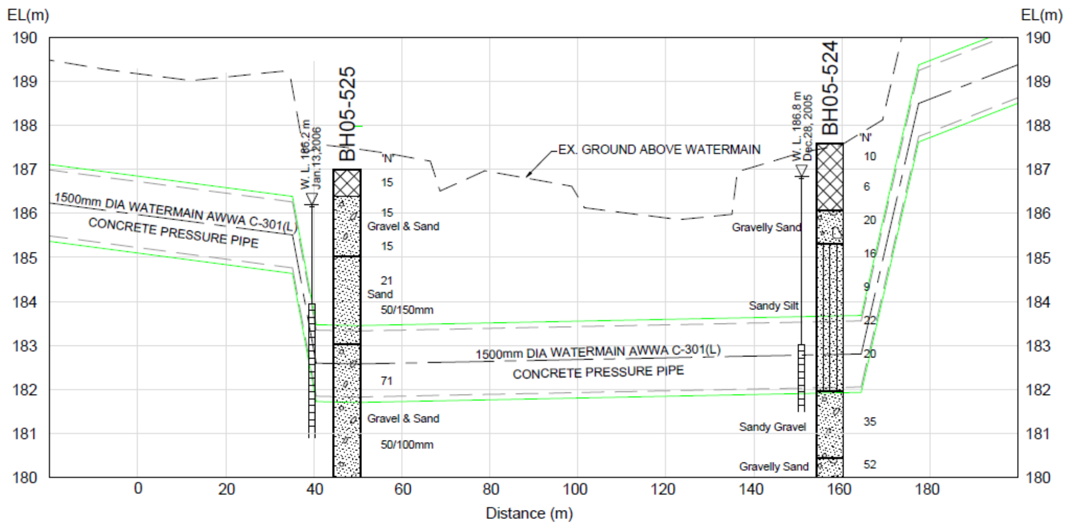


Figure 3. Generalized soil profile at the Credit River site

## 3 HERITAGE ROAD FEEDERMAIN CROSSING THE CREDIT RIVER

For the installation of a 1500mm diameter feedermain crossing beneath the Credit River in 2006 in the City of Brampton, Ontario, pipe ramming was used to install a 150 m long steel liner in saturated sandy silt to gravel and sand. The cohesionless soil within the ramming pipe could not provide a sufficiently resistant and impervious soil plug at the leading edge of the ramming pipe. The pipe ramming contractor had not installed an internal pipe bulkhead as a backstop to the soil plug. During ramming below the watercourse, internal erosion of the soil plug occurred, leading to rapid ingress of soil and water entering into the launching shaft through the ramming pipe.

### 3.1 Soil Characteristics

One borehole was drilled at each side of Credit River, near the launching and receiving shafts, respectively. The boreholes encountered a 0.6 to 1.5 m thick loose sandy silt fill or stiff clayey silt fill overlying predominantly cohesionless deposits of sandy silt, sand, sandy gravel, sand and gravel and sandy gravel, extending 14.0 to 18.5 m below the existing ground surface, the exploration depth of the boreholes. The SPT 'N' values measured in the cohesionless deposits in the two boreholes ranged from 9 to more than 50 blows per 300 mm penetration and thus, the deposits were considered to be in a loose to very dense compactness condition, but generally in a compact to very dense condition since the presence of gravel sizes may have artificially elevated the 'N' values in some samples. The groundwater table measured from the monitoring wells lay at a depth of 0.3 to 0.4 m (El. 187.3 to 186.6 m) below existing ground surface. Figure 3 shows a generalized soil profile at this location.

The hydraulic conductivity (permeability) of the granular deposit, as estimated from pumping tests was $3.8x10^{-2}$ cm/s.

### 3.2 Suitability of Tunneling/Trenchless Methods

It goes without saying that extremely challenging ground and groundwater conditions were expected at this crossing site, whether the pipe was constructed by means of open cutting in cofferdams, or using trenchless methods.

The casing centroid was proposed at approximately 3.4 m below the riverbed and approximately 4.4 m below the surface water level. The saturated sandy silt to sand gravel was classified as a "flowing ground condition" under the behaviouristic classification system for tunnelling (FHWA, 2009).

Traditionally, in the GTA, prior to the introduction and acceptance of microtunneling, jacking and boring methods were commonly used for the installation of feedermain pipe above the groundwater table over relatively short distances of say 50m or less. Jacking and boring is a process consisting of constructing a temporary horizontal jacking platform and a starting alignment track in a launching shaft at a desired elevation. The casing is then jacked along the starting alignment track towards the receiving shaft with simultaneous excavation of the soil being carried out by a rotating cutting head (i.e. continuous flight augers) within the casing annular space. The excavated soil is transported back to the launching shaft by auger flights rotating inside the casing. Jacking and boring generally provides limited tracking and steering capability as well as limited or no support to the excavation face. The jacking and boring method is not suitable for the installation of the pipe crossing the river under potentially "flowing ground condition".

Horizontal directional drilling (HDD) could be considered for the pipe crossing the Credit River. The HDD method consists of pilot boring, back reaming and product pipe pull-back. Drilling begins with a small diameter pilot hole along a designated alignment, using flexible drill rods with remote controlled steering system. After the pilot boring, a back reamer is installed and drilled back through the pilot hole to achieve the required diameter for the pipe to be installed. Typical ratio of diameter of reamer to pipe is 1.5. Drilling fluid is used to prevent the collapse of borehole as well as providing a lubricant for the drilling and flushing spoils. The HDD method is often used in the GTAA successfully for smaller diameter pipe installations with diameter less than 1.2 m. Due to hydraulic reasons and the Region's lack of track record with using HDPE for trunk watermains, the civil design engineer rejected the concept of bifurcating the watermain over the crossing length. There were additional obvious concerns that the drilling fluid could not support the bore crown/walls in gravelly and potentially cobbly soils.

Microtunneling is a method whereby reinforced concrete jacking pipe (RCP) or steel casing is pushed into a bore mined remotely using a tunnel boring machine head fixed to the lead pipe segment. Operation of the microtunnel boring machine (MTBM) is done from the launching shaft. Spoils removal, using the slurry shield MTBM method is performed by mixing the excavated soil in the front chamber of the MTBM head to the consistency of a slurry with conditioning using bentonite, water and other soil conditioning agents and then pumping this slurry back to the launching shaft where it is de-sanded and thickened. The rate of slurry removal from the chamber is carefully controlled and matched to the advance rate of pipe jacking such that the predicted lateral earth pressures are balanced with the slurry pressure. A large laydown area is needed for the de-sanding support plant. The RCP segments must be precast by a manufacturer with previous experience in jacking pipe. Close quality control is needed in the production of this product. Bentonite lubrication of the annular space is used to reduce frictional resistance to jacking. Sealed shafts are most typically used in MTBM operations. This can consist of cast-in-place concrete circular shafts constructed top-down, excavated in–the-wet progressively as the concrete ring segments are cast and pushed down. Use of sealed shafts will mitigate, to a large extent, the requirements for dewatering and loss of ground issues, provided a base plug of concrete is cast in the wet by tremi-methods. The MTBM seems to be best method for the installation of the feedermain crossing the river. Unfortunately, the MTBM was not widely used in the GTA 10 years ago. Today, it is the method of choice for such challenging condition.

Consideration was given to using pipe ramming (PR) to install a steel casing under the river. The dynamic force transmitted by a percussion hammer attached to the end of a casing pipe is used to ram pipe. There are two major categories of pipe ramming: closed-face and open-face (Iseley and Gokhale, 1997). A cone-shaped head is welded to the leading end of the first segment of pipe to be rammed in closed-face PR. The head penetrates and compresses the surrounding soil as the casing is rammed forward. Typical diameters of pipe installed by closed-face PR are 100 to 200 mm. A bore hole of the same size as the casing can be cut in the open-face PR, which allows most of the in-line soil particles to remain in place, with only a small amount of soil compaction occurring during the ramming. Typical diameters of pipe installed by open-face PR are 0.1 to 1.5 m. After the casing installation process is complete, the soil that has entered the casing is removed by applying compressed air or water from either end for small-diameter casings. For large casings, augers, mini-excavators, or hand work can be used to mechanically remove the soil from the inside of the pipe.

Pipe ramming below the groundwater table, especially in sands, can cause a problem of flooding, because groundwater can easily flow through spoils in the pipe and enter the insertion pit. The amount of water and sand entering the pipe can be reduced with the installation of plugs at the front end of the pipe. The plug can be created by either filling the pipe with sandbags or by leaving the spoil in the front section of the pipe. In addition, a mechanical seal, usually composed of a rubber flange, can be mounted to the wall of an insertion pit to guard against groundwater flooding. Such seal also prevents an inflow of drilling fluids into the insertion pit during the ramming operation.

The typical open-face PR procedure includes: (1) constructing launching and receiving shafts; (2) installing a band on the leading edge of the casing, placing the casing in launching shaft, and adjusting for desired line and grade; (3) attaching the hammer device and connect to pneumatic or hydraulic power source; (4) initiating the drive; (5) after each segment installation, removing the hammer, welding another pipe segment to the end of the previous casing, and repeating the cycle until the installation is complete; (6) cleaning out casing; and (7) remove the equipment.

The challenge for the PR crossing the river was to determine the resistance force of the plug and to develop a sufficient length of soil plug to prevent development of quick conditions/internal erosion. In order to reduce the seepage force and increase the resistance, a recommendation was made to bifurcate the feederrmain into two smaller diameter pipes. It was also recommended that the launching and receiving shafts be constructed as "watertight" cofferdams consisting of interlocking steel sheet piles and the toe of the sheet piles be driven at least 4 m below the excavation level to guard against heave failure with dewatering by means of deep wells, particularly near the entry points where the pipe would enter into the shafts.

### 3.3 Bore Face Failure and Discussions

The contractor chose to ram a 1.5 m diameter steel casing crossing the river using the open-face PR technology and the PR methodology was not reviewed by the geotechnical engineer. There was no internal bulkhead nor any provision to install such bulkhead inside the pipe. The shaft was constructed using interlocking steel sheet piles, but the details were not reviewed by the geotechnical engineer. During the PR while passing beneath the watercourse, rapid ingress of sand/silt/water slurry flowed into the launching shaft through the jacking pipe.

The ramming pipe centroid was about 3.4 m below the riverbed and approximately 4.4 m below the surface water table. Assuming that $\phi$ is 32º and $\gamma'$ is 11 kN/m$^3$ for the compact to very dense sandy silt to sandy gravel, the required lower (safe) and upper (unsafe) bound support pressures for the tunnel face is 15 kPa and 12 kPa, respectively based on Eqs. 1 and 2 with the consideration of 22% of the hydrostatic pressure based on the study of Lee et al. (2003). The required lower support force on the tunnel face is about 26 kN. For the PR inside the cohesionless soils, the support force can only be provided by the friction, F between the lower half part of casing and plug soil, which can be estimated as follows

$$F = 0.5\gamma D^2 L \tan\delta \qquad [3]$$

where D is the diameter of steel casing, L is the length of plug soil; $\gamma$ is the total unit weight of plus soils and $\delta$ is the friction angle between steel casing and plug soils. Note that the steel casing is impermeable and thus the total unit weight can be used for the estimation of the friction.

Assuming $\delta$ is 24$^o$ (75% of the soil friction angle) and $\gamma$ is 21 kN/m$^3$, the required minimum length of soil plug without a safety of factor is about 2.5 m, provided that the pore pressure at the end of casing had dissipated when the hammer was stopped during the break in ramming while welding the second segment pipe was in progress.

In order to further study the seepage force within the steel casing, a seepage analysis was performed using a two-dimensional finite element (FEM) computer program (RS2 by RocScience). The hydraulic conductivity of soil of 3.8x10$^{-4}$ m/s was determined from the pumping test. The steel casing and sheet pile were assumed as impermeable with a hydraulic conductivities of 10$^{-10}$ m/s. The groundwater level was assumed at El. 181.4 m (i.e. 0.5 m below the shaft base level) and the surface water level was taken at El. 187 m based on the water level measurement. The distance between the launching shaft wall and the river was approximately 25 m. Figure 4 shows the finite element mesh and the hydraulic conductivities of soil and steel used in the analysis. The output of seepage analysis showing the pressure head within the driving casing is provided in Figure 5. The seepage force at the tunnel face is about 15 kPa, or 34% of the hydrostatic pressure, which is higher than that recommended by Lee et al. (2003).

Based on the seepage analysis, the average seepage force within a 6 m long, 1.5 m diameter casing is about 22 kN and the resistance force estimated from Eq. 3 is 63 kN. The factor of safety against the quick condition is 2.9. This means that the quick condition will not happen if the pore water pressure at the end of driving casing is released and a typical 6 m long pipe segment is used.

If the pore water pressure at the end of the driving casing is not released, the required lower (safe) and upper (unsafe) bound support pressures for the tunnel face is 85 kN and 80 kN based on Eqs. 1 and 2 using hydrostatic pressure, which are greater than the resistance force provided by the 6 m soil plug of 63 kN. The insufficient resistance provided by the soil plus could be the reason that quick conditions happened during the PR crossing of the river.

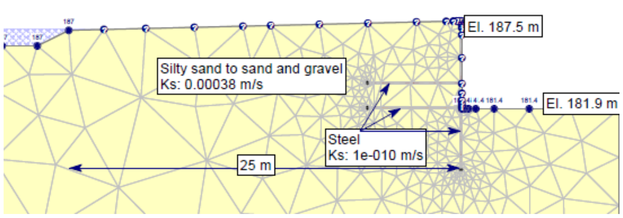


Figure 4. FEM mesh for seepage analysis

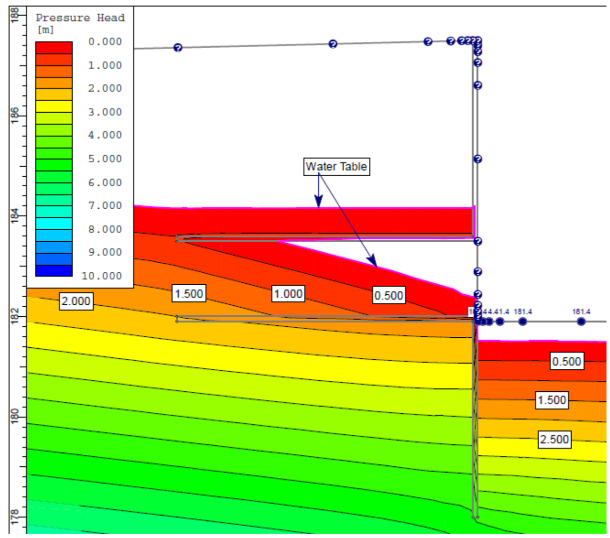


Figure 5. Distribution of pressure head from seepage analysis

## 4 CONCLUSIONS

From the two case studies, the following conclusions and recommendation can be made for tunneling/trenchless boring in potentially flowing ground conditions:

(1) Tunnel support should be designed for the lesser of full overburden pressure or 2 times tunnel diameter plus the full hydrostatic water pressure, which could a few times the required support pressure for the tunnel face.

(2) The pore water pressure inside the ramming casing needs to be relieved prior to removing driving hammer.

(3) For successful tunnelling, it is critical that the geotechnical engineer be engaged to review contractor's tunnelling methodology.

## ACKNOWLEDGEMENT

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