Secant Piled Shaft Construction and Microtunnelling in Sand

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Challenges from North to South
Des défis du Nord au Sud

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ABSTRACT
This paper outlines the installation of three 18m deep shafts up to 8.2m internal diameter, and two tunnels with an internal diameter of 1.2m in sand with the water table near the ground surface. The shafts were constructed using the secant piled method while the tunnels were constructed via microtunnelling. The challenges and problems associated with these difficult installations are presented, as well as the solutions used to overcome and mitigate the issues encountered.

RÉSUMÉ
Cet article décrit l'installation de trois puits de 18 m de profondeur et de diamètre interne allant jusqu'à 8,2 m, ainsi que deux tunnels de 1,2 m de diamètre interne dans le sable avec une nappe phréatique près de la surface. Les puits ont été construits en utilisant la méthode des pieux sécants, tandis que les tunnels ont été construits par la technique du microtunnelier. Les défis et les problèmes associés à ces installations complexes, ainsi que les solutions utilisées pour surmonter et atténuer les difficultés rencontrées sont présentés.

1 INTRODUCTION

The project location at the intersection of Mayfield Road and Kennedy Road in Brampton, Ontario has notoriously poor ground conditions as documented by previous works in the area. Extensive sheet piling was required through the intersection as part of a previous project in order to support the widening of Mayfield Road, as well as supporting the underground infrastructure. The sheeting and tiebacks remain in the ground.

The shafts and tunnels are also located in an environmentally sensitive area which is home to a provincially significant wetland with peat/bog area under the jurisdiction of the Toronto and Region Conservation Authority (TRCA).

The launching shaft for both tunnels was an 8.2m internal diameter 18m deep shaft while both receiving shafts were 4m internal diameter and 18m deep. Figure 1.1 shows a plan of the project location with the launching shaft identified as "LS" and receiving shafts identified as "R1" and "R2". Both tunnels advanced from the LS. The launch shaft was converted to a permanent valve chamber with a 7m internal diameter upon completion of the tunnel. The two receiving shafts were backfilled on completion. The tunnel lengths were 91m from the launching shaft to R1 and 78m from the launching shaft to R2.

Owing to the previously placed sheet piling (shown as a red dashed line in Figure 1.1), the depth of the tunnels had to be adjusted relative to conventional watermain depth-of-cover requirements. The steel sheet piling presented a physical barrier to any tunnelling method, resulting in the tunnel alignments being deepened so as to pass underneath this man-made obstruction. Deep shafts were therefore needed in order to launch and receive the tunnels. The tunnels were to act as a casing for 750mm diameter Concrete Pressure Pipe (CPP) watermains and would connect all sides of the intersection where CPP had been previously laid. These interconnections were the remaining pieces of work required to commission this part of the water supply system.

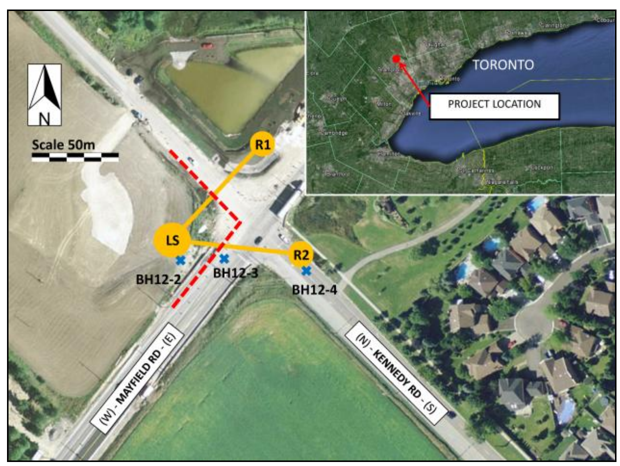

Figure 1.1 - Plan of Project Location in Brampton, ON

All three shafts were constructed using the secant piled method. Concrete for the piled shafts was, in all cases, placed underwater via the tremie method using a modified 42m concrete pump truck with 20m of discharge hose attached to it. A basal tremie plug was poured for all three shafts to mitigate basal boiling.

Given the saturated, cohesionless nature of the soils in the tunnel horizons, the use of a closed face Tunnel Boring Machine (TBM) was a requirement specified by the

Regional Municipality of Peel. This system has the ability to support the face of the excavation thereby preventing ground loss during tunnel advance.

2 DESCRIPTION OF SUBSURFACE GEOLOGY

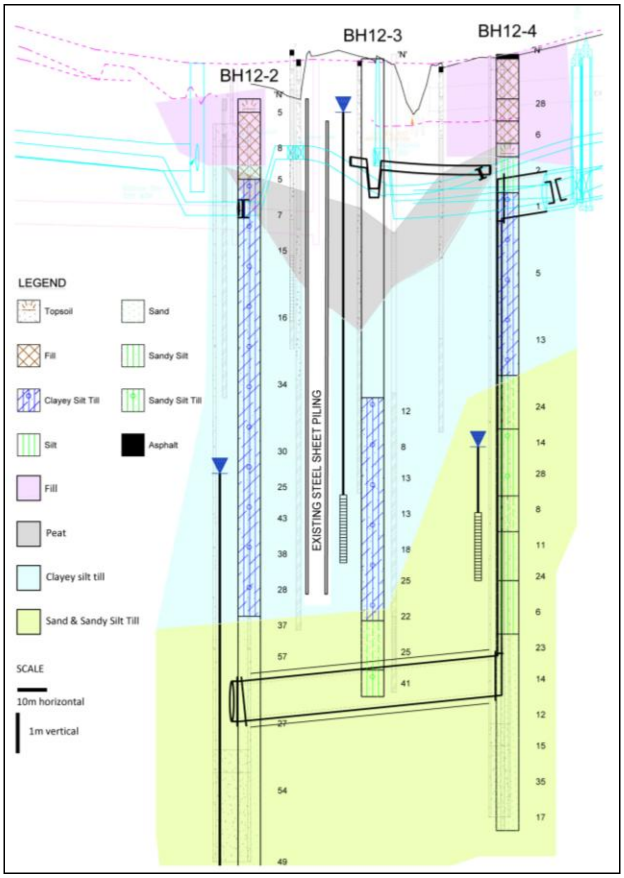

Figure 2.1 - Generalized subsurface geology

The generalized subsurface geology at the project location is illustrated in Figure 2.1. At grade, a variable thickness of surficial fill material is underlain by a layer of peat approximately 2.0m thick; however, locally the peat was up to 3.5m in thickness. Perched groundwater is present in both of these latter deposits. Below the peat layer, lies a cohesive, very stiff to hard glacial till of primarily clayey silt texture (plasticity index ~ 8%) which forms an aquitard. Beneath the glacial till cap, lie a sequence of stratified cohesionless deposits ranging in texture from silt with some sand to fine to medium grained sand with trace silt. Particle size distributions for samples of these cohesionless layers are depicted in Figure 2.2. SPT (uncorrected) N values for the cohesionless deposit were recorded as being in the 12 – 57 range at the tunnel horizons.

The tunnel horizons lie wholly within the cohesionless deposits. Piezometric heads within the cohesionless deposits were measured to lie approximately 3 to 5m above the upper surface of the deposit which represents a head of 6m above tunnel invert.

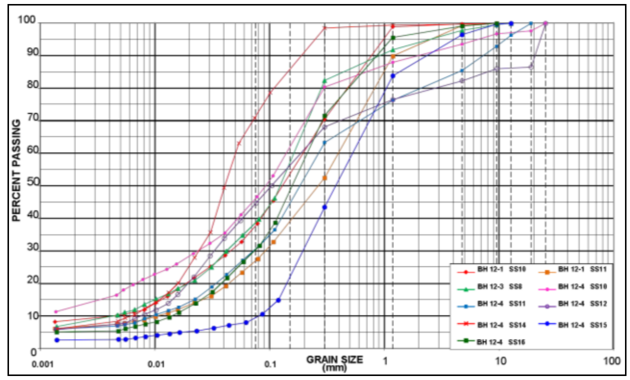

Figure 2.2 - Particle size distribution for cohesionless layer

3 SHAFT CONSTRUCTION

3.1 Piling and Placing Concrete

As mentioned in Section 1, the shafts were built with secant piles. 1.18m diameter piles were bored to a depth of 18m using a Bauer BG36H piling rig. Excavation of the material below the ground water table was done with a single cut drilling bucket. A steel liner was advanced ahead of the drilling bucket to prevent ground loss. The drilling bucket then excavated the material inside the steel liner. The concrete strength in both the primary and secondary piles was 15MPa. No additional reinforcement was used in any piles and the shaft was self-supporting. There were no supporting walers or tiebacks used. All piles were advanced through a cast in place surface drilling template to ensure accurate positioning of the pile centroids and the Bauer BG36 was equipped with on-board inclination sensors to ensure good interlocking.

For placing the concrete into the excavation, a specially modified 42m (boom length) concrete pump truck with 20m long tremie hose was used. A photograph of the piling rig and the modified concrete pump truck can be seen in Figure 3.1.

3.2 Shaft Excavation

All excavation was done from the ground surface in two stages. The first stage involved using a standard 50 tonne excavator reaching to a depth of approximately 9m inside the secant piles. Once the standard excavator had run out of reach, a telescopic clamshell mounted to the body of a 50 tonne excavator was used to complete the excavation to a depth of 18m. A photograph of this telescopic chamshell is shown in Figure 3.2.

After a depth of approximately 8m, all excavation was done (intentionally) underwater. Given that groundwater was being removed from the excavation with every clamshell cycle, water was returned to the shaft on a continuous basis via a nearby fire hydrant supply in order to keep a positive head inside the excavation. This would help prevent the ingress of sand or silt under the wall due to the inflow of ground water. The target was to keep the water elevation inside the shaft approximately 3m higher than that of the outside ground water.

Figure 3.1 - Bauer BG36H & modified 42m (boom length) concrete pump with a 20m long tremie hose attached

3.3 Tremie Plug Installation

Once the shaft interior had been excavated to the correct elevation as measured using a weighted tape around the edges, it was ready to be plugged to provide a watertight seal to the secant piles. There was approximately 12m head of water in the shaft which was turbid from all the agitated particles stirred up by the excavation process. It was therefore impossible to see what was happening at the bottom, or if the excavation had been carried out uniformly. The weighted tape provides only a snapshot of the elevation at a particular location around the edges. In order to remove as much suspended solids from the water as possible, an environmentally-benign liquid polymer was added to the shaft water and left to flocculate overnight. When the polymer was added to the shaft water, the telescopic clamshell was used to agitate and mix the polymer into the standing water. The following morning, an approximately 1.5m high sediment blanket (as measured by construction divers) had accumulated on the excavation base. The accumulated sediment was carefully removed with the clamshell so as to not cause turbulence thereby re-agitating the settled material into suspension. After removal of the sediment, construction divers were sent to inspect the uniformity of excavation base. In order to ensure a good bond between the tremie plug and the secant piles, the divers also cleaned any material which was sticking to the shaft walls using a water jet. The divers also checked the interlocking of every secant pile below the water to ensure that no voids were present.

After the walls had been cleaned and the base evenly excavated with guidance from the divers, a non-woven geotextile fabric was placed at the shaft base and then covered with approximately 300mm of clear stone. This was done firstly to act as a physical barrier between the tremie plug and the cohesionless material, and secondly to avoid introducing further solids into suspension from the jetting action of the concrete exiting the tremie pipe.

Some uncertainties associated with pouring tremie concrete include how far a specific mix will flow, how well the tremie concrete will bond with the secant piles (or existing vertical walls), does the tremie pipe remain completely embedded in the concrete and will any localized washout of cement occur due to the accidental removal and reintroduction of a tremie pipe. Because of these unknowns, extensive consultation was undertaken with the concrete supplier and a specialized mix developed to help answer as many of these questions as possible. The resulting mix was a self-consolidating, extra high flow cohesive concrete.

Figure 3.2 - Telescopic clamshell excavator capable of digging to a vertical depth of 26m

A 42m concrete pump truck was used to place the tremie plug for all shafts. The plug would have to be done as one continuous pour in order for it to be truly monolithic and most importantly, successful. With a traditional hopper fed tremie pipe suspended from a crane, the tremie can be simply lowered further into the previously poured concrete so as to stop the inflow to keep the feed pipe charged – this eliminates the possibility of water infiltrating the feed pipe and washing out concrete. With the concrete pump, it is more difficult to gauge whether the feed pipe is constantly charged with concrete or not. Measures taken to help reduce the risk of the feed pipe being accidentally removed from the pour or running out of concrete included marking the boom height relative to the standing water in the shaft as well as using an air bladder valve to close off the discharge point of the pump.

This bladder valve was used in case the concrete supply was interrupted or if for any reason the tremie pipe has to be removed and reintroduced to the pour. Marking the boom of the pump serves as a constant reference point as the water level in the excavation will rise at the same rate at which concrete is introduced meaning this mark should always be the same height above the water surface. The air bladder valve resembles a blood pressure monitor that fits around the arm and acts to constrict the flow inside the tremie pipe when inflated.

With all of these measures in place, the tremie plug was poured and then left to cure for 3 days. This gave the plug time to develop strength to resist the upward bending that would be applied when the standing water was pumped from inside the shaft. None of the three tremie plugs on the project leaked.

4 TUNNELLING

4.1 Introduction

Owing to the cohesionless nature of the deposits through which the tunnels would advance, a closed face system with the capability of balancing the existing in-situ stresses and ground water pressures was needed. The successful re-introduction of slurry shield microtunnelling to the Greater Toronto Area (GTA) in recent times meant that a sealed tunnelling system capable of balancing the existing in-situ stresses in front of the TBM along with providing a dry tunnel immediately after the heads passing was available. Hydraulic jacks in the main launching shaft advance jacking pipe into the bore which in turn advances the TBM to engage the ground ahead of the cutting wheel. The entire string, when pushed from the launching shaft, moves as one right up to the cutting wheel. The slurry shield utilizes hydraulic muck conveyance from the face to a solids separation facility usually located at ground level. The hydraulic mucking system has the ability to be pressurized to match the existing in-situ stresses so that equilibrium is maintained during excavation.

4.2 Equipment

The Herrenknecht AVN1200TC, shown in Figure 4.1, complete with a mixed ground cutting wheel was used on the project. The mixed ground cutting wheel has the ability to excavate all soil types from cobbles and boulders to fine silts and clays.

The mixed head contains picks and scoops for fine grained soils which need to be pulled from the parent material and cutting discs for boring boulders and larger, harder obstructions. The openings on the head are medium sized (approximately 250mm opening) so as to allow larger objects fall into the excavation chamber behind the cutting wheel where a rotating arm further crushes these particles down to approximately 50mm in diameter. These particles are then small enough to be transported by the hydraulic slurry circuit to the solids separation plant at ground level. The mixed ground head is a very versatile tool for boring through unpredictable deposits where rescue or intervention shafts would prove extremely difficult if not impossible to install. Although no boulders were anticipated, their existence in the glacial soils around the GTA is random and often localized in nests.

Figure 4.1 - Herrenknecht AVN1200TC with mixed ground cutting wheel attached

The AVN1200TC is the smallest size TBM that an intermediate jacking station (interjack) can be used with. Although the crossing lengths on the project were relatively short (91m & 78m), the ability to install an interjack on any tunnel length in such high risk ground is advantageous. Should the jacking forces after a given distance become too large or show a trend that more jacking force than normal is needed, the interjack can be installed at any point to help.

Access to the rear of the cutting wheel is provided on the AVN1200TC through a central access door. This door can be opened anytime throughout a tunnel drive to inspect the cutting wheel, change the cutter discs, remove obstructions or look at the excavation face. The ability to change the cutting tools from inside the TBM grants the project an additional lease of life that is not possible on smaller sized TBM's. This feature has been the difference between tunnel success and failure in the past and has eliminated the need for a rescue shaft. By upsizing a tunnel at the design stage to incorporate this feature, the chances of tunnel failure or the need for a rescue shaft can be reduced.

The solids separation system consisted of a three part solids removal process. The primary separation system consists of a high G force shaker deck which removes solids down to ~0.45mm in diameter. Slurry water directly from the TBM head is pumped over the first deck layer meaning this deck removes solids ranging in size from 50mm to 0.45mm. The secondary stage of separation involves the use of high pressure hydro-vacuum cyclones which apply a centrifugal force to the slurry water. The cyclones are usually vertical in orientation with the bottom of the cyclone being tapered. The taper increases the velocity inside the cone which causes larger particles to separate from the solution and discharge to the bottom. The cleaner water is caught in an upward spiral in the middle of the cone and is discharged back to the holding tank. The secondary system is capable of removing solids down to approximately 45μm. The third level of treatment is the centrifuge decanter. The centrifuge operates on the principle of accelerated settling. The slurry water entering the centrifuge is dosed with a liquid polymer in order to create large enough flocs to settle once the centrifugal force is exerted on the liquid. Clean water from the centrifuge is fed back to the holding tank and the cycle is repeated. A continuous and efficient operation of the separation system will mean that the same water can be recycled time and time again for use in the conveyance of cuttings from the TBM. A picture of a typical three stage separation system can be seen in Figure 4.2.

Figure 4.2 - Typical 3 stage solids separation system

5 PROBLEMS

5.1 Shafts

During the inspection of the base and the secant pile interlocking in the R1 shaft by divers in December 2013, it was found that there was an approximately 300mm sq. void along one interlock on the north east of the shaft. The diver could insert his arm and reach to full extension without encountering soil. Even with a 1m long probing tool and his arm at full stretch, he was not able to make contact with any soil. Visibility for the divers was zero and he was relying on touch alone for exploration. It was evident that a void existed outside the shaft although it was thought at the time to be localized. The location of the void in the interlock was at a depth of approximately 14m below the ground surface. The shaft had not been pumped down at this stage and any ground loss had most likely occurred while excavating with the telescopic clamshell. An existing 600mm diameter high pressure watermain was running 1m to the north west of the R1 shaft. The watermain was 3m deep and ran parallel to Mayfield Road.

It was decided that the best course of action was to pour the tremie plug in the shaft and then pour an additional secondary concrete skin inside the secant piles, and on top of the tremie plug to act as a bandage on the open void. All remedial works had to take place underwater. A picture of the circular steel formwork used to form the underwater patch can be seen as it is being lowered into the R1 shaft in Figure 5.1. The annular space on the outside of the form and the inside of the secant piles was then filled with concrete using a tremie pipe mounted to a 42m concrete pump truck. A diver was at the excavation base for the entire concrete pour and was responsible for guiding the tremie pipe around the formwork and ensuring an even rate of rise. He also placed concrete into the void area. The underwater patch was successfully poured and the opening sealed with concrete.

Figure 5.1 Lowering a steel circular form to sit on the tremie plug – concrete poured on outside of steel form

After returning from the Christmas break in January 2014, a large void was noted at ground level to the north east of the R1 shaft. The volume of this void was approximately $7m^3$ and tapered towards the shaft in a half-conical shape. This void at ground level was filled with U-Fill concrete upon discovery. Up to this point, the extent of ground loss was not known. As a precaution, the contractor proposed lining all shafts internally with an additional skin of concrete from the top of the tremie plug to above the water table which was considered the danger zone. Both reception shafts would be relined before any dewatering took place. Given the circular form shown in Figure 5.1 is just 2.5m high, another method of forming the secondary skin had to be used. It was decided to use

3.3m diameter corrugated culvert piping placed vertically to support the concrete. A photograph of this pipe can be seen in Figure 5.2. In order to prevent the culvert pipe becoming unbalanced, two concrete pumps filled the annular area on opposite sides simultaneously. The corrugated steel pipe was also used to line the full height of the R2 shaft even though no problems were uncovered during diving inspections.

Figure 5.2 - 3.3m diameter corrugated steel culvert pipe used to reline shafts

5.1.1 Shaft Ground Loss Remediation

At the R1 shaft it was still uncertain as to whether or not a void or voids remained at a deeper elevation in this high risk location. In order to ascertain if the subsurface geology had been altered in the critical area around the watermain, Cone Penetration Testing (CPT) was performed and Utility Monitoring Points (UMPs) were installed on important nearby infrastructure.

Three CPT were performed on the north east corner of the R1 shaft. Their results were then assessed against the tender site investigation data to see if a noticeable difference in relative density at a given depth was evident since the ground loss event. No noticeable difference was observed. The results from all three CPT were broadly similar leading to the conclusion that the ground through which all three probes had passed was similar and uniform throughout – i.e. the cohesionless soils had 'self-healed'. No obvious voids were detected using the CPT method.

Although no voids were discovered during the CPT phase, grouting was required at the R1 location in the eventuality that undetected voids still existed. Pumping out of the shaft water could not take place until it was known that any voids around the shaft had been filled. A Klemm KR 702-2 drill rig was used to drill two 200mm diameter cased holes to a depth of 19m - just below the secant pile depth. This drill rig is shown in Figure 5.3. Throughout grouting, the previously installed UMPs were being constantly monitored to watch for any signs of movement as a result of the grouting operation. As each casing was retracted, liquid grout was poured into the hole until the top of grout was at ground level – then another casing would be removed. This method continued until all augers had been removed and the top of grout in the hole remained static at ground level. Two holes were drilled and filled in this fashion until both holes had a stable grout elevation. A total of approximately 5.7m^3 of grout was gravity-fed into

Figure 5.3 - Klemm 702-2 vertical drill used for remedial grouting

the ground. The volume of the drill holes totaled just 1.2m^3. Therefore, an additional 4.5m^3 of grout had been introduced to voids in the ground. No movement had been observed on the UMPs throughout the grouting operation.

R1 shaft was finally dewatered slowly on March 5th while the UMPs were being monitored for movement and the shaft observed for signs of stress. No movement was observed on the UMPs and the shaft integrity was sound throughout and after dewatering.

5.2 Tunnelling

The TBM and all trailing pipework provide a sealed system that prevents the inflow of groundwater or fine soil particles into the tunnel. The TBM and trailing pipework could tunnel into a pure water body without the inside of the tunnel getting wet. The risk therefore lies with the entry of the TBM to the ground from the launch shaft and the receiving of the TBM into the reception shaft.

At entry and exit points particularly for deep alignments, the possibility exists for the large confining pressures which are normally kept in equilibrium (at rest) to become exposed to atmospheric pressure which is much less than what is needed to maintain equilibrium. If the seal is breached, the area of high pressure (the ground) will tend to move to the area of low pressure (the shaft). The result is ground loss. A great deal of effort must therefore be spent on keeping both of these zones apart. For this reason, custom launch and reception seals were fabricated to facilitate the safe entry to, and exit from the ground. Double rubber seals were used and are shown in Figure 5.4.

When the TBM is entering the ground from the launch shaft, the natural tendency is for the rubber seal to bend in the same direction as the TBM is moving – in this case towards the virgin ground. This inherently makes the seal at the launch shaft a strong one as the rubber has to double back on itself before allowing the ground pressure to exit into the launch shaft. Additionally, movable stiffener plates which are shown in the top photograph of Figure 5.4 are closed around the TBM body once the wider cutter wheel has entered through the seal. These plates further prevent the seal blowing back due to ground pressure at the launching side.

Figure 5.4 Tunnelling launch (top) and reception (bottom) seals

As the TBM enters the receiving shaft, the natural tendency is again for the rubber to bend onto the pipe in the direction the TBM is advancing. In other words, the rubber seal moves out into the shaft around the pipe. This is the weakest possible position for a portal seal as it does not require much pressure on the retained side to push its way past the rubber and into the shaft.

Since the piezometric head acting at the tunnel horizon was only about 6m, which is normally tolerable for standard portal seals, it is clear that vertical communication of the upper ‘perched’ water table had occurred, conveying the full water pressure (almost consistent with the ground surface or some 16m of head) into the tunnel zone. This likely occurred along the outer secant pile shaft-to-soil interface, effectively breaching the upper clayey silt till aquitard. This is surprising, since one would normally expect that the rough bore wall formed by the secant pile would be sealed tightly against tremie-placed concrete.

In hindsight, some form of ground improvement outside the shaft at the portal seals might have been of benefit to mitigate flowing ground at the portals. This could have taken the form of jet grouting, ground freezing or additional secant piles; however, such measures have not been required in the past for intermediate-depth tunnels such as these.

Problems were encountered with both receptions whereby ground pressure was able to force its way past both rubber seals. After the failure of the first reception seal, a significant amount was spent modifying the reception seal for the second tunnel. These modifications included the reduction of the internal diameter of the rubber sheets so the sheet would stretch out further on the TBM upon entry. It provided more of a gripping area should additional banding be required around the circumference to stem the inward flow to the shaft. Another precaution taken was that when the TBM hit the receiving shaft pile, it cored into the pile a distance of about 200mm so that it would not sink and then all boring was suspended for one shift. Extremely thick bentonite lubrication with a marsh funnel time of approximately 130 seconds (although difficult to measure at this viscosity) was batched at the launch shaft and injected into the annulus around the TBM head and trailing tube while they were stationary. It was believed that if we could permeate viscous bentonite a distance into the surrounding ground immediately outside the shaft – creating a circumferential filter cake – that it would help support the earth during the critical receiving process. Although the reception of the TBM for the second tunnel was initially less troublesome, significant ground loss eventually occurred once the first concrete pipe emerged through the rubber seal. The rough concrete pipe surface had dragged some dried sand with it and when it emerged through the rubber seal, it allowed water to jet out thereby washing out the sand while feeding it with more from behind.

A total of $6m^3$ of sand had entered the shaft and tunnel during the R1 reception of the TBM and $13m^3$ entered while receiving the TBM into the R2 shaft – both volumes give the bulked quantity.

The ground loss at the R2 shaft was of grave concern given that it was within 5m of a live road. Widespread settlement was visibly noticeable at ground level shortly after the TBM had entered the shaft. The existing road pavement around the shaft had begun to crack perpendicularly to the areas of settlement. An 80t mobile crane which was servicing the shaft had to be quickly de-rigged because its outriggers were being undermined and the operators levelling bubble (for ensuring a level setup) had shifted off plumb. The ground loss had the capability to cause at least 3 additional serious accidents if due care was not taken. In the end, the road traffic was not affected by the settlement at R2 and the crane was safely moved further away from the shaft.

A point worth noting which amplified the ground loss issue beyond what it may have been is that because the reception shafts were now smaller due to the installation of the “bandage” at the bottom of R1 and the steel culvert liner in R2, the TBM could not be removed as one full piece which is the norm. The TBM length is 3.2m. It has an articulated joint in the middle for steering corrections. It is impossible to remove a 3.2m long TBM (1.5m in diameter), from a 3.2m diameter shaft, therefore the radical decision was taken to split the TBM at its steering joint once it emerged through the reception seal so it could be removed in two 1.6m long pieces. The splitting of a microtunnelling machine at ground level is a complex and arduous task involving heavy equipment, highly trained personnel and many man hours – let alone performing it at the bottom of a 15m deep shaft in less than factory conditions without the proper equipment - twice. The TBM was split in three hours on both occasions, however, during these three hours continuous ground loss was occurring around them as they worked. It is only when the first concrete pipe emerges through the reception seal that the inflow of sand, silt and water can be stemmed permanently. A standard TBM reception should take in the order of 2 – 3 hours. Due to the smaller shaft sizes and splitting the TBM, the receptions on the project took about 8 hours. A photo of the front of the TBM after splitting and being removed from R1 is shown in Figure 5.5.

Figure 5.5 Split TBM being removed from R1 (cutting wheel facing away from photograph)

5.2.1 Tunnel Ground Loss Remediation

After the TBM was received into the R1 shaft and the first concrete pipe sealed onto the rubber portal, grouting took place from inside the tunnel via grouting ports left in every 6^{th} pipe. The grouting operation was concentrated towards the reception shaft end so as to deliver grout to where it was needed most. Due to the ground loss which had occurred to the south west of the R1 shaft where the TBM had entered, a concrete slab at ground level cracked and began to list away from the shaft – this crack exposed a void below, which was assumed to lead to the crown of the tunnel. Along with the $10m^3$ of grout pumped through the grout ports in the tunnel, $6m^3$ of grout was pumped into the ground through the cracked slab before refusal was achieved.

In a similar fashion to R1, grout was pumped into the ground from inside the microtunnel pipe once the TBM was removed from R2. The majority of the grout was installed towards the reception shaft end. A total of three days were spent grouting the pipeline and some $20m^3$ of grout was pumped before refusal was achieved. Grout was not poured from ground level at R2 as a local swallow hole was not present. Instead, mass uniform settlement had occurred.

6 CONCLUSIONS

Despite two significant problems which occurred during the project, related to the shaft construction and the portal seals, the tunneling aspects of the project were deemed highly successful. The pro-active specifying of a slurry-shield MTBM coupled with sealed shaft construction by the Region of Peel proved to be well-founded. This was a particularly enlightened decision since it remains uncommon in Ontario for Owner’s to accept any risk associated with means and methods of construction, especially where temporary works are concerned. It is highly unlikely that any other tunneling methodology could have completed these bores given that dewatering/aquifer depressurization was not permissible on this project in this environmentally sensitive locale.

The ability for silty fine sands and sandy silts to flow and exert high lateral earth pressures must never be underestimated, even where such materials are densely packed in the undisturbed state. The relief of overburden pressure due to shaft sinking and related dilation induced by secant pile boring are enough to liquefy these soils and render them into a fluid state.

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