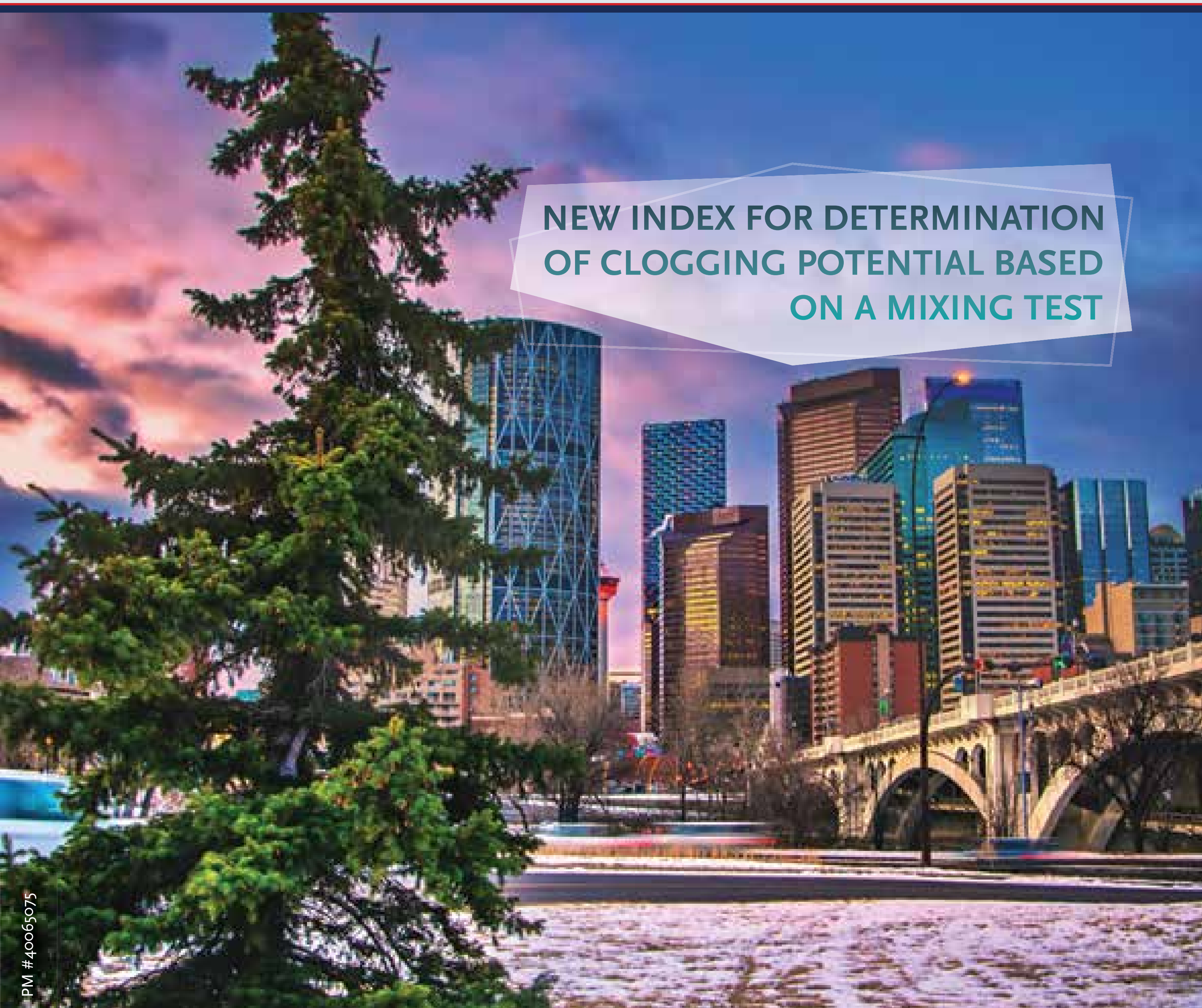
2021 | NORTHWEST FALL/WINTER

# TRENCHLESS JOURNAL

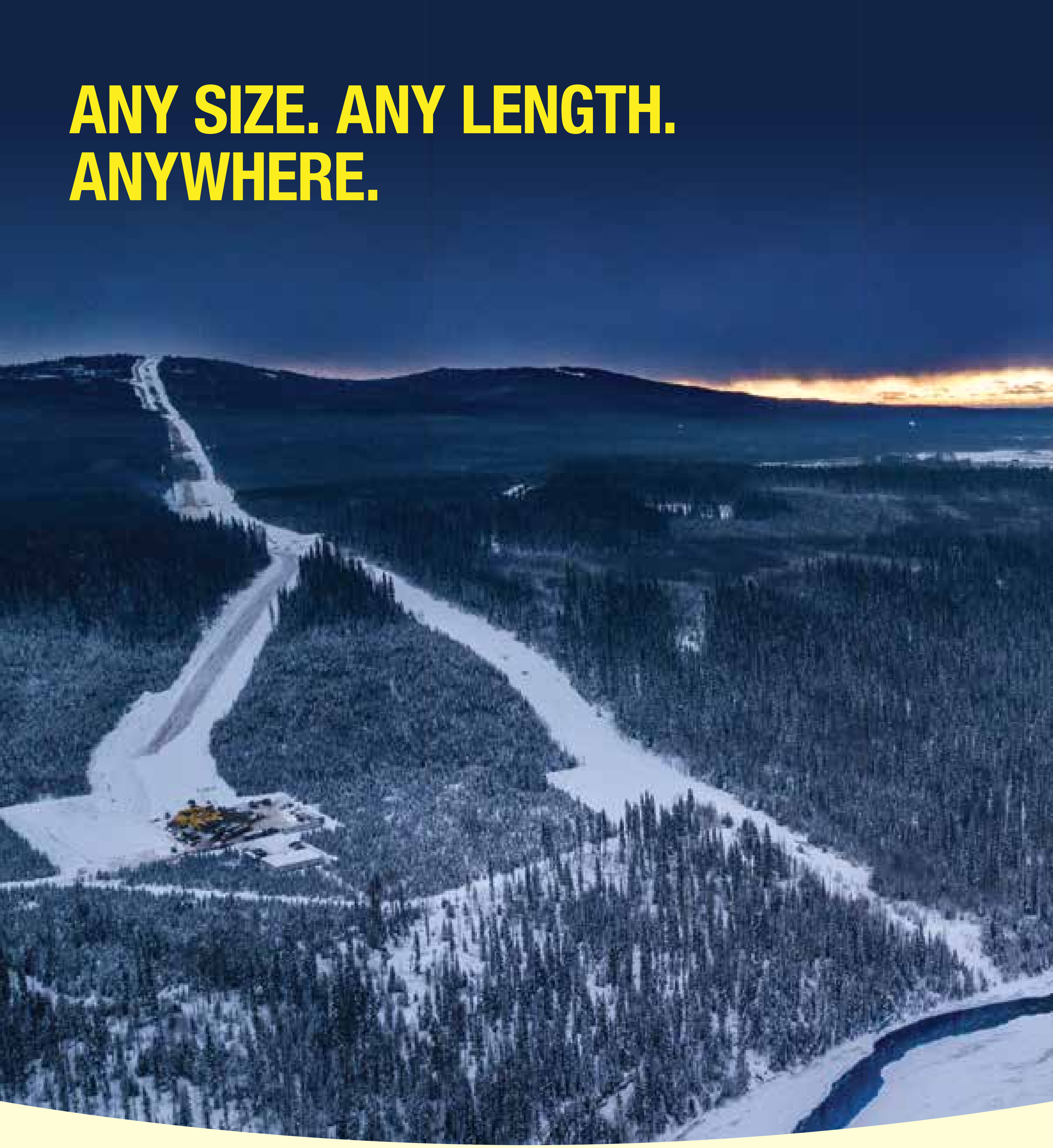
THE OFFICIAL PUBLICATION OF THE NORTHWEST CHAPTER OF
THE NORTH AMERICAN SOCIETY FOR TRENCHLESS TECHNOLOGY



## NEW INDEX FOR DETERMINATION OF CLOGGING POTENTIAL BASED ON A MIXING TEST

PM #40065075

DESIGNED & CONSTRUCTED MICROTUNNEL FOR DOWNTOWN CALGARY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2021 | NORTHWEST

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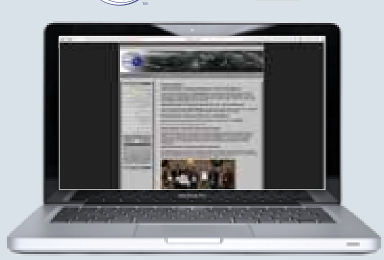


ON THE COVER: Winter sunrise over downtown Calgary. | dreamstime.com

# SEE YOU AT NO-DIG NORTH!

Vancouver Convention Centre, Downtown Vancouver, BC

Welcome to the Fall/Winter issue of the *Northwest Trenchless Journal*, the official publication for members of the NASTT-NW Chapter. The big news in this issue is the **No-Dig North** event coming up in November. For more information and to register to attend, please visit *www.nodignorth.ca*. We hope you can join us in Vancouver, BC this year!

## In This Issue

In this issue of the magazine, we look at a new index for determining clogging potential when using tunnel boring machines. You can find this article on pages 13–15.

Also in this issue, we put the spotlight on a case study about the designed and constructed microtunnel for the downtown Calgary transmission reinforcement project. This No-Dig 2021 case study begins on page 19.

## Find Us Online

Remember to join the Northwest Chapter on LinkedIn, at *www.linkedin.com/groups/4430433*.

You can also find the most recent issue of the Northwest Trenchless Journal online at *https://nastt.org/resources/regional-chapter-magazines*.

## Coming Up

The Chapter magazine is published twice a year. The next issue of the Northwest Trenchless Journal is scheduled for distribution in the spring of 2022. It will include the Project of the Year winners, and the NW Chapter's 2022 Buyers' Guide.

We are looking for relevant, regional feature content to share with members. Please contact Carlie Pittman at *pittmanc@ae.ca* by April 3, 2022 for more information and to let us know if you have an article or paper you would like to contribute to the next issue of this publication.


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# INNOVATION & RECOVERY IN THE TRENCHLESS INDUSTRY

Hats off to NASTT and the Board of Directors for making the decision to host an in-person and virtual No-Dig Show in Orlando this past March. The in-person conference was well attended with safe networking and educational events. Both the No-Dig Planning Committee and the No-Dig Development Committee are hard at work to bring you exciting new networking events in Minneapolis in April 2022. Make sure to mark your calendars and save the date for what we expect to be a record level of attendees.

The No-Dig Show in Orlando was a wonderful opportunity to get back to normal and reconnect with our trenchless colleagues. Now that things are beginning to safely re-open throughout North America, we are excited to see the No-Dig North Show and regional conferences scheduled to take place in the last quarter of 2021. Our Canadian partners have not been able to participate in many events over the last two years. There is much anticipation for the in-person 2021 No-Dig North at the Vancouver Convention Centre in Vancouver, British Columbia on November 8 to 10. The show will consist of two days of technical paper presentations and industry exhibits in the trenchless technology field. Pre-event Good Practices Courses will also be available to attend on Monday, November 8, followed by an opening reception for all attendees.

"In many cases these regional conferences are the first exposure to trenchless technology."

Many of the other regional chapters have finalized their conference program/events for this year. There is a lot of excitement around the South Central and Northeast Chapter Conferences being held in Sugar Land, TX and West Point, NY. In many cases these regional conferences are the first exposure to trenchless technology. The regional chapters are the grassroots of this society and what continues to grow trenchless technology at a local level.

NASTT Board of Directors has recently published the Strategic Plan 2021–2023 which identifies the importance of our committees. The growth and leadership within these committees have allowed industry experts to work together for the common good of the trenchless industry. If you want to represent a part of trenchless technology, then I encourage you to get involved with NASTT committees or at the regional level and reap the benefits. Contact NASTT if you are interested in a committee. Specifically, our Students, Education, and Publications committees are very active working on the next webinar/training documents.

It is an exciting time in the trenchless industry, and we are leading the way in training, education, and research. There may be uncertainty in our world, **but one thing we know for sure, Infrastructure is Essential**. The trenchless industry is strong and resilient, and here to stay!

# Call for Submissions

If you would like to submit your project paper or other content and photos for an upcoming issue of this Northwest Chapter magazine, please contact Carlie Pittman, Magazine Committee Chair, at *pittmanc@ae.ca*.

Editorial submissions for the *Northwest Trenchless Journal* are welcome and due for our next publication by April 4, 2022.

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# NEW INDEX FOR DETERMINATION OF CLOGGING POTENTIAL BASED ON A MIXING TEST

## BACKGROUND

Clogging can be a major problem for projects conducted using earth pressure balance tunnel boring machines (TBMs), especially for fine cohesive soils with moisture content in the middle range. While geotechnical testing is standard for tunnelling projects, testing done in advance of the project is limited to the borehole or sampling sites. For in-progress projects, onsite clogging potential tests using simple equipment can be used to complement existing geotechnical information and mitigate the risk of clogging.

There are many factors to consider when developing a simplified test model to accurately assess clogging potential. These include the inherent tendency of the soil to stick to itself (cohesion), and the interaction between the soil and a surface (adhesion). Analytical approaches such as measurements of adhesive and cohesive forces are attractive in terms of simplicity and ease of application. However, they do not fully capture the complex, dynamic nature of the interactions between the ground and the cutter face. Since it is impractical to replicate the motion of a TBM in a laboratory setting at full scale, compromise is necessary. Physical methods such as mixing tests – while they do not fully capture the motion of a cutter at a TBM face – have benefits over other approaches, since they are dynamic systems and involve shear forces.

This current work focused on assessing clogging potential based on a simple mixing test. A new index was also introduced to improve reproducibility of the results.

## METHOD

Building on mixing tests previously described in the literature (Zumsteg and Puzrin, 2012; Kang et al., 2018), portable, readily available equipment (a Hobart mixer and beaters of various shapes) was used to refine an existing test method and gain insight into the clogging potential. To simulate soils with a range of properties, mixtures of kaolin and bentonite (ranging from pure kaolin to 10%, 30%, and up to 90% bentonite, indicated by $M_{10}$, $M_{30}$, etc.) were used for test samples. Atterberg tests were done to determine the plastic limit and liquid limit for each of the mixtures.

To perform the mixing test, bentonite and kaolin were mixed (as dry powders) in the appropriate ratios. Then, a predetermined volume of distilled water – enough to give the targeted moisture content – was added to the sample while mixing for a set time. After mixing, the beater was then pulled out of the sample and weighed. Inevitably, some of the sample remained stuck on the beaker.

In previous studies, the stickiness ratio, λ, i.e., the ratio of the mass of material stuck to the beater ($G_{MT}$) to the total sample mass ($G_{TOT}$) has been used as the index for a mixing test (Zumsteg and Puzrin, 2012). However, since the amount of sample stuck to the beater depends on the surface area – i.e., the contact area between the metal beater surface and the sample – the index *M/A*, or mass of soil stuck to the beater (*M*) per unit area covered by the sample (*A*), was investigated in this work. Although the same sample mass was generally used across the different mixtures, the distance that the sample extended up the beater (*d*) varied. The beater area (*A*) was determined using a method based on a three-dimensional computer model of the beater and measurement of *d*.

Mixing test results were mapped onto the soil classification chart developed by Hollman and Thewes (2013), another tool proposed to assess clogging potential based on plastic and liquid limits and water content. A sensitivity analysis was also conducted on sample mass and beater shape to determine whether these parameters influenced the mass-to-surface-area ratio (*M/A*).

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# NEW INDEX FOR DETERMINATION OF CLOGGING POTENTIAL BASED ON A MIXING TEST

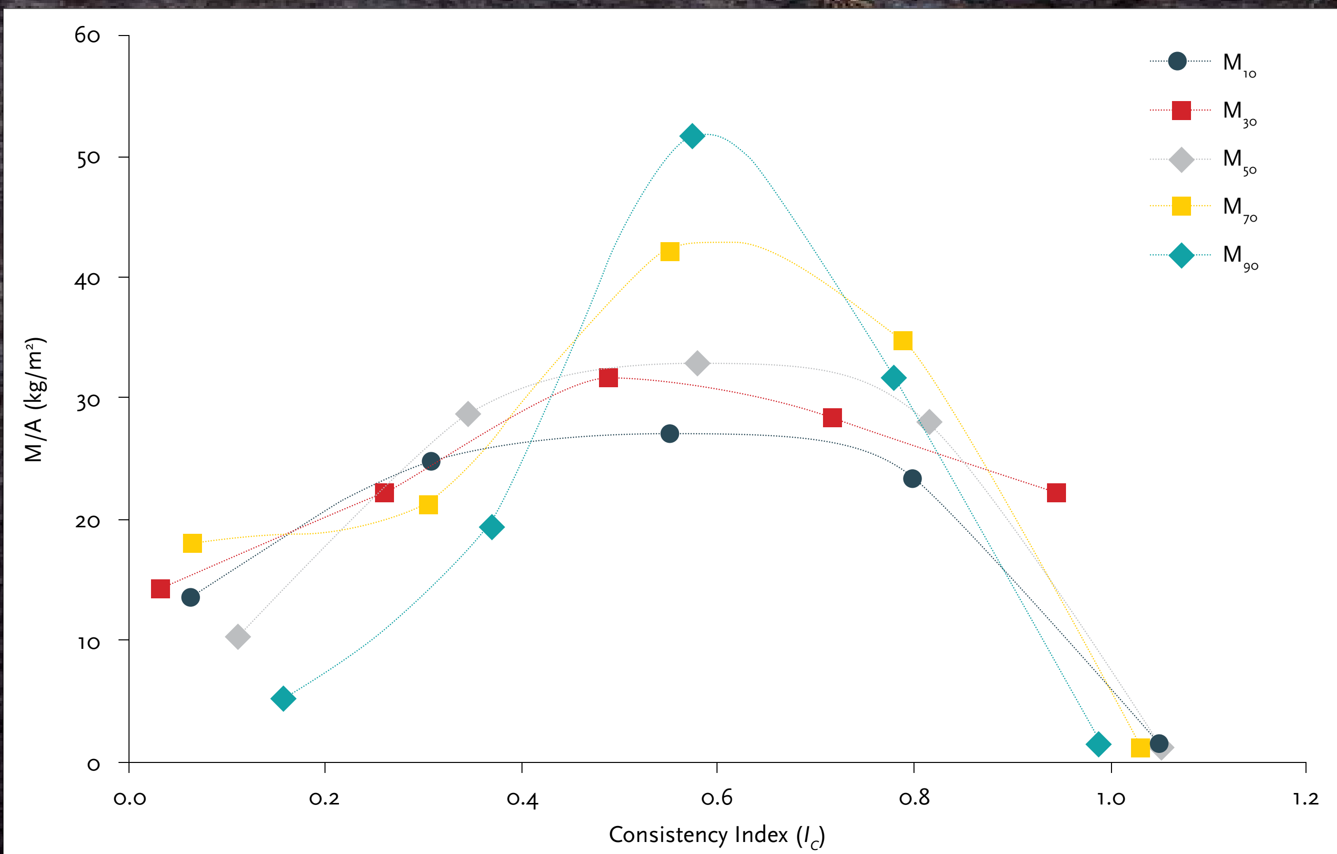


***Figure 1.** Mass-to-surface-area ratio vs. consistency index.*

## RESULTS

The mass-to-surface-area ratio (*M/A*) peaked when the consistency index was between 0.5 to 0.6 for each sample, as shown in **Figure 1**. Each data point represents three replicate measurements. The peaks correspond to the region of high clogging in the empirical clogging diagram proposed by Hollman and Thewes (2013), as indicated in **Figure 2(a)**. The new index *M/A* increases with consistency index, reaches a peak, and then declines, as expected. It was also noted that more reproducible test results were obtained using *M/A* compared to the stickiness ratio (λ).

Several factors could influence the test results, including the mass of sample and the beater size and geometry. A sensitivity analysis was conducted to validate the results and check whether M/A was independent of these factors. For sample mass, the test was repeated for four different masses (0.5 kg, 1 kg, 1.5 kg and 2 kg) using mixture $M_{50}$ (50% bentonite and 50% kaolin). For the lowest sample mass (0.5 kg) about one-third of the beater was in contact with the sample, and for the highest mass tested (2 kg), the beater was completely covered by the sample. Based on the results (not shown), it was determined that *M/A* was less sensitive to sample mass than the stickiness index for consistency indices above 0.4. A similar sensitivity analysis for beater size indicated that the mass-to-surface-area ratio showed less variation than the stickiness index for a series of mixing tests conducted with different beater sizes.

## CONCLUSIONS

Based on these results, it was demonstrated that a simple mixing test can be used to indicate the clogging potential by analyzing the ratio of mass of sample remaining on the beater to surface area (*M/A*). The index *M/A* allowed the clogging potential to be assessed quantitatively and gave more reliable results than the stickiness ratio. The mixing test based on the proposed



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# NEW INDEX FOR DETERMINATION OF CLOGGING POTENTIAL BASED ON A MIXING TEST

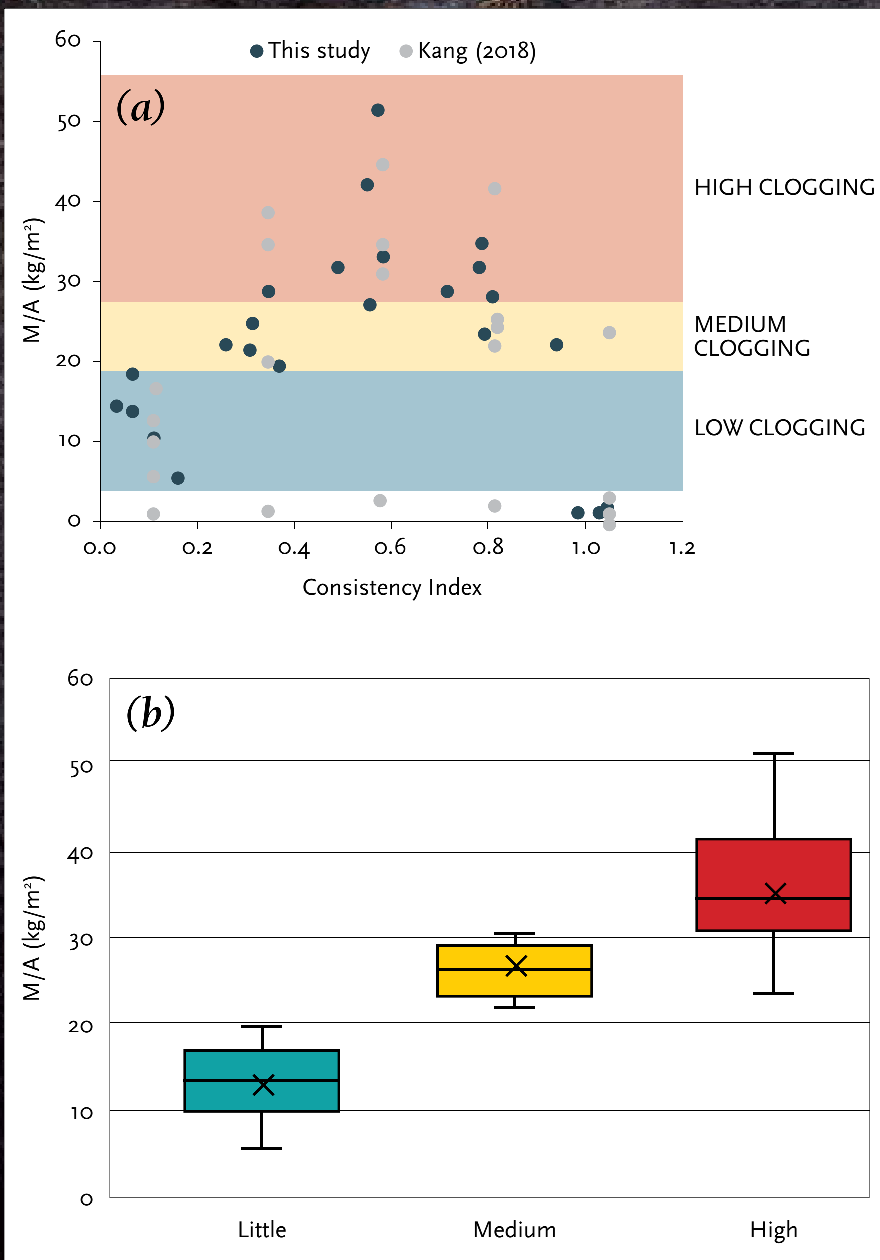


*Figure 2.* ***(a)*** *Classification of clogging potential based on mass-to-surface-area ratio. Green indicates little clogging, yellow indicates medium clogging, and red indicates high clogging.* ***(b)*** *Statistical analysis of the same data, with x indicating the median, and the line indicating the average.*

*Figure 2(b) shows the statistical distribution of M/A across the various samples (for two data sets). Based on these results, samples with values of M/A below 20 have little clogging potential. Values of M/A from 20 to 30 indicate medium clogging. A value of M/A above 30 indicates high clogging.*

index (*M/A*) has several advantages over earlier methods proposed for determining clogging potential. First, it is quicker and simpler to perform this procedure than to determine Atterberg limits and water content of soil samples (as required to use the qualitative diagram proposed by Hollman and Thewes [2013]). Furthermore, the apparatus used in the testing is easily portable and the testing can be done on site. The results of this proposed method can be used to complement geotechnical tests conducted in advance of tunnelling projects and mitigate clogging risks.

## ACKNOWLEDGEMENTS


This article gives an overview of research conducted by Yang Zhou, MSc and Chao Kang, PhD (University of Northern British Columbia). Thank you to Lana Gutwin of the Consortium of Engineered Trenchless Technologies (CETT) at the University of Alberta for her assistance in the preparation of this article.


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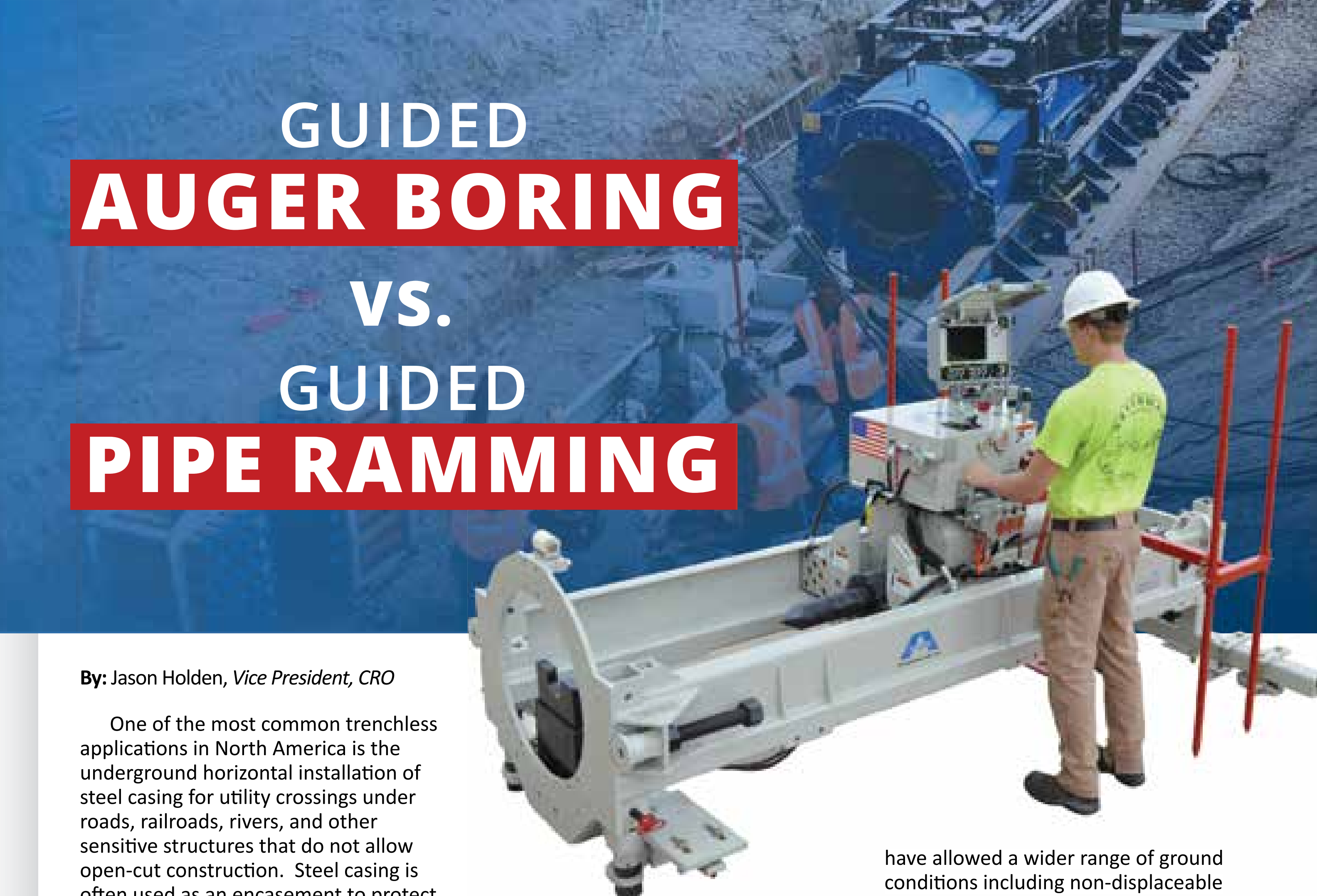
# GUIDED AUGER BORING vs. GUIDED PIPE RAMMING



By: Jason Holden, *Vice President, CRO*

One of the most common trenchless applications in North America is the underground horizontal installation of steel casing for utility crossings under roads, railroads, rivers, and other sensitive structures that do not allow open-cut construction. Steel casing is often used as an encasement to protect the product pipe from damage while supporting the bearing loads applied from the earth and structures located above the ground. Several trenchless techniques exist to install steel casing, however the most widely used and often most economical methods to accurately install steel casing in North America has been by Guided Auger Boring (GAB) and Guided Pipe Ramming (GPR).

Guided Auger Boring and Guided Pipe Ramming are two very similar trenchless techniques but are uniquely different in a way that can provide a contractor a distinct advantage in certain conditions. Understanding when to apply these techniques and the tooling variations available may help reduce future project risks, reduce overall costs, and increase productivity.

*Figure 1: Total Trenchless of Calgary, AB installing pilot tubes for a guided auger boring project.*

Akkerman GBM systems provide critical line and grade accuracy for both auger boring and pipe ramming by installing a dual wall pilot tube from the launch shaft to the reception shaft. Using a remote-controlled, theodolite guidance system with optional data recording, the operator installs pilot tubes on the desired line and grade while feedback from the GBM frame provides additional geotechnical insight along the alignment. Oftentimes sufficient geotechnical investigations are difficult to acquire under certain crossings, so getting a better assessment with a pilot tube is often highly beneficial prior to starting the casing install.

The installation of pilot tubes has historically been only used in displaceable ground conditions. Developments in recent years have allowed a wider range of ground conditions including non-displaceable ground and soft rock. Typical accuracy of installation is +/- 1 inch from design grade and +/- 3 inches from the design alignment at any location with proper equipment set-up. Hybrid setups are also being implemented with pneumatic hammer steering systems with both conventional and sonde guidance system steering heads for applications allowing slightly higher tolerances.

Guided Auger Boring, or also commonly referred to as Pilot Tube Guided Auger Boring (PTGAB) combines the pinpoint accuracy of the pilot tube installation to control line and grade with the excavation process of horizontal auger boring.

During the Guided Auger Boring process, the excavation is done by either a Weld-On Reaming Head or rotating cutterhead that connects to the pilot tubes via a bearing swivel. Excavated material is removed by rotating augers as the steel casing is advanced by thrust forces applied from the auger boring machine located in the launch shaft. With a typical capacity to remove rock and cobble up to 30% of the casing diameter due to auger flight restriction, the final diameter of the steel casing should be determined after considering the geotechnical conditions along the alignment.

Guided Auger Boring allows several types of cutter head options based on the ground conditions that are anticipated. For moderately displaceable ground conditions, which are typically viewed as having a blow count or N-value of less than 30 blows per ft., Akkerman suggests using a standard Weld-On Reaming Head (WORH) to transition from the pilot tube to the steel casing. Shown in the figure below, the WORH allows the contractor to insert a lead auger into the casing for spoil removal. As the casing is advanced, the rotating auger conveys the material to the reception shaft. An advantage of the WORH is that the contractor can easily trip back the augers in an event there is an obstruction as the connection to the pilot tubes is rigid through the steel casing. If the ground conditions appear to ravel or flow, the contractor also has the ability to remove a section of auger. This creates a natural plug in the lead section of casing prior to the excavation process and promotes a stable face.

*Figure 2: Akkerman Weld-On Reaming Heads (WORH)*

In most applications, casing diameters up to 36 inches can be installed by connecting directly behind the pilot tubes. For larger installations, it is recommended to incrementally step-up the size of the casing to provide adequate bearing support to promote an accurate alignment.

Guided Pipe Ramming combines the line and grade accuracy of the pilot tube installation with the power of a pipe ram affixed to the rear of the casing to provide pulsing thrust loads.

During the Guided Pipe Ramming process, the steel casing is attached to the pilot tube string with a special Guided Ramming Head. Appearing very similar to the Weld-On Reaming Head (WORH) shown in Figure 2, the ramming head is purposely built to withstand the extreme loads applied through the pneumatic hammer.

Guided Ramming Heads are manufactured with the same attention to detail required for alignment, however, different steel properties are used, and they include both external and internal relief bands with a more tapered arm design promoting lower thrust requirements.

*Figure 3: Guided Ramming Head (GRH)*

Guided Pipe Ramming has been becoming an increasingly popular trenchless method as it is effective in a wide variety of ground conditions proven difficult with auger boring such as gravely, boulder-laden, and unstable ground. This method was popularized as a back-up in the event Guided Auger Boring projects had encountered difficult ground conditions or obstructions along the alignment. In situations where manned entry was not permitted for obstruction removal or a “911 shaft” could not be constructed, contractors often employed the force of a pneumatic hammer to “thump” the casing across while allowing it to swallow the obstruction. Over time the success of this method became more visible and acceptance in the engineering community now shows it as a preferred technique in several applications.

During the Guided Pipe Ramming Method, the steel casing is advanced by the pipe ram and follows the intended alignment provided by the installed pilot tubes on line and grade. As the steel casing advances and is displaced into the ground, the spoil is forced into the casing pipe. The spoils are not removed from the inside of the casing until the end of the bore or excessive thrust loads are being realized. Cobbles and boulders that may have been too large for removal via augers are potentially swallowed by the casing or displaced to the outside of the pipe, while the spoil inside the casing acts as a plug for unstable ground conditions. Removal of the spoils is typically done with augers or slurry with no risk of over excavation since an encasement pipe is fully inserted along the entire alignment.

The Akkerman Model 240A GBM system is the perfect compliment for both Guided Auger Boring and Guided Pipe Ramming. Contractors now have the option to use the 240A system on any manufacturer’s auger boring rig with the adjustable base frame and remote power pack or use directly with a MBM auger boring rig that features the GBM quick connect package from Michael Byrne Manufacturing. To discover the versatility of Akkerman’s guided boring products, visit our website, or contact one of Akkerman’s knowledgeable sales representatives for more information.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NASTT 2021 No-Dig Show
Orlando, FL

CASE STUDY:

# THE DESIGNED AND CONSTRUCTED MICROTUNNEL FOR THE DOWNTOWN CALGARY TRANSMISSION REINFORCEMENT PROJECT

Samuel Wilson, P.Eng, CCI Inc., Edmonton, Alberta
Thomas Husch, CET, CCI Inc., Edmonton, Alberta

## ABSTRACT

As part of the ENMAX Power Downtown Calgary Transmission Reinforcement Project (DCTRP), ENMAX Power Corporation (ENMAX) constructed a 138 kv electrical transmission powerline between two existing substations within downtown Calgary, Alberta. Some portions of this route were designed with a trenchless methodology to reduce the impact of construction activities within the downtown core and other sensitive areas. In all, 1,517 m (4,977) of microtunnel was designed for this project.

The most critical section of the DCTRP was the construction of this powerline to be completed through a highly congested corridor of the downtown core in Calgary. To avoid the obvious impacts to public and private transportation that conventional open-trench excavation would impose, CCI designed this critical section with a complex microtunnel primarily under 9th Street.

The unique nature of this product, complex design, congested logistics, length, and difficult ground conditions has likely expanded the capabilities of microtunnelling and possible project scopes in the industry. This article looks at the challenges faced both during the design and construction phases of this project.

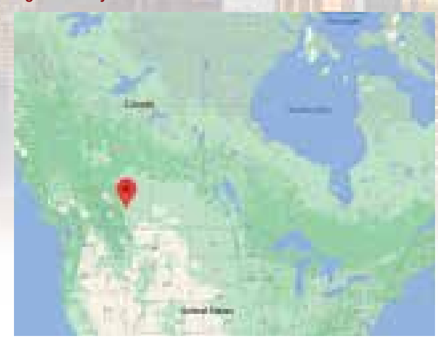
## PROJECT BACKGROUND

CCI was initially contracted by ENMAX, a regional electrical utility to assist in feasibility assessments on general routing options for the project as well as trenchless methodology options to develop trenchless designs at critical points along the route through Calgary, a major city of more than one million people in the western Canadian province of Alberta. This project involves the construction of a 138 kV single circuit transmission line to meet the increased power demands of downtown Calgary.

Continuing after the initial feasibility assessments, the CCI scope was increased to develop "Issued for Review" 90% microtunnel design drawings, to be used as required for permit applications and to be provided in the bid packages for the construction work. Alternative Direct Pipe Installation (DPI) for the Blackfoot Trail/Canada Pacific (CP) Rail crossing, and Guided Auger Bore crossing designs for the Elbow River were also developed at this phase of the project for the Owner to adequately evaluate the economics of these alternate crossing options in the marketplace. The successful contractor was expected to finalize the most feasible designs for each installation based on their construction execution plan.

The successful contractor that was awarded the construction work for the DCTRP was Robert B. Somerville Co. Limited (Somerville). Somerville's subcontractor selected to complete the microtunnel construction was Ward & Burke Microtunnelling Ltd. (Ward & Burke). CCI was then further engaged when subcontracted by Somerville to finalize the microtunnel designs based on Ward & Burke's execution plan and construction practices input, under Somerville's project requirements. CCI's scope was to provide engineering support throughout construction and develop final as-built drawings.

*Figure 1. Project Location*



The 9th Street microtunnel was divided into two S-bend drives under 9th Street, in a north and south alignment with the proposed common launch point in a parking lot between 9th Ave and CP Rail right-of-way. Both microtunnel alignments passed under densely packed urban traffic roads, freight rail, light passenger rail, pedestrian crossings, and hundreds of infrastructure crossings. Receiving pits were targeted near 5th Ave to the north, and 14th Ave to the south. The nominal depth of approximately 10 m was chosen for the 1,540 mm (~60") reinforced concrete pipe under 9th Street. The north tunnel length was 468 m (1,535'), while the south tunnel length was 510 m (1,673') for a total of 978 m (3,208') from the single common launch pit. Both microtunnels required complex S-bend designs and were constructed primarily through mudstone/siltstone, then gravel with silt and sand as the tunnel transitions up towards each shallower exit pit.

Additional challenging microtunnel crossing designs were also developed as part of this project, under the Elbow River, MacLeod Trail, and Blackfoot Trail/CP Rail but are not the main subject of this paper.

## PROJECT ROUTING

CCI assisted with completing a routing assessment for the entire DCTRP project throughout the early stages of the project, providing input on the trenchless design requirements for the various proposed options. The route traversed through downtown Calgary from a power station at 32nd Ave and 10th Street SE to the ENMAX Substation 8 at 4th Ave and 8th Street SW in the downtown core.

The final selected route included five trenchless installations, listed in Table 1 at top right.

The final route is shown in Figure 2, at middle right.

## METHODOLOGIES CONSIDERED

CCI completed a detailed methodology review for each of the identified crossing locations. The methodologies considered included Horizontal Directional Drilling (HDD), direct pipe installation, jack and bore, and microtunneling.

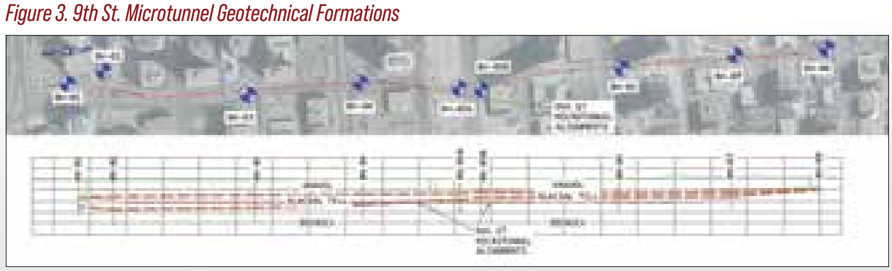
Each crossing site had various workspace restrictions, length, and depth considerations, and varying geotechnical condition considerations. HDD and jack and bore were quickly ruled out for all but the Elbow River crossing due to the previously noted restrictions. Direct pipe and microtunnelling were progressed through the design phase of the Blackfoot Trail/CP Rail crossing before ultimately selecting microtunnelling as the methodology which would be utilized for all of the project crossings.

*Table 1. DCTRP Trenchless Crossings*

| CROSSING | LENGTH |
|---|---|
| 9TH ST. NORTH DRIVE | 468 M (1,535') |
| 9TH ST. SOUTH DRIVE | 510 M (1,673') |
| ELBOW RIVER CROSSING AT THE 1ST ST. SE BRIDGE | 96 M (315') |
| MACLEOD TRAIL CROSSING | 135 M (443') |
| CARTMOUTH RD., CP RAILWAY, AND BLACKFOOT TR. CROSSING | 308 M (1,010') |

*Figure 2. DCTRP Route*


*Figure 3. 9th St. Microtunnel Geotechnical Formations*



**"THE UNIQUE NATURE OF THIS PRODUCT, COMPLEX DESIGN, CONGESTED LOGISTICS, LENGTH, AND DIFFICULT GROUND CONDITIONS HAS LIKELY EXPANDED THE CAPABILITIES OF MICROTUNNELLING AND POSSIBLE PROJECT SCOPES IN THE INDUSTRY."**



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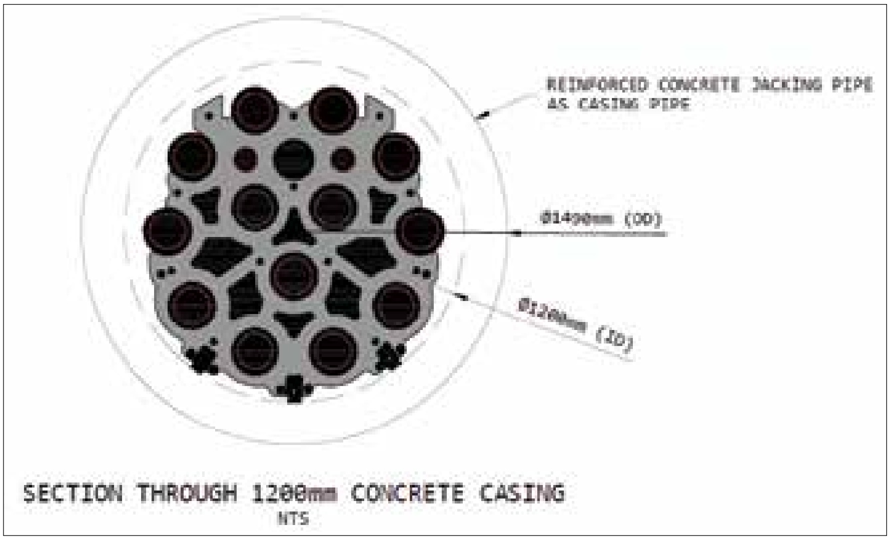
*Figure 4. 9th St. Microtunnel Cross Section*




## GEOTECHNICAL INVESTIGATION

A geotechnical investigation carried out by Tetra Tech Canada Inc. was completed for the project, which investigated and compiled borehole logs of the expected formations to be encountered during construction. For the 9th Street microtunnel, comprised of a North and a South microtunnel drive, a total of nine boreholes were drilled along the anticipated route.

Gravel, sand, and clay fill materials were encountered in all boreholes beneath the asphalt pavement.

The glaciofluvial sands and gravels were encountered at depths ranging from 0.3 m to 7.4 m, typically overlying glacial till deposits. Gravels were typically sandy, poorly graded, with trace to some cobbles with potential boulders. Sand deposits were encountered in two boreholes and were poorly graded to compact.

Glacial till deposits comprised of clay, silt, and sand were encountered in every borehole below granular deposit except for BH-08. The majority of glacial till deposits encountered were clay till, which was generally described as silty, some sand to sandy, trace to some gravel, damp to moist, low to medium plastic with varying N values. The silt and sand till encountered was described with trace gravel, non-plastic, and compact to very dense.

Bedrock consisting of primarily mudstone and siltstone, with occasional sandstone layers was encountered in all boreholes ranging in depth from 5.5 m to 13.7 m, with the deepest bedrock in the northernmost borehole BH-01 and the shallowest in BH-08. Bedrock strength varied with depth and bedrock type, with the shallow mudstone bedrock having estimated unconfined compressive strength up to 2.0 MPa within the upper 3 m with higher strength of 8 MPa expected in the siltstone and sandstone.

Slotted PVC groundwater monitoring wells were installed in all boreholes. Groundwater was observed in all boreholes except BH-08 and was measured during drilling at depths ranging from approximately 3.1 m to 12.2 m. It was ascertained that groundwater was at the elevation where granular materials were encountered. Groundwater in this area is hydraulically connected to the Bow River, which is located approximately 150 m northwest of BH-01 with a surface approximately 10 m below the street surface.

## 9th STREET ENGINEERING

For the 9th Street microtunnel drives, the specified minimum inside diameter for the microtunnel jacking pipe was 1,200 mm and were specified to be made of reinforced concrete with a maximum outside tunnel diameter of 1,524 mm. The final tunnel cross section is shown in Figure 4, including the arrangement of the power cables to be installed once the trenchless installation was completed.

The final tunnel cross section exceeded the requirements for the 138 kV transmission line but allowed for future installations to be pulled through.

**Alignment Details**

Multiple alignments were considered to reach substation 8. The most favourable feasible routing was to traverse north along 9th Street SW. The route minimized the overall construction within the downtown core and provided a direct path to Substation 8 from the residential area at 9th Street and 14th Ave SW to the final destination. The identification of viable workspaces for the location of launch and receiving shafts required for microtunnel construction contributed to the feasibility of the route.

The microtunnel design underneath 9th Street SW was split up into two separate drives, both starting at a shared launch shaft located on the south side of the intersection of 9th Street and 9th Ave SW, on the north side of the CP Rail tracks.

The design paths for both drives included crossings of multiple infrastructure elements, including roads, railroads, light rail tracks, buried power and telecom cables, sanitary sewer, storm sewer, and water lines. Additionally, pile foundations for LRT structures had to be considered in the final design. Multiple daylighting and survey programs were conducted to determine the location and elevations of critical infrastructure.

The LRT structure pile foundation data was not available in the initial data set during early design work. LRT structures included signal poles, catenary poles, and service boxes. Early construction surveys identified these structures and records were found regarding the as-built depths of each pile. Slight design profile modifications were required on the north drive to maintain a minimum clearance of 3 m from the piles during the final design stage.

To facilitate crossing agreements with the LRT authority all pile structures were surveyed to determine their top of foundation elevation, then the as-built information for the depth of each was used to determine proximity to the microtunnel path. Specialized "Issued for Permit" drawings were drafted that showed the proximity to each structure to facilitate crossing agreements. To complement the clearances to the underground pile structures shown on the drawings, the survey elevations and calculated bottom of foundation elevations were included as well.

The depth of each of the drives was designed to provide adequate clearance from other underground infrastructure and to maintain the tunnel paths within either the glacial till material or the bedrock formation. Where possible, horizontal and vertical curves in the tunnel path were kept nearer the receiving shafts to minimize stress on the casing pipe joints. This was possible for the vertical curves in both drives; however, this was not possible for the horizontal alignment of the tunnel path, and horizontal curves along the alignment of both drives had to be implemented to maintain the tunnel path within the allowable utility corridor of 9th Street and to stay out of any private property for both the launch shaft and tunnel path, except for where rights-of-way encroachments had been granted.

In addition to the amount of space required for equipment and construction activities, shaft design also included consideration for placement of permanent manholes to access the constructed power utility. Manhole design was completed by other parties and planned for placement of pre-cast concrete manholes within the construction shafts before backfilling the excavation.

## 9th ST. NORTH MICROTUNNEL DESIGN

The north drive was launched from horizontal at approximately 12 m depth below ground surface for 414 m. An “S” bend curve was included within the horizontal tangent to navigate down the roadway and not intercept any buildings or utilities. The curves were designed at a 400 m radius for a length of 99 m and 98 m. The final concrete pipe selected by the tunnelling contractor used specially designed rubber gaskets at the pipe joints to allow for articulation of the pipe along the radii specified on the drill path while withstanding the expected jacking forces.

At 414 m measured distance from the launch shaft, a vertical curve was included in the design profile at a 308 m radius for a 27 m length into a 5-degree tangent that extended to the receiving shaft. This final portion of the design had been modified prior to construction to provide the required clearance to the pile foundation for the LRT structures. Design modifications included:

- Decrease in vertical curve radius from 400 m to 308 m
- Increase in angle of the final exit tangent from 2 degrees to 5 degrees
- Change in location of the vertical curve from 311 m from launch to 414 m from launch
- Increase of receiving shaft depth from 5.8 m to 8.7 m
- Modification of the alignment, including the lengths and deflections of the horizontal curves to accommodate the relocation of the vertical curve without creating the need for a compound curve

The final designed slack length of the north drive was 468 m.

## 9th ST. SOUTH MICROTUNNEL DESIGN

The south drive launched from the shared launch shaft at a 0-degree angle and crossed beneath the CP Rail tracks along a straight alignment crossing through the private property on either side of the tracks. A 3 m wide right-of-way was obtained to install the microtunnel within the private property. The right of way on both properties was contained within the portion of each property used as a parking lot. Another “S” curve was required immediately after crossing the tracks to bring the bore path in alignment with 9th Street on the south side of the tracks. Additionally, the alignment had to avoid the private property and building foundations immediately south of the tracks across 10th Ave. The horizontal curves were

*Figure 5. 9th St. North and South Drive Design Alignment and Profile*

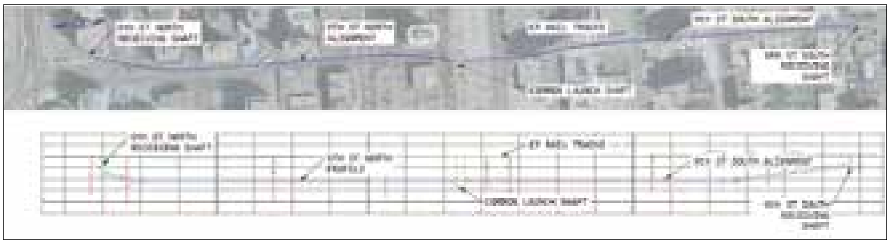


*Table 2. 9th St. North Drive Jacking Force*

| DYNAMIC | W/ SAFETY FACTOR (1.5X) | STATIC | W/ SAFETY FACTOR (1.5X) |
|---|---|---|---|
| 1,615,938 LBS | **2,423,908** LBS | 2,379,576 LBS | **3,569,363** LBS |

*Table 3. 9th St. South Drive Jacking Force*

| DYNAMIC | W/ SAFETY FACTOR (1.5X) | STATIC | W/ SAFETY FACTOR (1.5X) |
|---|---|---|---|
| 1,759,888 LBS | **2,639,831** LBS | 2,450,199 LBS | **3,675,298** LBS |

designed with 400 m radii and were 49 m and 44 m long. The vertical curve was incorporated 400 m into the tunnel path design at a radius of 300 m into a final tangent at 2.5 degrees.

Two design changes were required immediately prior to and during construction. The design change prior to construction was to maintain clearance from a surveyed manhole location south of the launch shaft as the tunnel path crossed 10th Ave SW and to provide additional clearance to the daylighted locations of the storm sewer line near the receiving shaft north of 14th Ave. The design change during construction was based on the tunnelling subcontractors request to maintain the tunnel path within the bedrock for as long as possible and was based on the observed formation during the construction of the receiving shaft north of 14th Ave.

Design modification prior to construction due to the surveyed location of the manhole and storm sewer included:

- Shift in horizontal alignment, including minor changes in the horizontal curve geometry and a shift 0.5m west of the straight portion of the alignment and the receiving shaft within 9th Ave

Design changes during construction to maintain the tunnel path within the bedrock included:

- Decrease in vertical curve radius from 400 m to 300 m
- Change in location of the vertical curve from 343 m from launch to 400 m from launch
- Increase of receiving shaft depth from 6.8 m to 8.8 m
- Increase in launch elevation by 0.35 m

## JACKING FORCE CALCULATIONS

Industry standard calculations were used to calculate the jacking force for the jacking of the pipe and Microtunnel Boring Machine (MTBM). The jacking force calculation incorporates the following:

- The front force at the cutting head of the microtunnelling machine that is required to excavate the material is a combination of the support pressure (derived from the depth of the tunnelling machine, depth of water table and soil properties) and the mechanical force required to turn the cutter head.
- The friction between the pipeline and lubricant fluid which is calculated from the circumference of the pipe, length of pipe within the borehole, and the friction coefficient between the pipe and fluid.
- The friction between the pipeline and the borehole wall which is calculated by multiplying the effective weight of the section (depends on buoyant condition of the pipe sections), coefficient of friction, and the length of the section in the borehole.
- The tunnel path is divided into a sequence of straight and curved sections. At each section, the frictional forces between the pipeline and the borehole, and the pipeline and the lubricant fluid from the previous section (if applicable), is added to the value for the current section. Straight and curved sections can be seen in Figure 5.
- The friction force between the soil and pipeline will increase in the curved sections if the effective weight of the pipeline (due to buoyancy) is lower. The total built up friction force at the end of the curve is calculated based on the friction force at the beginning of the curve.

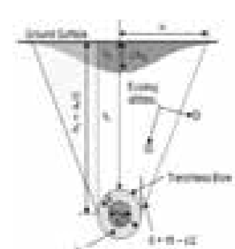
*Figure 6. Settlement Calculations*

**Input Parameters**

| | | |
|---|---|---|
| Dia. of Pipe, $d_p$ | 1424 mm | 56.063 in. |
| Bore hole size (inch.) $d_b$ | 1524.0 mm | 60.000 in. |

| Factors Affecting Annular Volume Translation | $V_a$ Reduction Factor |
|---|---|
| Soil Mass Loosening | 0.87 |
| Arching (κ) | 0.59 |
| Lubricant Replacing Removed Soil | 0.65 |

Total Factor: 0.33 *(typically 0.3 - 0.5)* Ref. 3)



| ID # | Feature Classification | Horizontal Distance from Entry (m) | Depth Of Borehole from Facility ($h_c$) | Friction Angle (ϕ) | Volume of Annulus $V_a$ ($m^3/m$) | Adjusted Volume Of Annulus $V_a$ ($m^3/m$) | Soil Mass Loosening | Arching | Fluid | ΔhCL (m) | Trough width (w) (m) | *Settlement (mm)* |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | Railroad | 25 m | 10.4 m | 30 | 0.2314 $m^3/m$ | 0.0773 $m^3/m$ | YES | YES | YES | 0.01 m | 7.21 m | 10.7 mm |
| 2 | Road | 200 m | 10 m | 30 | 0.2314 $m^3/m$ | 0.0773 $m^3/m$ | YES | YES | YES | 0.01 m | 6.98 m | 11.1 mm |

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Jacking force calculations were repeated for all tangents and curved sections to reach the maximum expected values listed. Theoretical jacking force calculations for design did not consider use of intermediate jacking stations.

Construction included the utilization of internal jacking stations (IJS) to distribute and reduce the total required jacking force that was applied from the launch point during construction of each installation.

**Crossing Permit Support**

To facilitate the approval of third party crossing permits, CCI provided application support to the Owner in the form of application submissions, Issued for Permit design drawings, technical clarification support, and compliance reviews.

In particular, the crossing of the CP Rail right-of-way to the south required a detailed application and review process. Settlement calculations were assessed at the maximum allowable overcut gap (see Figure 6).



Considering a borehole size of 1,524 mm, the minimum OD of casing pipe suitable is 1,424 mm, which equates to approximately 50 mm of allowable radial overcut. The resulting expected settlements under road and rail are within acceptable tolerances at this overcut gap.

On the north drive, crossing beneath the LRT required significant technical coordination with the rail operations group to ensure tunnelling activities would not have any impact to the rail or public. CCI provided additional permit drawings including proximities to all below ground LRT structures to support the crossing permit application.

The entry shaft was constructed within a City-owned parking lot approved for project use. However, private lot boundaries adjacent to this entry location restricted options for optimized location and orientation of the north and south drives and required further assessment of proximity to building foundations and potential impact of the microtunnels. Furthermore, the numerous buried utility crossings seen within these crossings added to the various crossing agreement requirements and reviews completed as part of the design process.

## CONSTRUCTION

The combined construction team, including Somerville, Warde and Burke, and CCI completed the final IFC package for the proposed project prior to execution.

**Launch and Receiving Shafts**

Ward and Burke completed the engineering for the temporary concrete caisson shafts required for construction. The wall sections of the shaft are each constructed at ground level. The walls of the shaft were constructed from reinforced cast in place concrete that provide a rigid structure that will resist all the lateral earth loads applied.

Steel sheet piles were not selected for the construction of the shafts primarily based on two considerations. The first is the high impact and vibration caused by construction that could affect nearby structures and cause further disturbance in the congested area of construction. The second consideration was the necessity to cut a hole in the metal through which to launch the MTBM, leaving the ground temporarily unsupported with the potential for soil and/or groundwater to flow into the shaft.



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The structure sinks under its own weight on the bottom cutting edge as the ground inside the shaft is excavated in and even manner. Once the walls are at the correct depth, the concrete floor is poured. Once the floor has been set, any groundwater within the shaft is pumped out. This method of caisson shaft construction eliminated the need for continuous dewatering as the structure was completely sealed once the concrete wall base is placed.

A launch seal is bolted to a flat formed concrete face on the shaft wall before the MTBM launches. The MTBM advances through the seal and tunnels through the concrete wall that has been left free of any reinforcement then progresses into the soil or rock formation. This methodology ensured that groundwater ingress into the shaft was prevented during the launch of the MTBM and allowed the contractor to support the annular space between the overcut and the Reinforced Concrete Pipe (RCP) using the bentonite lubrication fluid. This in turn reduced frictional forces and helps to mitigate settlement on surface due to soil collapse around the pipe.

Final shaft depths were:

- Launch Shaft = 13.7 m
- North Receiving Shaft = 8.7 m
- South Receiving Shaft = 9.0 m

The depths included the base slabs which for the receiving shafts, were a 600 mm concrete slab and for the launch shaft a 1,000 mm-thick base slab with an additional 500 mm reinforced concrete slab above it. The first slab provides the initial seal of the bottom of the shaft, allowing any water to be removed from the shaft. The second base layer is poured in the dry base of the shaft to be used as the work surface.

The launch shaft outer dimensions were 9.8 m long and 7.2 m wide, with inner dimension of 8.2m long and 6.0 m wide.

The receiving shaft outer dimensions were an outer diameter of 5.2m (4.8 m radii with 0.4 m infills) and an inner diameter of 4.0 m.

Challenges that arose during construction were primarily related to the actual daylighted and surveyed location of the storm sewer compared to the as built location which the design was based on. The south shaft of the 9th Street south drive was required to shift 0.5 m to the west to maintain a minimum of 1.0 m from the storm sewer. An underground telecom cable was also present within the planned location of this receiving shaft and was moved for construction with approval from the utility owner.

*Figure 7. Reinforced Concrete Pipe*

**Microtunnel Construction**

*Pipe Design*

The contractor selected reinforced precast concrete pipe (Class V) with an outside diameter of 1,490 mm and the prescribed minimum inside diameter of 1,200 mm, which was suitable to satisfy the intent of the design. Joints of the casing pipe contained a rubber gasket protected by a circumferential steel band collar. The gasket was compressed with each joint of pipe during installation and was designed to withstand a hydrostatic head of over 30 m. The selected pipe also allowed for installation of lubrication ports, which were placed at regular intervals along the drive length, allowing for the injection of bentonite lubrication to minimize the jacking forces required

*Microtunnel Boring Machine (MTBM)*

An AVN1200 Herrenknecht Microtunnelling Boring Machine was selected to be used with a mixed face cutter head. The cutterhead was selected for the robust structure and was specifically designed by Herrenknecht for types of soil conditions present along the designed tunnel path. The MTBM created a 25 mm radial overcut around the casing pipe which was supported by the bentonite lubricant during the tunnelling process and intended to maintain the annular space of the tunnel machine to reduce risk of soil settlement. The MTBM cutterhead allowed for pressure to be maintained at the tunnel face, ensuring the stability of the tunnel face throughout the excavation cycle. The MTBM system included sensors for real-time monitoring of multiple tunnelling parameters such as slurry pressure.

*Figure 8. Jacking Frame in Launch Shaft*

The pressure and viscosity of the lubricating fluid pumped into the overcut had to be sufficient to ensure that the surrounding soils did not collapse on the casing pipe. If cohesionless soils were encountered, it was expected that pressures of up to 5–6 bar would be required to support the annulus.

Once the drive was completed, a grout mixture was pumped through the injection ports to maintain ground stability and mitigate the risk of surface settlement.

*Jacking Frame/IJS*

The jacking frame provided by the contractor was of a unique space-saving design and provided a maximum 3 m stroke to install the 3 m long RCP sections.

Intermediate jacking stations were installed at regular intervals and at specific positions within the casing pipe. Placement of these stations considered the geometry of the designed tunnel path to reduce the overall jacking force

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required of the main jacking frame to complete the drive and navigate through the designed tunnel path.

*Construction Summary*
During the excavation of the receiving shaft on the north side of 14th Ave, the granular and glacial till material encountered was less favourable for tunnelling than initially determined based on the geotechnical investigation. The design was modified as described in section 8 to maintain the drive within the bedrock formation for as long as practical. This was to eliminate risks associated with exiting the bedrock at a shallow angle and tunnelling into the undesirable material above it. Primary risks increased by exiting the bedrock included overmining and settlement of the roadway.

Temporary construction tie backs were intercepted during the drive process. These tie backs were initially utilized in the excavation for the construction of the surrounding buildings and were not removed post construction. As-built data was reviewed and due to the curving alignment of the drive, only a handful of tie backs were actually intercepted, causing minimal delays due to tunnelling through the tie backs.

Outside of the design changes described previously, overall construction activities progressed smoothly with minimal delays or incidents, resulting in completion of the tunnelling activities according to schedule.



## CONCLUSION

The ENMAX DCTRP project was designed and constructed successfully utilizing a high degree of care in the engineering design and construction implementation. CCI is proud to have been an instrumental part of this project which was completed with the utmost consideration for safety to the personnel, public, and environment and would like to thank ENMAX, Somerville, and Ward & Burke for their teamwork and professionalism contributing to the overall success of the project.

## REFERENCE MATERIAL

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