EPB TBM UNDER THE CITY CENTRE OF VANCOUVER: RISK MANAGEMENT AND SETTLEMENT CONTROL

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Keywords: urban tunnelling, risk management, settlement control

INTRODUCTION

The Canada Line is a 19km long automated light rapid transit system with 16 stations in Vancouver, BC, Canada. The Concession was awarded to InTransitBC to design, build, partially finance, operate, and maintain the system for 35 years. The project must be completed by November 2009 to be ready for the 2010 Winter Olympics to be held in Vancouver and Whistler, BC. The bored tunnel section consists of 2.45km of twin bored tunnel and 3 stations. The station work is limited to the excavations and temporary support necessary to facilitate the bored tunnel works. A joint venture between SNC-Lavalin Constructors Pacific and SELI (SSJV) has recently completed the design and construction of this section. Work for the bored tunnel commenced in November 2005 with shoring and excavation works for the tunnelling operations pit that will become Olympic Village Station and will be completed in the spring of 2008.

The tunnels were driven with a 6m diameter Earth Pressure Balance Tunnel Boring Machine (EPBM) and lined with 250mm thick, 5.3m diameter precast concrete segments. Seven hundred and fifty meters of each tunnel were driven through glacial and interglacial deposits of sand, silt, and clay with granitic boulders. The paper discusses the challenges faced in negotiating these soils which contained unforeseen geological conditions. General facts and information are also presented.

ALIGNMENT AND GEOLOGY

The tunnels negotiate several combined vertical and horizontal curves with radii as tight as 212m through the most difficult section of the alignment. An aerial view of the alignment can be seen in Figure 1. Both inbound and outbound tunnels were driven northwards from the tunneling operations pit. The tunnels' alignment dips steeply after exiting the start pit, passing from rock to soil under False Creek, and continues in soil for 750m before reentering the rock for the remainder of its length. Figure 2 shows the geological profile.

The rock is sedimentary and referred to locally as the Kitsilano member. It is comprised of interbeded sandstone (approximately 75%), siltstone, and claystone with a regional dip of about ten degrees towards the south. The sandstone is generally weak with UCS of 5 to 10MPa and is interspersed with cemented sandstone "floaters" with UCS of ~50MPa. The rock is generally massive, with very high R.Q.D. Jointing is poorly developed or absent, except along the bedding. Volcanic intrusions of basalt and seams of coal are also found in the Kitsilano member. Tests have revealed quartz content up to almost 50%.

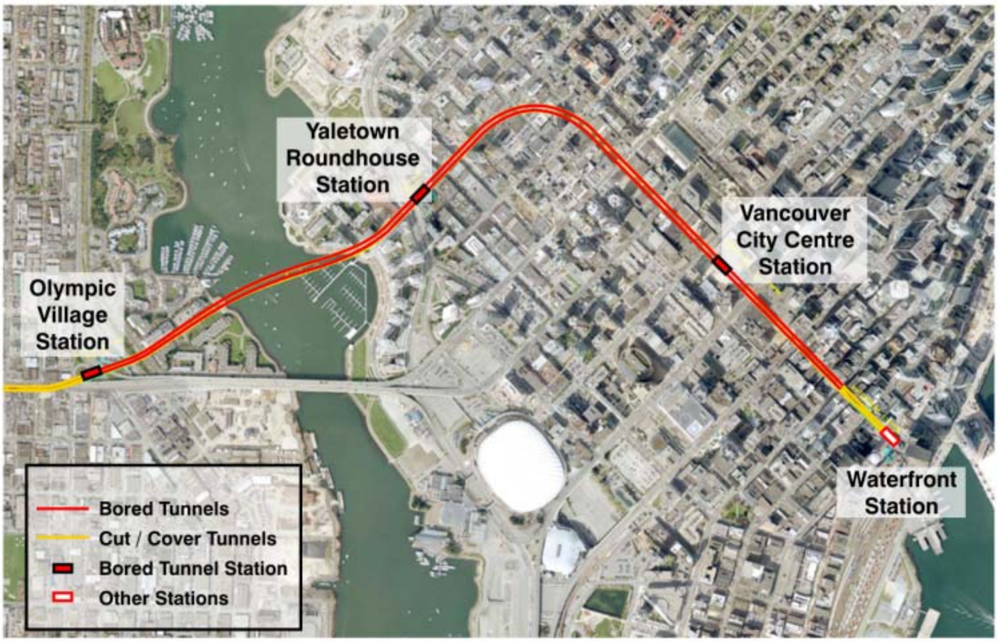

Figure 1 – Bored Tunnel Alignment and Stations

The soils are described in the geological report as glacial till with minor non till like soils. It consists of a clay/silt/sand matrix with gravel. Fines content is typically 45-50 percent, although sandy pockets are knows to be present. The till is predominantly plastic with only 13% of samples showing non-plastic properties. The till also contains granitic boulders up to 2 or 3m across. In geological terms these soils are referred to as Vashon Drift and form the erosion resistant cap of the area that was consolidated by hundreds of meters of ice during the last glaciation. The till is usually very hard and impermeable and was expected to be an excellent tunneling medium. The presence of boulders was one of a few foreseeable risks.

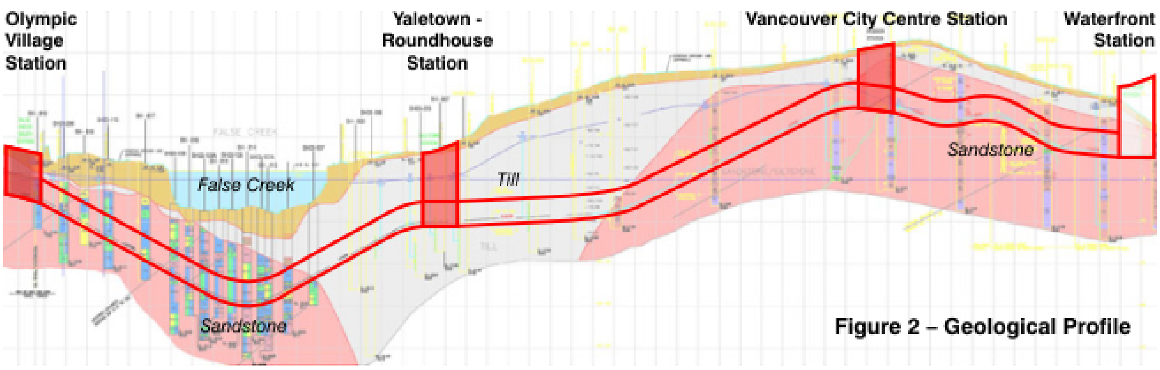

Figure 2 – Geological Profile

Figure 2 – Geological Profile

Alignment within the till (before risk assessment)
The tunnel horizon enters the till under the "river like" False Creek, 25m below high water level. The interface with the sandstone is approximately 60m long at a tunnel grade of 5.5%. The tunnel comes ashore with only 7m of till cover and close to multi-storey residential towers with deep basements. From there the tunnel progresses straight along Davie Street at depths ranging from 12m to 25m with high rise towers and deep basements on each side. For the final 150m of till section, the tunnel negotiates a combined vertical and horizontal curve (212m radius) before entering the sandstone along a shallow transition; the transition zone occurring 7m below the foundations of a residential tower over 30 storeys high.

TBM FEATURES AND PERFORMANCE

The Tunnel Boring Machine (TBM) needed to be capable of boring through rock and soil in a dense urban environment. The control of ground movement is extremely important where high rise buildings and utilities exist. The grading curve for the till is shown in Figure 3 and it can be seen that these soils fall mostly within the zone of normal Earth Pressure Balance Machine (EPBM) operation. Although the rock is weak it is generally very stable so face support wasn't seen as being problematic. An EPBM was chosen with foam soil conditioning capability to provide adequate face support and achieve minimal loss of ground.

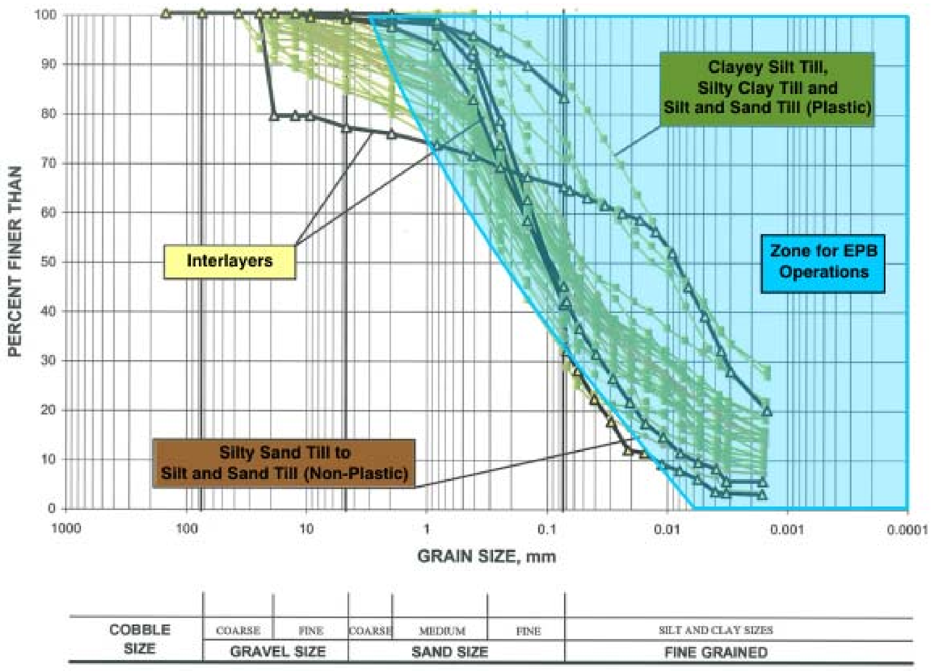

Figure 3 – Grading Curves for Till Material

Depending on the ground properties, the overburden, the altitude of ground water table and the requirements for reducing ground settlements, the face support pressure was calculated for the whole alignment by Professor Kovari and M. Vogelhuber.

The TBM procurement was competitively tendered and awarded to Lovat. The 6m diameter shield has a mixed face cutting head dressed with twin tipped 17" disc cutters that are fully exchangeable with rippers. There are 7 injection ports on the cutting head and others in the plenum and screw conveyor. The TBM is driven by six variable frequency electric motors capable of producing 6,180 kNm of torque. Muck removal is through a standard EPBM screw conveyor onto a conveyor belt and into muck cars. TBM advance rates for the project are shown in Table 1 below. General working hours were 24 per day, 6 days per week.

Table 1 – TBM Advance rates

	Best 24hr production		Best weekly production	
	Drive 1	Drive 2	Drive 1	Drive 2
Sandstone	18 Rings 25.2m	20 Rings 28m	97 Rings 135.8m	100 Rings 140m
Till	19 Rings 26.6m	21 Rings 29.4m	77 Rings 107.8m	105 Rings 147m

To prevent ground movement into the void left around the tunnel lining as the shield advances, the TBM utilized through the tailskin grouting. Accelerated grout is injected as the machine advances and gels within 10 to 15 seconds to fill the annulus and prevent movement of ground. As a safety feature, the machine can not be advanced without grouting.

The TBM was equipped only with a standard length screw conveyor (approximately 9m long) that creates a steep pressure gradient between the inlet and discharge points and not an extended screw conveyor that allows the pressure gradient to be much shallower. The seals on the bearing allow the TBM to operate at 4 bar pressure.

RISK ASSESSMENTS

The contract required that SLCP-SELI (JV) produce a risk assessment for the entire alignment. This was carried out section by section for the entire alignment by Professor Kalman Kovari and R. Filippini of Switzerland in conjunction with project staff. Beyond this, the JV met on a weekly basis with its client (SNC Lavalin Inc), InTransitBC, and Canada Line Rapid Transit Inc. (CLCO). CLCO is a wholly owned subsidiary of Translink, the ultimate owner. At the weekly meetings, risks, mitigation measures, progress, and problems were discussed with the sole aim of finding the correct technical solutions for the project. CLCO retained the services of Professor Zdenek (Dan) Eisenstein as its expert attending the meetings.

The Risk Assessments were made by reviewing all available soil, alignment and structural data for the buildings provided by the JV. A memorandum summarized the results of the risk management that aimed to reduce the residual risk to an acceptably low level. By definition, the residual risk is the remaining risk after mitigation measures including unknown risks. From the risk assessment, the JV would develop mitigation measures that were discussed and agreed upon in the risk management meeting before developing the Risk and Safety Plan. The Risk and Safety Plan highlights all the risks and mitigation measures for each section of alignment and sets EPB pressures among other things. This information is used by the engineers, TBM manager and TBM operators to execute the tunneling in the safest way possible.

DRIVE 1 RISK ASSESSMENT AND MITIGATION MEASURES

Five distinct areas were identified as having higher than acceptable risk, all due to the location of the tunnel relative to the raft foundations of buildings and the possible local occurrence of unfavorable ground conditions. The compounding factors for the first two buildings were that they were adjacent to the shore of the "river like" False Creek and, as the transition from sandstone to till was under False Creek, the influence of tunnel boring within the till had not been measured and the elastic modulus had only been assessed by back-calculation and not testing. The tunnel was located slightly under the buildings with as little as 7m of till between the crown and foundations. Although the vertical alignment could not be changed beyond the maximum 5.5% gradient required to reach the next station, the horizontal alignment could be changed to move the tunnel outside of the building. Figure 4 shows the shift in alignment for these two buildings. Two multipoint extensometers were also installed onshore immediately before and after the first building to assess ground behavior while boring under the designed boring parameters.

Figure 4 – Shift in alignment implemented after Risk Assessment

The highest risk section of the drive through the till was identified as the under passing of Brava Towers, twin residential towers over 30 storeys high with five levels of underground parking. The main risk was due to the possible non predictable occurrences of lenses or layers of silt and sand with no cohesion under high water pressure. Also the presence of major boulders of extreme high uniaxial strength (90% quartz) would have caused major problems under the building. The first drive (outbound tunnel) was to pass within 8m of the core footing and individual spread footings. In addition, the alignment here negotiates a 212m radius horizontal curve, a vertical curve (from -5.0% to +5.5%) and approaches the transition from till back into sandstone where geological anomalies may exist. The risk assessment recommended that the horizontal alignment be changed to avoid the footprint of the Towers entirely. However, this proposal was rejected as a revised alignment would require additional property uptake at significant cost and with possible delays. As an alternate mitigation measure, the alignment was lowered to increase the tunnel separation from the foundations by 7m. At this juncture, the performance of the TBM within the first 200m of till section was known to be good with minimal ground loss indicated by the low extensometer movements and surface/building settlements. Based on this performance, and the increased clearance to the tunnel, the revised alignment was viewed by the risk management group as having acceptable calculated risk, with good face support and soil conditioning being key.

DRIVE 1 PERFORMANCE AND GROUND CONTROL

The transition from sandstone to till went smoothly without any geological anomalies becoming apparent. Once the machine was wholly in the till face inspections, under atmospheric pressure, showed the face to be comprised of strong impermeable till. These conditions continued as the TBM approached the shoreline of False Creek and the loss of ground indicated from the first inclinometer was less than 0.5% and building movement limited to a maximum of 4mm. Shortly after, the screw conveyor became blocked on several occasions by boulders. At this stage the TBM was adjacent to the first building foundation and operating under a face pressure of approximately 1bar, so the screw blockage was cleared and excavation continued until the machine was clear of the building. It was known from observation of the extracted muck that the ground conditions were good so a cutting head inspection was done under atmospheric conditions. The cutting head was designed to allow boulders of maximum size of 300mm into the cutting chamber and into the screw conveyor. This was done by placing grizzly bars across the opening of the cutting head. It was apparent from cleaning out the blocked screw conveyor that larger boulders were entering the chamber. The face inspection showed that the grizzly bars had been knocked out (as they were only bolted in), allowing larger boulders to enter. To prevent this situation, all bolted grizzly bars were replaced by thick welded plate during an intervention that lasted several days. After this work, no large boulders entered the chamber but frequent face inspections were required to check if the openings were blocked by boulders and if so, they were removed. Figure 5 shows the strong impermeable till and boulders stuck in the 300mm openings where the grizzly bars had not been knocked out. No further screw blockages were encountered after replacing the grizzly bars.

Figure 5 – Strong impermeable till and boulders stuck between intact grizzly bars

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

After Yaletown, the alignment drops at 5% to implement the mitigation measure of increasing the cover under Brava Tower, while ground level elevation increases at about 4%; ground water level following ground level. Normal boring conditions continued for approximately 120m with good advance rates, small surface and building settlements (2 or 3mm) and face support pressure increased from 1 to 1.2 bar. However, the muck was noted to contain higher proportions of wet sand and silt. The next intervention revealed a layer of wet sand within a stronger till matrix that could not be supported in normal atmospheric conditions due to the mobilization of groundwater. The chamber was closed without being able to make the necessary cutting tool changes. At this stage, it was assumed that the wet sand belonged to a finite pocket of material as described in the geological reports and as experienced by deep excavation contractors. From this point on the total thrust of the TBM and the cutting head torque began to steadily increase with a steady decrease in penetration rate. However, extracted muck weights and settlement stayed at, or slightly above, normal so the TBM was advanced to find suitable ground for a face intervention but the muck exhibited higher percentages of cohesionless sand showing the waterlogged permeably material was not confined to a pocket, but was a layer with recharging water. In this area, there was more than 20m head of water above the crown. At this stage the face support pressure was increased to prevent the possibility of over excavation. Pressing forward, the penetration rate continued to decrease with increased thrust and it became clear that the cutting tools were at the end of their useful life. To make matters worse, the face became 100% sand with cohesionless silt with increasing quantity of silt as the tunnel came closer to its lowest point with 25m head of water. Face interventions were still not possible and sensor observations made during lowering the pressure in the excavation chamber showed that even with compressed air inside the chamber the pore water pressure would increase. With the tunnel crown over 30m deep the only option was to continue boring in the hope that the ground conditions would return to the predicted "till" or become more favorable. After approximately twenty advances (855 to 875) in extremely poor ground conditions of silt and sand the ground conditions became better with the volume of water decreasing significantly. Over this stretch of poor ground conditions, the surface settlements reached up to 10mm.

Throughout the soil section, the ground was conditioned with foam (Foamex TR from Lambertti) to work the muck into a paste to properly transfer face support pressure and to form an impermeable plug in the screw conveyor to avoid loss of ground from the face. The increasing coarseness of the material combined with the high water pressure meant the foaming agent was not effective, or was less effective due to the high water pressure and flow of water breaking down the foam bubbles. In these instances, the muck was permeable and would allow water to flow through the screw conveyor, bringing particles with it (flowing ground). Even in non-plastic silt under the same water pressure the operator had managed to condition the muck to prevent flow of water through the material. However, this conditioning was less than ideal and the material would flow through the screw conveyor if not mechanically managed by partial closure of the guillotine door.

Around advance 875 the ground conditions improved somewhat allowing a reduction in face support pressure with adequate ground conditioning. This allowed penetration rates to remain acceptable while progressing under some low rise buildings. However, with the cutting tools in such poor condition, there was significant concern that the machine could become immobilized as it entered the sandstone under Brava Tower. Also, it was expected that the ground conditions at the soil/sandstone interface could include anomalies such as nested boulders or inclusions of poor ground. These mixed face conditions would make the already problematic ground conditioning and face support pressure transfer even more difficult.

Due to the foreseen problems, the TBM was stopped under Seymour Street (adjacent to the Towers) to make extraordinary attempts at changing the cutting tools. At advance 915 the chamber was entered to inspect the tools. The face of silty sand remained stable. However, water and soil was inflowing over the shield, its action propagating slow but progressive collapse of the soil overhead. The chamber was closed and the plenum refilled and the TBM advanced. To try and prevent the flow of water and soil from around the machine, attempts were made to fill the over cut annulus around the TBM with expanding polyurethane foam by drilling through the tailskin of the machine. In addition, it was expected that the metal fins that prevent grout from traveling forward to the face of the machine (along the outside of the shield) were damaged and therefore there may be a flow of water from the annulus around the ring (assuming that the grouting was not completely successful). Therefore, polyurethane foam was also injected through the ring. The next attempt to open the chamber revealed that these attempts had been unsuccessful.

The decision on how to continue the drive under the highest risk section with a machine of reduced capacity due to excessive wear and probable damage was based on a number of factors. The settlement readings in all buildings along this section had remained very low (less than 4mm) and surface settlements had remained less than 3mm in all but one section (where the road surface displayed significant previous disturbance). Analysis of the Brava Tower structure showed that with a settlement of 9mm of an individual footing (more than twice that previously experienced), the stresses in the structure would increase only by 10% locally. Experience had shown that adequate ground control (low loss of ground) could be achieved even with worn cutting tools and very poor ground conditions unsuited to normal EPBM operations. In addition, as the building was designed to be fully drained, the head of water above the TBM was somewhat reduced to approximately 15m instead of 25m.

Considering all facts, a calculated risk was taken to continue the drive. The TBM under passed the building (34 advances) in less than 5 days with the penetration rates reducing from 50 to 30mm/min. The maximum settlement experienced at any footing was 6mm with 3 or 4mm being normal for other footings directly over the alignment.

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

REASSESSED GEOLOGY

The problematic ground conditions experienced on the first drive led to a reassessment of the geology after Yaletown-Roundhouse Station. Further interpretation of borehole logs changed the assumptions about the reasons for "loss of recovery" of samples from the boreholes. The high SPT values recorded at/around these poor recovery zones had previously been interpreted as indicating good strength sandy till and poor recovery of sandy samples was interpreted as pockets of sand as per local experience. The experience from the tunneling operations and problems encountered during installation of extensometers and micropiles (shoring for Yaletown-Roundhouse Station excavation) led the team to reassess this interpretation.

It was known from the first drive that once unconfined the sandy layer is cohesionless; being under water table, the sand flows. Pockets of sand enclosed in the till flow when unconfined, but these pockets are reported to be limited in extent and not hydraulically connected (no water recharge). To confirm suspicions that the cohesionless sand and silt were present in a continuous stratum, boreholes were made up to 40m depth with Sonic Drilling techniques that allow full core recovery. These holes showed the presence of a thick layer of sand and silty sand from approximately 15m depth under a cap of strong till. The strong till soils belong to the Vashon unit. The well-sorted sand and silt layers were reassessed as belonging to the Pre-Vashon geological deposits, probably Quadra unit of deposited fluvial sediments.

Reinterpretation of the borehole logs, as well as compiling all the data learned during the first drive and the installation of drilled micropiles, extensometers and boreholes, allowed the geometry of the Quadra unit to be established. Sieve analysis of samples taken show less than 10% fines content which puts the grading curve outside the zone of recommended EPBM operations.

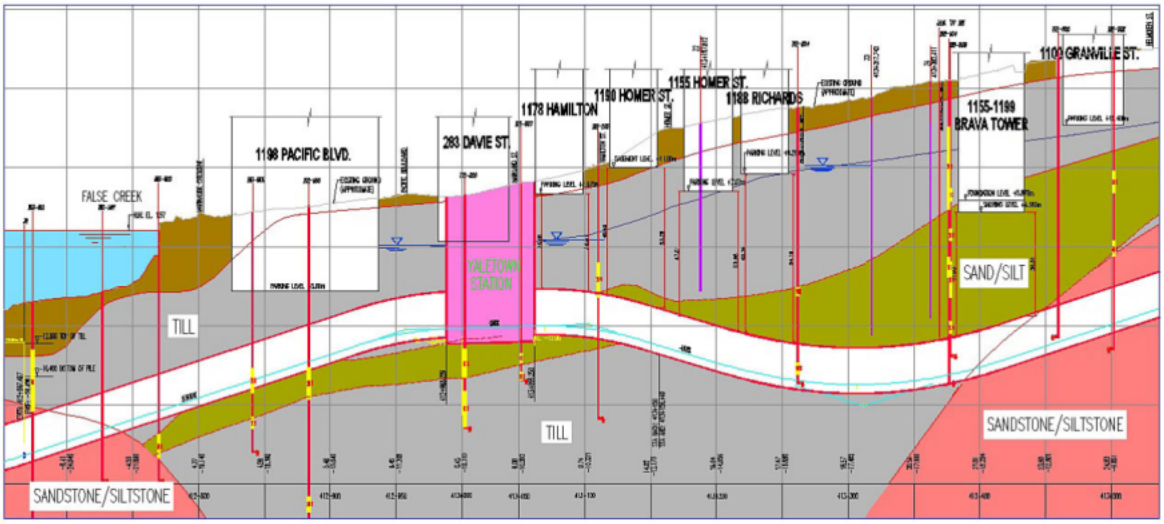

Figure 6 – Reassessed Geological Profile

Figure 6 shows the interpreted layout. The TBM encounters this unit for approximately 350m and it ranges in thickness from approximately 2m at the station to more than 15m.

Further documental investigations revealed unpublished bedrock contouring interpretations suggesting that the alignment along Davie Street progresses towards the head of a buried valley with steep sided bedrock at the head of the narrow valley under Brava Tower.

DRIVE 2 RISK ASSESSMENT AND MITIGATION MEASURES

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

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

The risk assessment for the second drive did not identify any explicit hazard in the first 200m of the till section, but focused primarily on the stretch beyond Yaletown-Roundhouse Station, and specifically on the critical passage close to and underneath two high-rise buildings (1155 Homer and the Brava Towers). Due to the unfavorable geological conditions and the dense urban environment in this section several mitigation measures were suggested in the risk memorandum. Of those the following were implemented that were discussed and agreed at the risk management meetings and were deemed to reduce the risks to acceptable levels:

- Tunnel separation increased by 1 diameter (from 12m to 18m) underneath the Brava Towers to minimize their interaction, avoid the tower's core foundation and move the Drive 2 tunnel toward more undisturbed ground conditions (out of the natural plastic radius of Drive 1). This realignment is shown in Figure 7.

Computational analyses were made to assess the effect of Drive 1 on the soils of Drive 2;

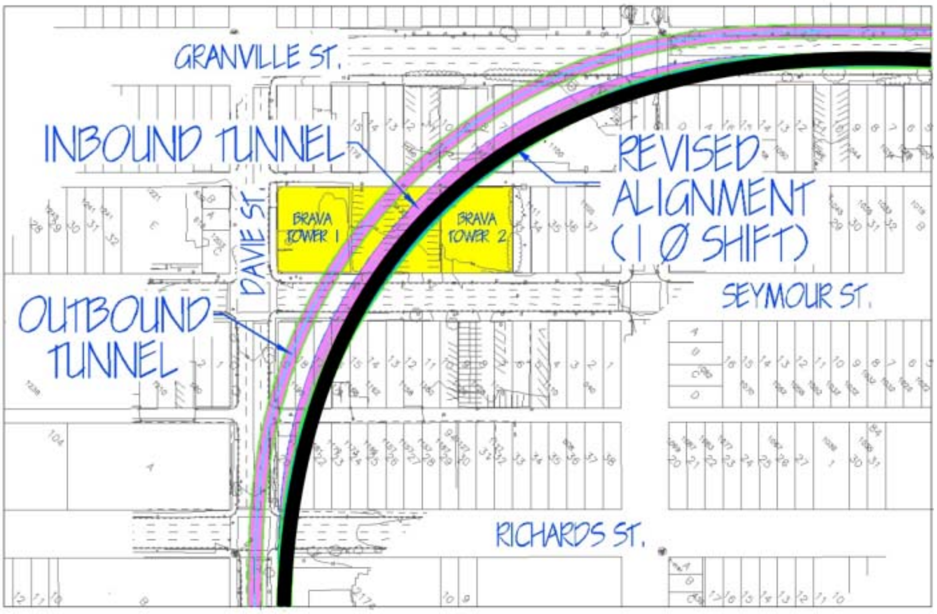

Figure 7 – Horizontal alignment shift

- Use of long-life and high wear resistance cutting tools with a mixed-face configuration (ripper teeth and cutters) to minimize the effects of the erratic presence of boulders and the uncertainty as to when ground conditions would allow safe chamber entry and cutterhead maintenance;
- Addition of another row of steel wire tailskin brushes (third) to allow the grout to be injected at a constant pressure of 2 to 3 bars, and therefore properly fill and seal the annulus around the ring;
- Automatic grout line cleaning: at the end of each stroke, a shot of hydraulic oil is automatically injected through the grouting lines to prevent blockages. This improvement helped to avoid stops during (and even after) the erection of the ring that, during Drive 1, caused critical discontinuities in the excavation process (e.g. excessive water inflows in the excavation chamber). Blocked lines also prevent proper filling of the annulus allowing the presence of high pressure water at the tailskin brushes;
- Polymer injection: the addition of an automatic integrated Polymer System pump to the original conditioning foaming system. Specific laboratory tests were conducted to verify conditioning of the wet sands and it was determined that the addition of polymers into the foam can remarkably lower the soil's permeability and limit the possibility of high pressure water and water flow destroying the foam bubbles;
- Installation of a piezometer in the excavation chamber to constantly monitor water pressure;
- Additional monitoring instrumentation and 24-hour monitoring (surface, buildings, and extensometers) during the most critical sections;

- Locating maintenance areas based on the Drive 1 reassessed geological mapping and records, where safe and "open face" cutting head maintenance might be possible;
- Hyperbaric crew: specialized crews for hyperbaric intervention available on site (until the tunnel is fully into the sandstone formation);
- Operations and boring parameter control: phases and values checked daily based on shift reports and PLC automatic data logger. Daily toolbox meeting for coordination and planning;
- More frequent calibration of weight scales as the belt scales are adversely affected by very wet soils and the vertical and horizontal curves.

EPB PRESSURES AND SOIL CONDITIONING

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

Having encountered no real problems in the stretch before Yaletown-Roundhouse Station during the first drive, the design face support pressures were maintained the same. Beyond the station the principal criterion used for the EPB pressure calculation was to counter balance the water table pressure and add 20kPa as a safety buffer. The sand section was split into three subsections where the pressure was raised from 150kPa to 200kPa, reaching a maximum of 220kPa going toward the Brava Towers and the sandstone transition. The cohesion and the permeability of the till matrix, if thoroughly analyzed, would have resulted in lower effective and total pressures but would require more complicated and time consuming investigation and study. In addition, an experimental section was successfully driven with high face support pressures (up to 2 bar) before approaching the station to check the TBM performance and possible problems under high operating pressures. Two similar test sections were made after the station at which several tests were conducted to determine the appropriate soil conditioning parameters including Foam Expansion Ratio, Foam Injection Rate and injection pressures. Variations of water flow, foaming agent, and polymer concentration were investigated through slump tests, permeability tests, and visual inspections of the muck. As a result of these tests the set of selected parameters produced a pulpy, dense, and water-tight paste suitable to stop the water flow, and with the right workability. The overall aim was to operate at the design face support pressure and properly conditions the soils using foam agents and if necessary add the polymer to stabilize the foam and reduce overall permeability. The addition of polymer would become most important if the operating pressure had to be dropped below the design value to allow the TBM to advance with worn cutting tools (as in drive 1). In such circumstances the hydraulic gradient would allow water to flow through the foam conditioned material causing the foam to breakdown. Adding polymer at this stage would stabilize the foam and reduce the permeability of the soil.

DRIVE 2 PERFORMANCE

As said in the previous chapter the design of the excavation between the sandstone/till transition and Yaletown-Roundhouse Station did not fundamentally change for Drive 2. Furthermore the disruption caused by the boulders on Drive 1 was completely avoided thanks to the welded-in-place grizzly bars. The ground control confirmed by the surface, building, and soil monitoring instruments were very similar to Drive 1. With respect to the maintenance of the TBM, passing through the Yaletown-

Roundhouse Station box provided the opportunity to fully refurbish the cuttinghead and switch to a mixed configuration of cutting tools as prescribed in the mitigation measures (the first drive passed through the station box prior to excavation). The mixed configuration was half rippers and half disc cutters. The intent of the rippers was to try and achieve better soil-conditioning agent mixing in the sandy material before it enters the cutting chamber forming a more homogeneous conditioned paste.

Beyond the station, three maintenance areas had been planned, the location of which was carefully analyzed from the experiences of the Drive 1. At the first maintenance area 70m from the station, the sand layer was expected to be approximately 2m deep and half way up the face with hard till (with boulders) above. The chamber was emptied for the first intervention and this assumption was confirmed by visual inspection. However, the sand layer had partially flowed into the chamber as the chamber was emptied leaving a small letter box type void in front of the cutting head. For this reason, the chamber was closed and excavation continued. At this stage the layer was not extensive enough and with only 8m head of ground water, the water pressures not high enough, to be noticeable under normal excavation procedures (low face support pressure with foam conditioning only). The brief stop had allowed some limited inspection of the cutting tools and it was already obvious that the rippers, even special rippers with carbide inserts, had been completely worn down or broken off. The plan for the soil section was to ensure, wherever possible, that the cutting head would be refurbished at each maintenance stop, and as most of the rippers were already worn out only 70m into the section of adverse ground conditions, it was imperative that the cutting head was immediately refurbished. A second stop was planned after a few more advances with the construction team making the following decisions as measure to prevent the sand layer from entering the chamber or managing the situation if it did; thus allowing a full maintenance stop:

- Condition the sand by injecting foam and polymer under high pressure to attempt to impregnate the sand layer ahead of the cutting head with the long chain polymer and "bind" it together preventing sloughing of the ground when the chamber was emptied;
- During the last half of the advance (before emptying the chamber), inject bentonite to form an impermeable "cake" of material on the face;
- Have prepared bricks, blocks, timbers and expanding foam to fill any small voids and stabilize the face in front of the cutting head.

The conditioning measures only partially worked, the sand layer again sloughing into the chamber. The water ingress was visibly slow and so the small void was filled with blocks and foam and the face was shored with timbers and foam to allow full cutterhead refurbishment. Most of the rippers, having been damaged by boulders or worn by the ground, were replaced with disc cutters. Only long life carbide insert rippers remained. The gage disc cutters were replaced with long life carbide insert disc cutters as the cutters in this position are subject to more wear.

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

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

A few days later the TBM experienced a failure of an outboard gear box bearing that disintegrated and caused damaged to the teeth of the main bearing's bull gear. The TBM was immobile for one week under 25m head of hydrostatic pressure. However, the machine was powered by six motors so after redressing the damaged teeth of the bull gear and flushing the system continuously to remove metallic debris from the hydraulic oil, the machine continued to advance using the remaining five motors.
Again, due to the good condition of the cutting head the machine continued under high face pressure with good ground conditioning through the sands.

The third and final planned maintenance area was 75% through the bad ground and shortly before the alignment entered the footprint of the Brava Towers. The maintenance area was again planned adjacent to a very short section of "good" ground experienced in Drive 1. However, the tunnels were now more than 2 diameters apart. The 2 hour intervention at this location began with a steady flow of water coming from the sand layer that changed into gushing water flow that started to cause sloughing of the ground above the crown of the machine. The intervention was abandoned but was long enough to change 4 disc cutters and do a good inspection of all cutting tools revealing that they were in relatively good condition. General maintenance was carried out on the entire TBM and back up system. Based on the good condition of the cutting head and maintained TBM, the construction team made the decision to complete the drive through the bad ground without further planned stops, the emphasis now being on continuous boring until the TBM was fully into the sandstone, through the soil/rock interface under Brava Tower.

The machine advanced at an average of 12 rings per day (17m), at 2.2 bar face support pressure at the top of the cutting chamber. The penetration rates, total thrust, torque, grout injection pressures and volumes all remained within expected limits with the torque and thrust increasing and penetration rate dropping as the machine entered the sandstone. The ability to maintain the design face support pressure (not having to drop the pressure to allow forward advance), thus balancing the hydrostatic pressure (therefore not mobilizing any ground water through the sand) allowed the soil to be conditioned only by the injection of foam. Conditioning of granular material, to make the muck into the required impermeable consistency, was therefore achieved at chamber pressures of 2.7 bars (at inlet to screw conveyor). The muck conditioning therefore allowed the operator to properly control muck extraction through a screw conveyor of normal length with almost 200 kPa differential pressure between inlet and outlet (only 9m apart).

GROUND MOVEMENT AND ANALYSES OF SETTLEMENT READINGS

The external monitoring system of the tunnel consisted of surface topographic control points and elevation targets installed at the lowest floors of the buildings along the tunnels' alignments.

The importance of the geological conditions of the soils encountered during Drive 1 underscores the importance of having additional information on the behavior of the soil during the excavation. Four multi-base extensometers were installed before the critical (risk) locations where the readings could provide information about the "loss of ground" due to the excavation, and suggest changes in the TBM parameters (particularly face support pressure and soil conditioning parameters).

The good TBM performance experienced throughout the soil section was also reflected in the results of the monitoring system. A predictable incremental increase of surface settlements over those recorded from Drive 1 has been noticed only in a few locations. Whereas the area that experienced most movement from Drive 1 showed values equal to or lower than those measured during Drive 1. Figure 8 shows the comparison of ground surface settlement for both drives. The capability of maintaining high EPB (face support) pressures and effective soil conditioning had directly contributed to these significant good results.

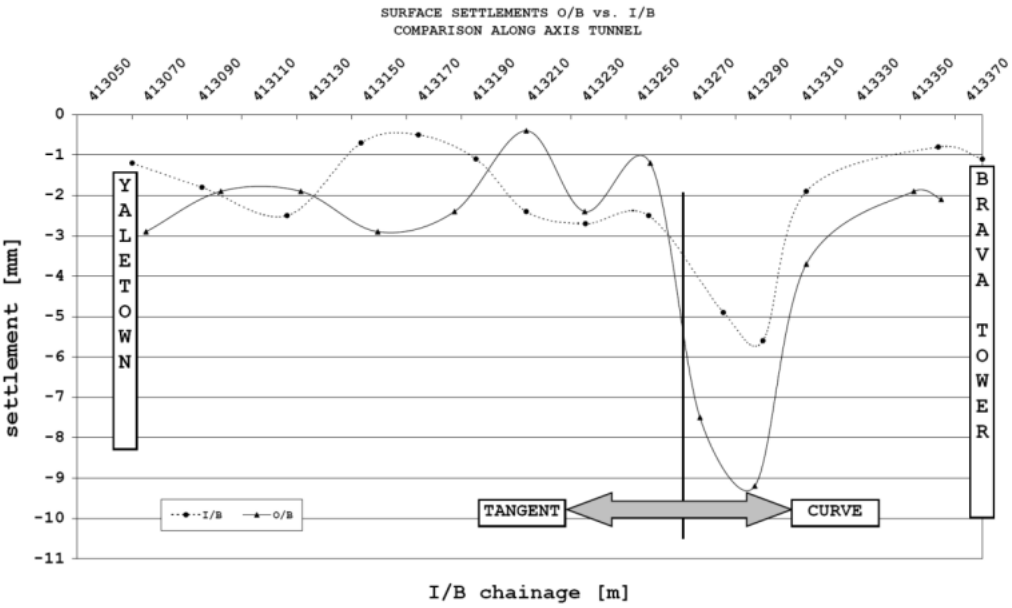

Figure 8 – 1st and 2nd Drive Surface Settlement Comparison along the Axis

Furthermore the calculation of "loss of ground" based on the data from the extensometers and the settlement readings of the cross-sections show a significantly low average value of 0.1% (max 0.25% - min 0.06%) in the 750m of the stretch of sandy ground. Settlement values for Extensometer Z2 and surface settlement section 5 are shown in Figure 9 below.

The vertical displacements recorded inside the lowest levels of the two critical buildings identified in the Risk Assessment (1155 Homer St and Brava Towers) did not exceed 0.8 mm (0.8mm and 0.6mm respectively). The first drive recorded 6mm (maximum) settlement of a Brava Tower footing directly over the tunnel. There was no increase in settlement of the footings over and around Drive 1 due to excavation of Drive 2. Settlements up to 8mm had been predicted for the footings; remarkable, only 0.6mm was realized.

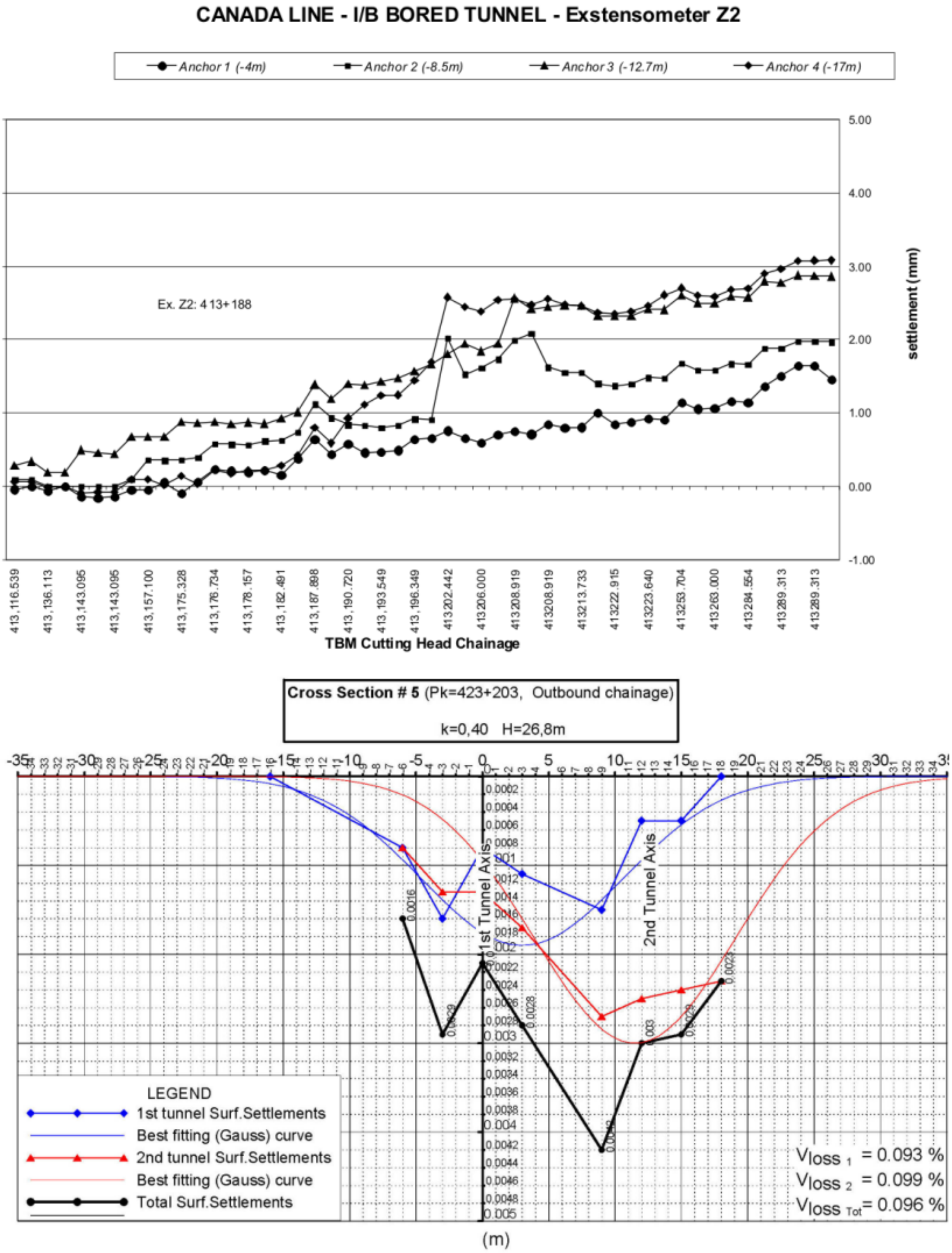

Figure 9 – Extensometer and Cross-Section Results

CONCLUSIONS

Tunneling in a dense urban environment like downtown Vancouver, under the foundations of multi storey residential buildings presents a great challenge with many risks. Add to the list of risks ground conditions that are outside the normal range of the TBM and the challenges become severe. This was the reality for this project where a full face of sand and erodable silts under high water pressure dominated the geology on the section of the alignment with the most critical risks and the toughest geometry; stretching out over 300m.

Several factors were keys in the successful completion of this section of tunnels.

- The systematic approach to Risk Assessment and Risk Management, combining the skills of the construction team with the open communication and cooperation of the contractor, client and owner and the knowledge of two world leading experts in EPBM construction and risk assessment, was fundamental;
- The ability of the construction team to effectively analyze problems, learn from mistakes and plan and implement effective mitigation measures;
- Proper planned maintenance of the TBM combined with the correct face support pressures and ground conditioning agents and the knowledge, experience and communication between the engineers and operators.

The end product is an urban tunnel successfully built in very difficult ground and ground water conditions that are outside of the normal operating conditions of the TBM, with almost zero loss of ground. The risks were properly assessed and mitigated and the residual risks were properly managed. New levels of ground control have been achieved and the range of operation of Earth Pressure Balance Tunnel Boring Machines has been expanded.

ACKNOWLEDGEMENTS

The authors wish to acknowledge the contributions of their colleagues who made this project a success. In particular, the support of Canada Line Rapid Transit Inc, in particular Prof. Eisenstein, InTransitBC, SNC Lavalin Inc. and Prof. Kovari, by working with the contractor to identify and effectively manage the risks, has been paramount.