Geotechnical Baselines for Microtunneling in Difficult Ground Conditions in Calgary, Alberta

Anwar Majid & Kristen Tackney
AECOM Canada Ltd., Calgary, Alberta, Canada
Melanie Gray and Shawn Pillai
The City of Calgary, Calgary, Alberta, Canada

GeoCalgary 2022 October 2-5
Reflection on Resources

ABSTRACT
The Inglewood Sanitary Trunk (IST) is located in Inglewood, a mature and highly developed community in Calgary, consisting of dense residential and heavy industrial properties. The IST is located in a highly congested area with existing infrastructure both above and below ground and limited space for a trunk of the proposed size. Due to the limited space for open cut construction and to reduce socio-economic impacts, the IST was installed using a trenchless method with the exception of a small open cut section near the Bonnybrook Wastewater Treatment Plant. The IST installation was divided into two phases, i.e., Phase 1 and Phase 2. This paper presents the trenchless section of Phase 1 which has recently been completed. Phase 1 involved the installation of an approximately 3370 m long trunk, varying in outer diameter from 2756 mm to 2980 mm, using a microtunneling method and eight sealed shafts.

The geotechnical investigations revealed difficult ground conditions for the tunnel construction including high groundwater and high groundwater inflow rates, variable soil conditions (gravel, sand, clay, clay till), extremely weak to strong bedrock (claystone, sandstone, siltstone) and the presence of extremely hard and abrasive boulders and cobbles. Previous tunnels constructed in the City of Calgary experienced difficulties during construction due to large volumes of groundwater inflow, clogging and stickiness of claystone, abrasive and hard layers of sandstone and extremely abrasive and hard boulders which caused major damage to the tunneling equipment resulting in schedule impacts and claims for differing ground conditions.

This paper presents the challenging aspects of setting geotechnical baselines for such ground conditions and discusses specific design and construction considerations for microtunneling. Approaches taken to optimise baselines considering managing risks and level of conservatism are explained. A comparison is also made between selected baselines and actual ground conditions.

RÉSUMÉ
L'Inglewood Sanitary Trunk (IST) est situé à Inglewood, une communauté mature et très développée de Calgary, composée de propriétés résidentielles denses et d'industries lourdes. L'IST est situé dans une zone très encombrée avec une infrastructure existante à la fois au-dessus et au-dessous du sol et un espace limité pour un tronc de la taille proposée. En raison de l'espace limité pour la construction à ciel ouvert et pour réduire les impacts socio-économiques, l'IST a été installé en utilisant une méthode sans tranchée à l'exception d'une petite section à ciel ouvert près de l'usine de traitement des eaux usées de Bonnybrook. L'installation IST a été divisée en deux phases, à savoir la phase 1 et la phase 2. Cet article présente la section sans tranchée de la phase 1 qui vient d'être achevée. La phase 1 impliquait l'installation d'un tronc d'environ 3370 m de long, variant en diamètre extérieur de 2756 mm à 2980 mm, en utilisant une méthode de microtunnelage et huit puits scellés.

Les investigations géotechniques ont révélé des conditions de sol difficiles pour la construction du tunnel, notamment des eaux souterraines élevées et des débits d'eau souterraine élevés, des conditions de sol variables (gravier, sable, argile, till argileux), un substratum rocheux extrêmement faible à solide (argile, grès, siltstone) et la présence de rochers et galets extrêmement durs et abrasifs. Les tunnels précédents construits dans la ville de Calgary ont connu des difficultés pendant la construction en raison de grands volumes d'afflux d'eau souterraine, du colmatage et de l'adhésivité de l'argile, des couches abrasives et dures de grès et de rochers extrêmement abrasifs et durs qui ont causé des dommages importants à l'équipement de creusement, entraînant des impacts sur le calendrier. et réclamations pour différentes conditions de terrain.

Cet article présente les aspects difficiles de l'établissement de bases géotechniques pour de telles conditions de sol et discute des considérations spécifiques de conception et de construction pour le microtunnelage. Les approches adoptées pour optimiser les lignes de base en tenant compte de la gestion des risques et du niveau de prudence sont expliquées. Une comparaison est également faite entre les lignes de base sélectionnées et les conditions réelles du sol.

1 INTRODUCTION

1.1 Project Background

The City of Calgary (the City) identified the need for additional sanitary sewer capacity to augment the two existing 15th Street Trunks (existing trunks) to meet future demands from wet weather flows and future residential growth. The existing trunks convey sewage flows from the northern half of the City and neighbouring communities of Cochrane and Airdrie to the Bonnybrook Wastewater Treatment Plant (BBWWTP) in Calgary.

The original study proposed the addition of a third trunk along 15th Street SE from the Bow River to the BBWWTP. It was found early in the preliminary design stage that the 15th Street SE corridor would be a challenging route due to numerous reasons including:

- The corridor was highly congested with many utilities including the two trunks
- The corridor crosses a densely developed residential area in the north near the Bow River and heavy industrial land use to the south
- The corridor crosses Alyth Yard, one of Canadian Pacific Railway's (CPR) largest and busiest rail yards in Western Canada which was known to have contaminated soils
- Portions of the original 15th Street SE right-of-way had been closed with easements registered to and/or encroached on by adjacent heavy industries

An alternate concept was developed that "offloaded" flow from the existing trunks by intercepting flows from the inner city via the Inner-City Trunk and from the west via the West Memorial Trunk. The routing of the new offload trunk was dubbed the Inglewood Sanitary Trunk (IST). The IST allowed for construction in a less congested environment with far less community impacts and a dramatically lower risk profile than adjacent to the existing trunks. However, ground conditions along the IST were challenging; as a result, the IST was largely installed with trenchless microtunneling construction methods which further reduced community impacts.

1.2 Project Description

Located in Calgary, Alberta, Canada, the IST is sanitary sewer which spans approximately 4250 m from the Bow River to the BBWWTP, as shown on Figure 1. The IST installation was divided into two phases, i.e., Phase 1 and Phase 2. Phase 1 has recently been completed, largely by microtunneling methods except a small portion near the BBWWTP which was completed using conventional open cut methods. Design of Phase 2 is nearing completion and consists of a twin inverted siphon crossing the Bow River. Construction of Phase 2 is expected to be completed by 2024.

This paper presents the trenchless section of Phase 1 of the IST which was installed using microtunneling boring machines (MTBMs) and sealed shafts to mitigate constraints and restrictions which would otherwise make the construction of a large diameter trunk in this location impractical or nearly impossible.

The trenchless section of Phase 1 was installed using seven circular shafts (Shaft 1-2 to Shaft 1-8) ranging in diameter from 8 m to 11.5 m and one 11.5 m by 8 m elliptical shaft (Shaft 1-1). The shafts were constructed to depths of approximately 16 m. Two different sizes of reinforced concrete microtunneling pipes were used for Phase 1 of the IST as below:

- Shaft 1-1 to Shaft 1-3: Pipe Inner/Outer Diameters = 2286 mm/2756 mm
- Shaft 1-3 to Shaft 1-8: Pipe Inner/Outer Diameters = 2500 mm/2980 mm

The shaft locations and locations of Phase 1 and Phase 2 are shown on Figure 1.

1.3 Geotechnical Challenges

A desktop study and a preliminary geotechnical investigation was performed during the preliminary design stage to collect information on the subsurface soil, bedrock and groundwater conditions in the project area and identify geotechnical challenges that may be encountered along the IST alignment. The desktop study consisted of a review of available information including geotechnical data reports and geotechnical baseline reports for similar projects in the area (between 2007 and 2017), geological information including mapping available from the Alberta Geological Survey and relevant information from installation methods of similar tunneling projects in the Calgary area (constructed between 2005 and 2016).

Figure 1. IST Alignment

The major challenges identified from the desktop review and preliminary geotechnical investigation were:

- Variable ground conditions along the IST alignment consisting of gravel, sand, silt, clay, clay till and bedrock consisting of claystone (CS), siltstone (SI) and sandstone (SS)
- Cobbles and boulders of variable sizes within gravel, sand and clay till layers
- Clogging potential of clay, clay till and CS
- Slake durability and swelling potential of CS
- Uniaxial Compressive Strength (UCS) and abrasiveness of boulders
- UCS and abrasiveness of SS
- High groundwater and groundwater inflow rates through sand and gravel layers

Owing to the above challenges, it was decided to use microtunneling to install a major portion of Phase 1 of the IST and to perform a detailed geotechnical investigation to prepare a Geotechnical Baseline Report (GBR).

2 DETAILED INVESTIGATION

2.1 Investigation Program

A detailed investigation program was performed to characterize the subsurface conditions along the IST alignment. The investigation program consisted of the following tasks:

- Drilling testholes at each shaft location and at 100 m spacing along the IST alignment. All testholes extended at least 2.5 times the tunnel diameter below the tunnel invert
- Performing hydraulic conductivity testing and pump tests at select locations to determine the hydraulic conductivity and groundwater inflow rates in the tunnel and shafts
- Performing a geo-environmental investigation to investigate the presence or absence of contaminants in soils, bedrock and groundwater

2.2 Laboratory Testing Program

Testing was performed on soil and bedrock samples to determine the index properties and strength properties of the soils and bedrock. The testing included:

- Index testing (moisture contents, sieve and hydrometer analyses and Atterberg Limits) – Soils
- Atterberg Limits testing – CS
- UCS and Cerchar Abrasiveness testing – Boulders
- Direct shear testing – Soils and CS
- Slake durability testing – CS
- X-Ray diffraction and free swell testing – CS
- UCS and Cerchar Abrasiveness testing – SS
- pH, Resistivity, Sulphate and Chloride Content testing – Soils and CS

For the environmental assessment of soils, CS and groundwater, the following testing was performed:

- Screening of vapours from soil samples for methane, hydrogen sulphide, carbon monoxide, lower explosive limit and oxygen content
- Analytical laboratory analysis of soil samples for benzene, toluene, ethylbenzene and xylene (BTEX), petroleum hydrocarbon fractions 1-4 (PHC F1-F4), polycyclic aromatic hydrocarbons (PAHs), volatile organic carbons (VOCs), salinity and metals
- Analysis of groundwater samples for BTEX, PHC F1-F4, VOCs, PAHs, metals and routine parameters

2.3 Summary of Geotechnical Investigation

The geotechnical investigation revealed the complex ground conditions along the IST alignment indicating that the tunneling would take place in soft ground (soils), in bedrock (CS, SI and SS) and in mixed ground conditions (soils and bedrock). The subsurface soil/bedrock units encountered along the IST alignment were divided into seven major stratigraphic units as below:

- Topsoil
- Asphalt
- Cohesive Soil Unit – clayey silt, silty clay
- Cohesive Glacial Till Unit – clay till
- Fine Granular Soil Unit – silty sand
- Coarse Granular Soil Unit – sandy gravel, gravelly sand, silty gravel, gravel and sand, sand and gravel and sand with some gravel
- Bedrock – CS, SI and SS

3 GEOTECHNICAL BASELINES

3.1 Baseline Geologic Profile

The baseline geologic profile along the tunnel alignment is shown on Figures 2 and 3. The baseline geologic profiles indicate that the tunnel sections are completely in soils (soft ground conditions), completely in bedrock and/or partially in soils and bedrock (mix ground conditions).

3.2 Baseline Ground Behaviour

For the description of the baseline behaviour of the overburden soils, the Tunnelman Ground Classification System developed by Terzaghi (1950) and modified by Heuer (1974) was adopted. A major portion of the IST was found to be in granular soils; therefore, a baseline was provided for the anticipated behaviour of coarse and fine granular soils at an unsupported vertical excavation face. The baseline was “coarse and fine granular soils will exhibit fast raveling or cohesive running above groundwater table but will immediately flow below the groundwater table at unsupported vertical excavation face. The coarse and fine granular soils are not suitable for open face tunneling.”

3.3 Boulders

Boulders were encountered in all soil units. The boulders varied in size from 200 mm to 900 mm based on actual measurements of boulders collected from testpits for the Bowness Sanitary Offload Trunk Project (AECOM 2014) and hydrovac holes completed for the IST. Boulders were strong to extremely strong and were extremely abrasive based on test results. The following baselines were provided for the number, size, UCS and Cerchar

Abrasiveness Index (CAI) of the boulders for selecting the MTBM cutter head and cutting tools:

- Assume three boulders between 200 mm and 500 mm in size in every 1 m^3 of coarse granular soils
- Assume one boulder between 200 mm and 500 mm in size in every 1 m^3 of fine granular soil, cohesive soil unit and glacial till unit
- Assume one boulder between 500 mm and 900 mm in size in every 75 m^3 of overburden soils
- Assume an average UCS of 260 MPa and a maximum UCS of 380 MPa
- CAI = 4.9

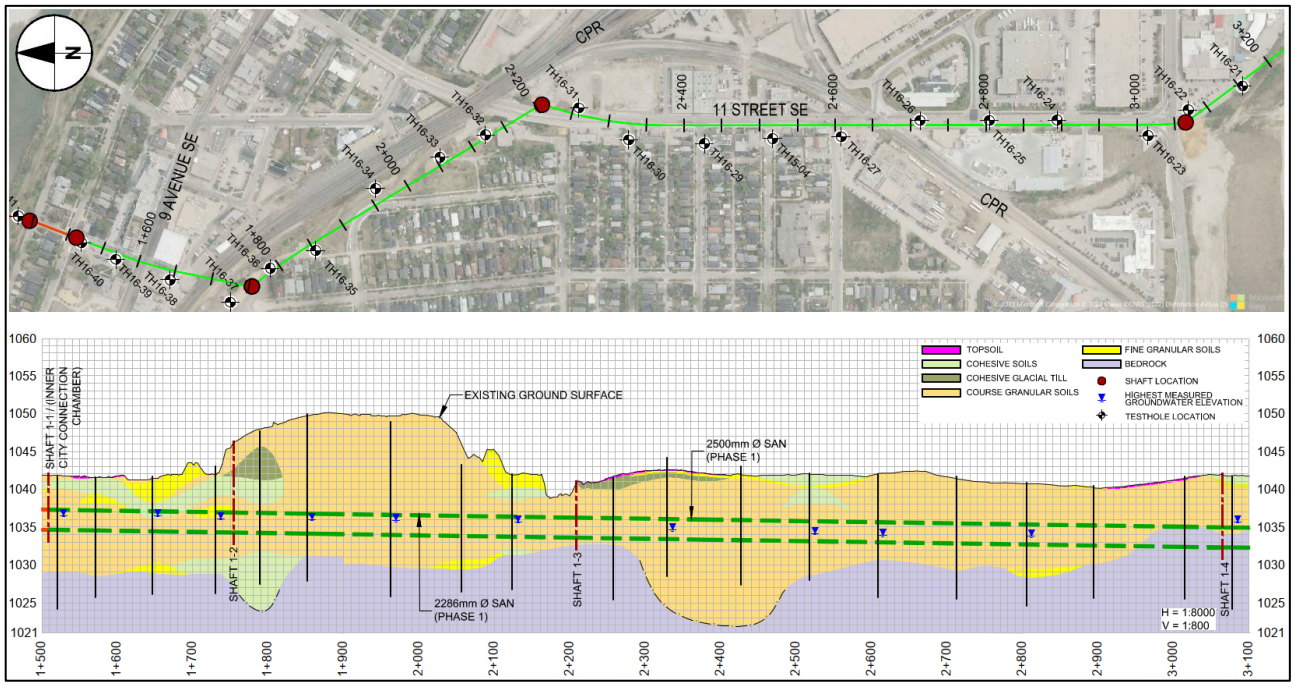

Figure 2. Baseline geologic profile STA 1+500 To 3+100

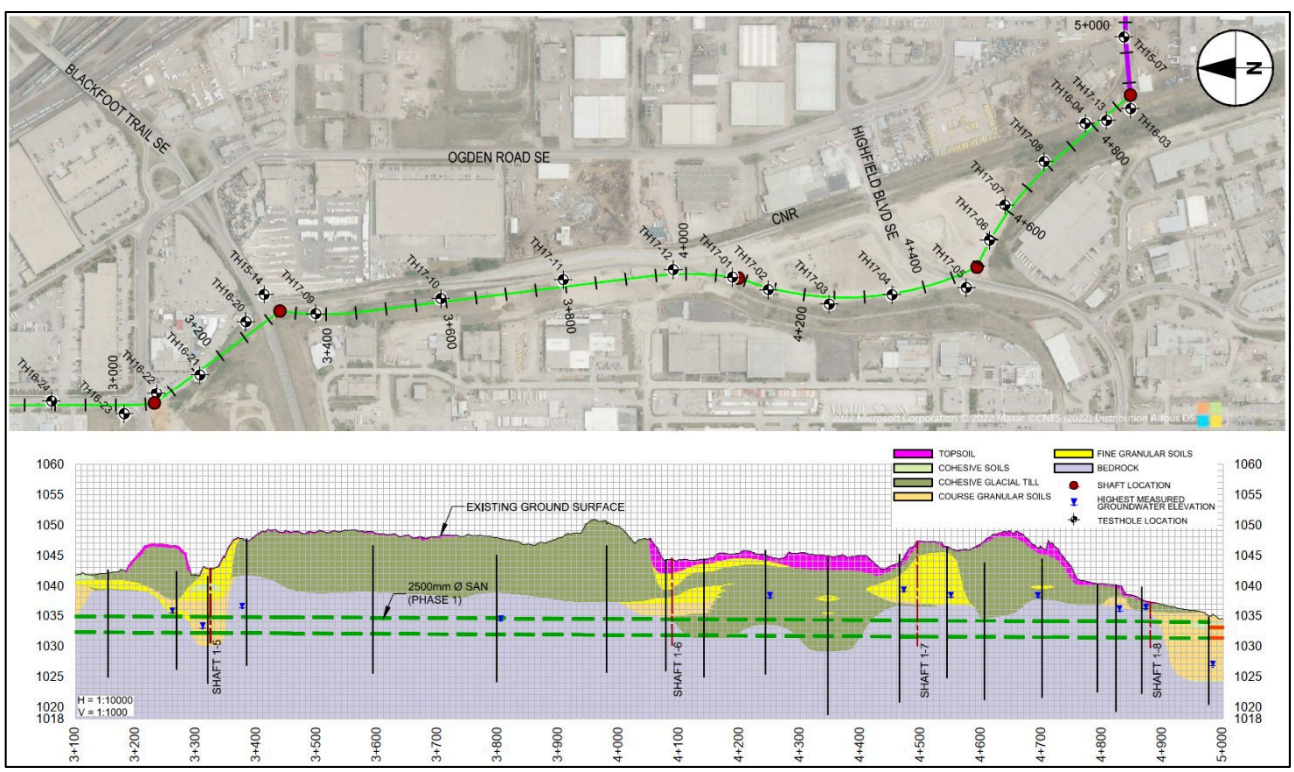

Figure 3. Baseline geologic profile STA 3+100 To 5+000

3.4 Clogging Potential – Cohesive Soil Unit, Cohesive Glacial Till Unit and CS

The excavation of cohesive soils and CS with a MTBM creates the potential for stickiness of the cohesive soils/CS at the cutter head, excavation chamber surface, in the transport system and in the separation plant. This stickiness may result in the clogging and blockages of the cutter head, excavation chamber, muck transport system and separation plant leading to delays and slower MTBM advance rates.

The potential for clogging while tunneling through the cohesive soil, cohesive glacial till units and CS were assessed using the universal classification diagram for critical consistency changes regarding clogging and dispersing suggested by Hollmann and Thewes (2013) as shown on Figure 4. The Liquid Limit (WL), Plastic Limit (WP) and natural moisture content (Wn) were used to assess the corresponding clogging potential of the cohesive soil, cohesive glacial till units and CS.

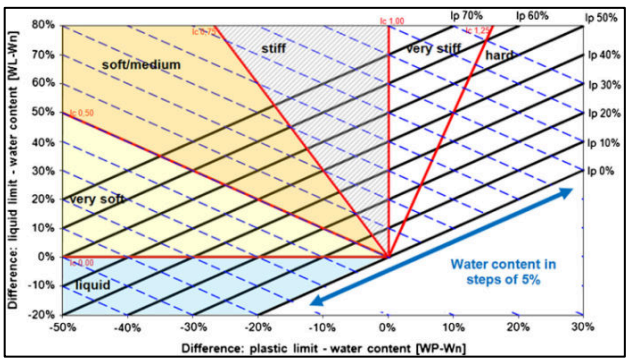

Figure 4. Universal classification diagram for critical consistency changes regarding clogging and dispersing (from Hollmann and Thewes, 2013)

Based on Hollmann and Thewes (2013), the stickiness of the cohesive soils, cohesive glacial till and CS in their in-situ state varies from little to strong-clogging; however, the addition of water during tunneling (groundwater, support liquid, cleaning water, etc.) may change the stickiness of the cohesive soils, cohesive glacial till and CS from medium-clogging to strong-clogging. For baseline purposes, the cohesive soil, cohesive glacial till and CS were assumed to have a strong-clogging potential.

Frequent muck/slurry testing, cleaning and flushing of the cutter head and muck transport system and the use of anti-clogging agents were specified to reduce stickiness, mitigate clogging potential and reduce potential delays caused by stickiness of the cohesive soils, cohesive glacial till and CS.

3.5 Uniaxial Compressive Strength – bedrock

The UCS of bedrock is critical for the selection of the MTBM cutting tools. UCS tests were performed on intact samples of SI and SS (ASTM D 7012 Method C). The measured UCS values varied from 1.35 MPa to 69.89 MPa indicating that the SI and SS layers are weak (R_0) to strong (R_4) in accordance with the Journal of the International Society for Rock Mechanics (1981). The frequency distribution of measured UCS values is presented in Figure 5.

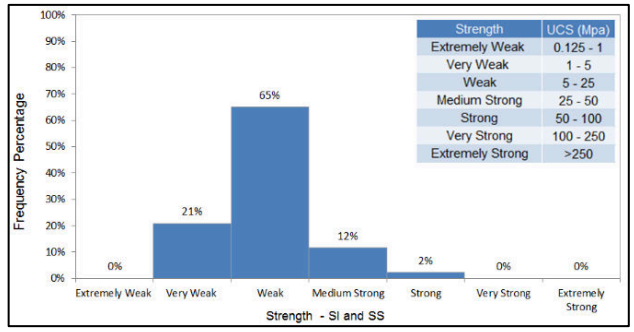

Figure 5. Frequency distribution of measured UCS – SI, SS

UCS of up to 160 MPa has been reported for SS for a similar project in Calgary; therefore, a UCS of 90 MPa was baselined for the selection of cuttings tools for the MTBM in the tunnel sections and for bedrock excavation in the shafts.

3.6 Slake Durability – CS

The CS in the Calgary area is extremely weak to weak and disintegrates when exposed to wetting, drying or abrasion. The ability of the CS to resist the effect of repeated cycles of wetting/drying and abrasion was assessed by performing slake durability tests in accordance with ASTM D4644. The slake durability test results are presented in the form of a slake durability index, which varies from 0 % (very low slake durability) to 100 % (very high slake durability). The slake durability index of the CS was less than 25 for the majority of the tested samples indicating that the CS has very low slake durability. Figure 6 shows a CS sample with very low slake durability subjected to cycles of wetting and drying.

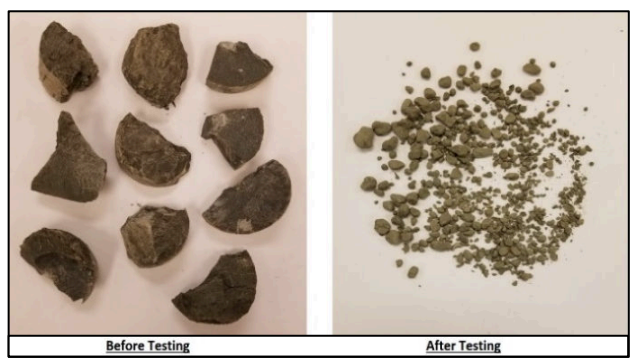

Figure 6. Slake durability test on CS (ASTM D 4644)

For baseline purposes, the CS was assumed to have a very low slake durability when exposed to a wetting and drying weathering process and will slake/disintegrate immediately at excavation and tunnel faces exposed to air and water.

3.7 Swell Potential – CS

The CS is comprised of non-clay and clay minerals and is known to have swelling potential. X-Ray diffraction tests were performed to investigate the swell potential of the CS. Based on the X-Ray diffraction test results, the measured non-clay minerals (quartz, feldspar, plagioclase, calcite and dolomite) in CS varied from 36.4 % to 85.5 %; whereas clay minerals in CS varied from 14.5 % to 63.6 %. The test results also indicated that CS has a higher percentage of non-clay minerals compared to clay minerals. Based on the test results, the clay minerals in CS consist of both non-swelling (illite and mica, kaolinite and chlorite) and swelling minerals (smectite). The non-swelling clay minerals in CS varied from 10.1 % to 25.4 % and swelling clay minerals varied from 2.8 % to 45.1 % which indicate that the CS has variable swelling potential.

To measure the magnitude of the swelling strain of CS, free swell tests were also conducted in accordance with ASTM D4546. A maximum free swelling strain of 3.9 % was measured for CS, as shown on Figure 7.

For baseline purposes related to the selection of the overcut around the MTBM and selection of lubricants to reduce swelling of the CS and friction on the MTBM, the CS was assumed to have a medium swelling potential with a baseline maximum swelling strain of 4 %.

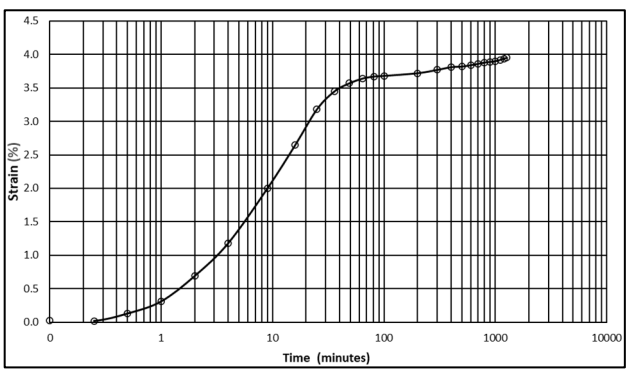

Figure 7. One dimensional free swell test (ASTM D4546 Method B)

3.8 Cerchar Abrasiveness Index – SI and SS

The abrasiveness of bedrock is a major factor that impacts the rate of wear of the MTBM cutting tools. Cerchar abrasiveness tests were performed on SI and SS samples to assess their CAI. The measured CAI of SI and SS samples varied from 0.67 to 1.92, which indicates a low to medium abrasiveness in accordance with ASTM D7625-10. A sample test result is shown on Figure 8. For baseline purposes, the bedrock, including SS, SI and CS, was assumed to have a CAI of 1.92.

3.9 Groundwater Inflow Rates

The measured groundwater inflow rates were high, and dewatering was not considered to be practical and/or feasible; therefore, dewatering for the purpose of lowering the groundwater level was not permitted. To control dewatering, a sealed shaft construction method was specified. To control groundwater inflow into the tunnel, gasketed precast concrete microtunneling pipes and Pressurized Closed Face MTBMs were specified.

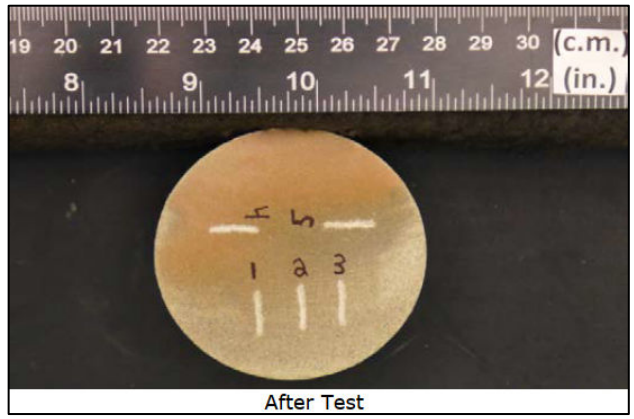

Figure 8. Cerchar abrasiveness test on SS (ASTM D7625)

3.10 Combustible Gases

Combustible gases, such as methane and hydrogen sulphide, were generally not detected in the testholes. Where encountered, the concentrations of methane were less than 3 % of the Lower Explosive Limit (LEL). For baseline purposes, a non-gassy classification was assumed for the soil and bedrock units; however, the implementation of air monitoring and ventilation was specified. It was also specified to equip MTBMs with continuous flammable gas monitors and alarms to detect any gas during construction.

3.11 Soil and Groundwater Quality

Analytical testing indicated that the soils, bedrock and groundwater were not contaminated; however, concentrations of some soil/bedrock parameters were slightly above the Alberta Tier 1 Soil Remediation Guidelines and the concentration of some groundwater parameters were slightly above the City Bylaw 14M2012 (amended by Bylaw 9M2015) Alberta Tier 1 Groundwater Remediation Guidelines. For baseline purposes, it was assumed that the in-situ soil, bedrock and groundwater are not contaminated; however, the contractor was required to monitor, test and analyze the soil and groundwater and perform necessary treatment to meet the applicable disposal guidelines provided in the project specifications.

4 PROJECT REQUIREMENTS

Based on the ground conditions and geotechnical baselines, it was required to:

- Use Pressurized Close Face MTBMs capable of tunneling though variable ground conditions
- Use pre-cast concrete gasketed microtunneling pipes to control groundwater inflow into the tunnel
- Use a sealed method of shaft construction to reduce dewatering. Dewatering for the purpose of lowering the groundwater table was not permitted

5 CONSTRUCTION

5.1 MTBMs and Shafts

A slurry type Pressurized Closed Face MTBM, as shown on Figure 9, was used for the project. The MTBM was equipped to tunnel though the variable ground conditions shown on Figures 2 and 3. Sealed methods of construction were used for all shafts. Most of the shafts were installed using cast-in-place sunk concrete caisson methods as shown on Figure 10. One shaft was installed using a secant pile method as shown on Figure 11. The secant pile method was only used for the elliptical shaft.

Figure 9. Pressurized closed face slurry type MTBM

Figure 10. Circular sunk concrete caisson shaft

Figure 11. Elliptical secant pile sealed shaft

5.2 Construction Issues, Claims and Resolutions

The ground conditions were generally consistent with the GBR prepared for the project; however, a few ground issues were encountered during shaft construction and tunneling. A brief description of these issues is provided in the following sections.

5.2.1 Frac-outs

Frac-outs were encountered at two locations. At both locations the frac-outs occurred due to encountering unforeseen objects resulting in increased pressures. At one location, the frac-out occurred after the MTBM encountered a tie-down anchor buried within the CPR right-of way (Figure 12). The tie-down anchors were installed during construction of an adjacent building. Most of the anchors were removed prior to tunneling; however, one or more tie-down anchors below the CPR right-of-way could not be removed. The frac-out shown on Figure 12 damaged the pavement. This area was also freshly backfilled with granular material following removal of the tie-down anchors. The area was cleaned of slurry and the pavement was repaired. At the second location, the drilling fluid flowed through an Enmax vault which was open from the bottom. The vault was located a sufficient distance away from the tunnel centerline. No obvious reason was found for this frac-out. The slurry was hydrovaced from the vault and the vault was repaired. After review of all information, the contractor's claims for the frac-outs were found legitimate and were paid. Frac-outs could be avoided/minimized by performing a detailed review of existing as-built information and records of any construction along the tunnel alignment to identify and remove man-made obstructions, voids, etc., that may trigger frac-outs. Frac-outs can also be avoided by increased soil cover above the tunnel and by maintaining the slurry pressure well below the soil resistance. Contingency plans should be in place to contain and clean the slurry and repair any damaged structures should frac-outs occur.

Figure 12. Frac-out near CPR right-of-way

5.2.2 Boulder in Shaft

A boulder > 2 m in size was encountered in Shaft 1-4 as shown on Figure 13. The maximum baselined size of boulders was 900 mm; therefore, this boulder was considered a change in ground conditions. The contractor's claim was found legitimate, was accepted and paid.

Figure 13. Boulder encountered in shaft excavation

5.2.3 Damage to the MTBM

Damage to the MTBM cutter head was reported at two locations while tunneling between Shaft 1-3 and Shaft 1-4. Rescue shafts were excavated to expose the MTBM and repair the cutter head as shown on Figure 14.

Figure 14. Rescue shaft to expose MTBM cutter head

The cutting tools were believed to have been damaged by boulders and the contractor believed that the numbers of boulders and their UCS and abrasiveness was greater than the baselined values; therefore, boulders were collected, counted and their sizes, UCS and CAI were measured. The boulders collected from a rescue shaft are shown on Figure 15.

Figure 15. Boulders collected from rescue shaft excavation

The maximum boulder strength from seven samples was 249 MPa which was less than maximum baselined value of 360 MPa. The maximum measured CAI was 4.87 which was less than baselined value of 4.9. The number of boulders were equal to or slightly less than the baselined numbers; therefore, this claim was partially paid as boulders were not measured and tested in the first rescue shaft and the contractor claim included the observation of metal pieces in the returned slurry.

6 CONCLUSIONS

The project was completed on the schedule and budget established during construction tendering. Major changes in ground conditions were not encountered during construction. The total cost of the geotechnical investigation program was approximately $1.5 million which is 2 % of the project construction cost of $78.6 million. The total claimed amount paid to the contractor for differing ground conditions was approximately $400,000 which is half a percent of the project construction cost. This indicates that the level of the geotechnical investigation program was appropriate, resulting in minimal claims and no delays. The following conclusions can be drawn from this case study:

- Performing adequate geotechnical, hydrogeological and environmental investigations are critical in the selection of construction means and methods for microtunneling projects
- A Geotechnical Baseline Report is an effective tool for resolution of disputes and claims with respect to changes in ground conditions

7 ACKNOWLEDGEMENTS

The authors would like to acknowledge the support provided by the City, Ward & Burk Microtunneling Ltd., Earth Drilling Co. Ltd. and AECOM geotechnical engineers, scientists and technicians involved on this project from start to end.

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