2020

TRENCHLESS SANS TRANCHÉE
Journal

THE OFFICIAL PUBLICATION OF THE NORTH AMERICAN SOCIETY FOR TRENCHLESS TECHNOLOGY
Great Lakes, St. Lawrence & Atlantic Chapter | Chapitre des Grands-Lacs, du Saint-Laurent et de l'Atlantique

Shore Approach Horizontal Directional Drill Crossings into the Bay of Fundy

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ALSO INSIDE:
- Rehabilitation of the Leslie Street Sanitary Trunk Sewer & Maintenance Holes

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ON THE COVER: Hopewell Rocks geologigal formations at low tide in the Bay of Fundy, New Brunswick, Canada.

TRENCHLESS SANS TRANCHÉE

Journal

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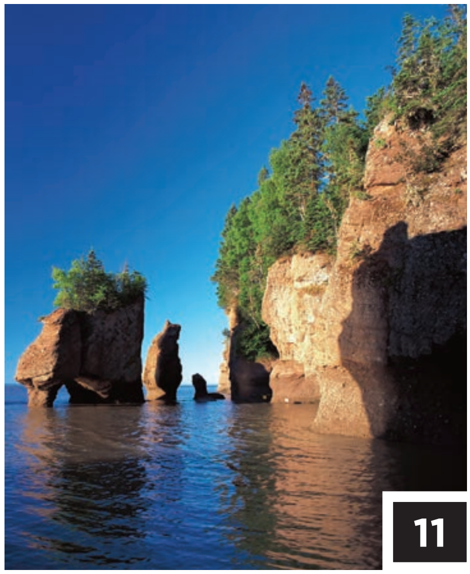

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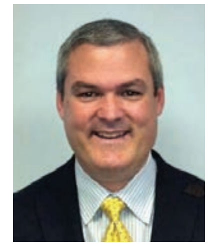
Working to Maintain Trenchless Momentum



Challenging times with unprecedented change have been the mantra for 2020, and through it the trenchless industry has continued to serve the infrastructure needs of communities across the country. The GLSLA Board of Directors and member volunteers are continuing to work to provide value to our members through training, publications, and our website. GLSLA continues to support NASTT initiatives to transition our in-person training courses, seminars, and conferences to the online environment and maintain trenchless momentum in the infrastructure industry. This has included trainers and participation from GLSLA members.

The three Canadian chapters of NASTT (Northwest, GLSLA, and BC) are continuing to plan for the 2021 No-Dig North Show in Vancouver, November 8 to 10. Our call for abstracts is open until March 1, 2021, so get your submissions in soon and share your project experiences with the industry. For more information on the show, please visit *www.nodignorth.ca.*

While we cannot meet in person yet, we hope you enjoy this latest issue of the GLSLA Chapter magazine, filled with great project and industry articles. For more information on GLSLA and our events, magazine, and training sessions, or to publish an article in our magazine, please visit our website at *www.glsla.ca.*

> "The GLSLA Board of Directors and member volunteers are continuing to work to provide value to our members through training, publications, and our website."

Surtout, ne pas perdre l'élan!

Année éprouvante et changements inédits : les maîtres mots de 2020! Mais l'industrie du sans-tranchées continue de répondre aux besoins en infrastructure dans tout le pays. Le conseil d'administration et les bénévoles de la section Great Lakes, St. Lawrence and Atlantic (GLSLA) travaillent sans relâche à valoriser ses prestations, qu'il s'agisse des formations, des publications ou du site Web. La GLSLA contribue toujours aux initiatives de la North American Society for Trenchless Technology (NASTT), qui concernent particulièrement, cette année, la transition entre les cours en personne et les cours en ligne ainsi que le maintien du dynamisme de l'industrie des infrastructures, et qui englobent donc les formateurs et la participation des membres de la GLSLA.

Les trois sections canadiennes de la NASTT (Nord-Ouest, GLSLA et C.-B.) poursuivent la planification du No-Dig North Show, qui aura lieu du 8 au 10 novembre 2021, à Vancouver. Vous avez jusqu'au 1er mars 2021 pour envoyer un résumé de votre présentation. Écrivez-nous vite pour être en mesure de faire connaître votre expérience aux membres de l'industrie. Pour en savoir davantage sur le salon, rendez-vous à l'adresse *www.nodignorth.ca.*

Il est certes impossible de nous réunir en personne, mais nous espérons que vous profiterez de ce nouveau numéro du magazine de la section GLSLA. Il regorge d'articles des plus intéressants sur l'industrie et ses grands chantiers! Pour en savoir davantage sur la section, nos activités, le magazine et les formations, ou pour publier un article dans le magazine, consultez notre site Web, à l'adresse *www.glsla.ca.*

> « Le conseil d'administration et les bénévoles de la section Great Lakes, St. Lawrence and Atlantic (GLSLA) travaillent sans relâche à valoriser ses prestations, qu'il s'agisse des formations, des publications ou du site Web. »

Stronger Than Ever

For everyone, 2020 has been quite a whirlwind year! Like the rest of the world, the staff and volunteers here at NASTT have been pivoting and evolving on a near daily basis to changes in how we do business due to the COVID-19 situation.

As this unprecedented year continues to unfold, NASTT is working diligently to continue to provide the training and education you need to do business and stay up to date with innovations in our industry. We are excited to have rolled out virtual events and training opportunities as we fulfill our mission to be the premier trenchless educational society in North America.

In August we launched our NASTT Good Practices Courses as virtual events. These courses are a rescheduling of the 2020 No-Dig Show Good Practices Courses, and our entire suite of courses will be available as live training events. Our four-hour courses will take place in one day and our eight-hour courses will be split into two-day sections to allow for schedule flexibility for our attendees. All NASTT Good Practices Courses include Continuing Education Units, a training manual, and the accompanying NASTT Good Practices Guidelines book if applicable. Visit *nastt.org/training/events* for the full schedule and registration details.

> "We're going to come roaring back strong and break these records at No-Dig 2021 in Orlando! We look forward to growing and learning from these recent challenges and coming back stronger than ever."

Our goal is to represent our industry and provide valuable initiatives. To do that, we need the involvement and feedback from our members. We are always seeking volunteers for our various committees and programs. If you are interested in more information, please visit our website at *nastt.org/membership/volunteer.* There you can view the committees and learn more about the ways to stay involved with the trenchless community and to have your voice heard. Please consider becoming a volunteer – we would love to tap into your expertise.

We are looking forward to coming together in Orlando next March for the NASTT 2021 No-Dig Show. It will be particularly exciting to come together again as a group and celebrate the trenchless industry in North America as we learn and network together. By all accounts, the NASTT 2019 No-Dig Show in Chicago was a resounding success, hosting a record-breaking 200+ exhibitors and more than 2,200 attendees. We're going to come roaring back strong and break these records at No-Dig 2021 in Orlando!

We look forward to growing and learning from these recent challenges and coming back stronger than ever. Thank you for all your support and dedication to NASTT and the trenchless technology industry. We are only as strong as our regional Chapters. We are always looking for volunteers and new committee members, not only locally but nationally. Don't be afraid to get involved! With the trenchless market growing so fast, now is the time to join us!

Thank you for being a part of our organization and for dedicating your careers to the trenchless industry.

Plus forts que jamais

L'année 2020 a soufflé sur tous comme une énorme tempête. Comme tout le monde, d'ailleurs, le personnel et les bénévoles de la North American Society for Trenchless Technology (NASTT) ont pirouetté pour s'adapter pratiquement au jour le jour aux toutes nouvelles façons de faire dictées par la COVID19.

D'ici la fin de cette année à nulle autre pareille, la NASTT compte bien continuer de travailler avec soin pour vous offrir la formation dont vous avez besoin pour mener vos affaires et vous tenir au fait des progrès de l'industrie. Nous sommes très fiers d'avoir instauré des activités et des formations virtuelles, grâce auxquelles nous poursuivons notre mission : être la première société de formation sur la technologie sans tranchées en Amérique du Nord.

Nous avons inauguré en août la version virtuelle de nos cours sur les pratiques exemplaires, en remplacement de ceux qui devaient avoir lieu dans le cadre du No-Dig Show. Tous nos cours sont maintenant offerts en ligne et en direct. Les cours de quatre heures se donnent en un jour et les cours de huit heures sont répartis sur deux jours pour que les participants puissent adapter leur horaire. Ces cours sur les pratiques exemplaires donnent droit à des crédits de formation continue. Les droits d'inscription comprennent un manuel de formation et un guide des pratiques exemplaires le cas échéant. Vous trouverez le calendrier complet et les détails relatifs à l'inscription sur la page *nastt.org/training/events.*

Notre objectif est de représenter l'industrie et d'offrir des initiatives enrichissantes. Nous ne saurions toutefois y arriver sans la participation et la rétroaction de nos membres. Nous sommes donc toujours à la recherche de bénévoles pour nos divers comités et programmes. Intéressés?

> « Le No-Dig d'Orlando, en 2021, marquera un retour en force et ces records voleront en éclats! »

MARCH 28-APRIL 1 | ORLANDO, FL
NASTT NO-DIG SHOW 2021

Visitez notre site Web, à l'adresse *nastt.org/membership/volunteer.* Vous y découvrirez chacun des comités, la façon de rester connecté à la communauté du sans-tranchées et les moyens de faire entendre votre voix. Pourquoi ne pas devenir bénévoles? Nous aimerions beaucoup tirer parti de votre expertise.

Nous avons très hâte, il va sans dire, au No-Dig Show de mars 2021, à Orlando. Quel plaisir de vous revoir pour célébrer ensemble l'industrie nord-américaine du sans-tranchées, pour apprendre et pour réseauter. Au dire de tous, le No-Dig Show de 2019 a été un succès retentissant : plus de 200 exposants et plus de 2200 participants, c'est un record! Mais le No-Dig d'Orlando, en 2021, marquera un retour en force et ces records voleront en éclats!

Il nous tarde de tirer les leçons de la crise pour croître, apprendre et rebondir, plus forts que jamais. Merci de votre appui et de votre dévouement à la NASTT et à l'industrie de la technologie sans tranchées. Notre force dépend étroitement des sections régionales. La NASTT et les comités ont toujours besoin de bénévoles et de nouveaux membres, en région et à l'échelle pancanadienne. N'hésitez pas à être des nôtres. Le marché croît à grande vitesse et c'est le moment idéal pour vous joindre à nous!

Merci de faire partie de nos membres et de consacrer votre carrière à l'industrie sans tranchées.

BE SURE TO CHECK OUT OUR WEBSITE FOR UPCOMING SESSION DATES

TRENCHLESS TRAINING ASSOCIATION

Trenchless Training Association (TTA) is dedicated to helping move you in the right direction along with the HDD knowledge, skills and training for you to succeed.

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- Environmental Planning & Inspection
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SHORE APPROACH HORIZONTAL DIRECTIONAL DRILL CROSSINGS INTO THE BAY OF FUNDY, NEW BRUNSWICK

Samuel Wilson, P.Eng./CCI Inc., Edmonton, Alberta
Jeff Miller, E.I.T./CCI Inc., Edmonton, Alberta
Adam Van Nood/Marathon Underground, Ottawa, Ontario

ABSTRACT

New Brunswick Power Transmission Corporation (NB Power) is constructing the Fundy Isles Transmission Power Line Project which consists of 69 kV Submarine Cables from Deer Island, NB to Campobello Island, NB and from Campobello Island to Grand Manan Island, NB. This project will provide electricity to the islands and will replace existing cables installed in 1978 and nearing the end of their service life.

NB Power requested that the connection between the submarine and the land-based sections of the project be completed utilizing the Horizontal Directional Drill (HDD) methodology to mitigate any concerns of future exposure due to erosion from the large tidal action within the bay. The HDD crossings were to include the installation of three HDPE conduits, one 16-inch O.D. HDPE conduit along with two spare 3.5-inch HDPE conduits for future corrosion protection measures inside a 24” common borehole. The project included four shore approach crossings, designed to be progressed through 0-50% RQD Mafic Tuft, Shale bedrock, and overburden materials.

Marathon Underground (Marathon) was successful in the tendering process for the project. CCI Inc. (CCI) was subcontracted to assist with production of feasible designs and provide HDD construction experience and support.

The unique nature of the shore approach methodologies, space constraints, marine activities, and difficult ground conditions at the crossing locations culminated in complicated conditions for HDD installations.

Through comprehensive engineering design, significant combined HDD experience, and Marathon’s ability to adapt to onsite conditions, two of the four crossings were successfully completed by HDD methods and two were completed with open cut methodologies with all completion dates meeting the required in-service date.

This article will discuss the challenges faced during both the design and construction phases of the shore approach crossings.

PROJECT BACKGROUND

NB Power’s Fundy Isles 69 kV Submarine Cable Project constructed an electrical transmission line connecting Deer Island, Campobello Island, and Grande Manan Island, NB. The new installation project paralleled the existing transmission line for much of the alignment. The project consisted of three separate aspects: subsea cable installations, land-based cable installations, and the transition from the subsea installations to the land-based installations. The existing cable was installed within a conduit by traditional open cut methodology, backfilled, buried, and protected with rip rap.

As part of the new cable installation, four connection points (shore approaches) were identified in order to connect the subsea installations to the mainland. The connection points were identified as:

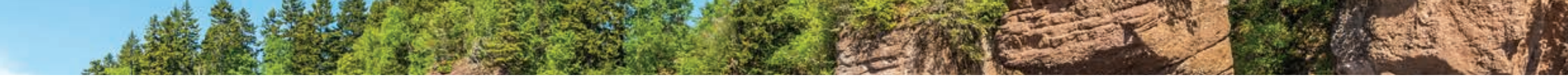
- Chocolate Cove
- Wilson’s Beach
- Little Whale Cove
- Long Eddy Point

Figure 1 shows the proposed crossing locations in relation to the Bay of Fundy. The planned method of installation for the shore approaches was a trenchless solution, Horizontal Directional Drilling (HDD). This methodology was chosen for its ability to provide added depth of cover for the conduit at the shoreline, protecting the conduit from the rapidly changing tide and erosion of the ground formations. The project required a single 16-inch HDPE and two 3.5-inch HDPE conduits to be installed within the same borehole at each shore approach location.

CHOCOLATE COVE ENGINEERING DESIGN AND CONSTRUCTION

SITE DETAILS

Chocolate Cove is located approximately 80 km southwest of Saint John, New Brunswick. NB Power has a substation situated at the top of the slope, approximately 70 m northwest of the shoreline. The top of the slope is approximately 25 m above sea level (low tide, 0 m geodetic). The slope drops 25 m vertically over 100 m horizontally



(approximately 25% slope or 14 degrees), and the tide at the crossing location rises approximately 10 m at its maximum, reaching 10 m, geodetic.

SITE-SPECIFIC GEOTECHNICAL CONDITIONS

A land-based geotechnical investigation was completed for the project by Stantec Consulting Ltd. (Stantec), which investigated and compiled logs of the expected formations which the planned HDD would intersect during construction. For Chocolate Cove, a total of three boreholes (designated as Boreholes CC BH1, CC BH2, and CC BH3) were drilled, located at the top of slope, partially down the slope, and near the bottom of the slope. Borehole locations are shown in Figure 2 and Table 1 summarizes the findings of the borehole program.

Figure 1. Project Location

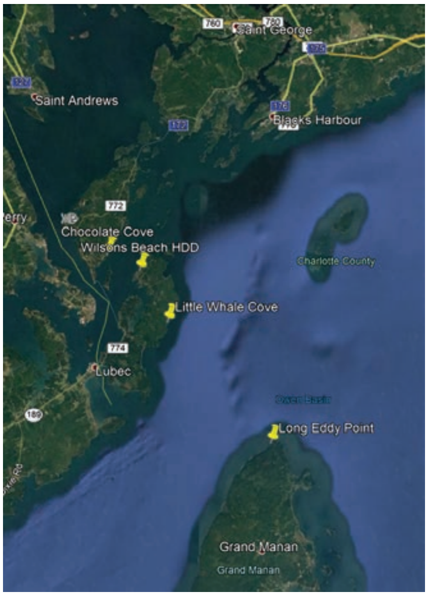


Figure 2. Chocolate Cove Borehole Locations

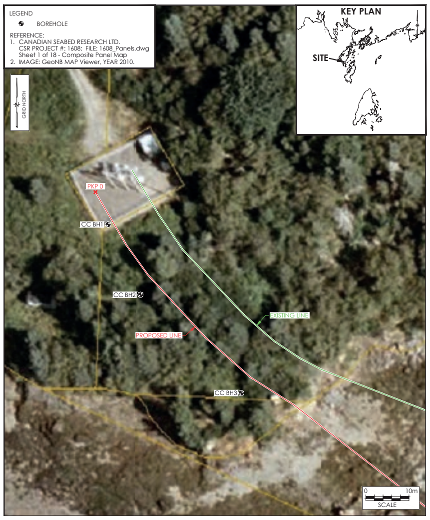


Table 1. Chocolate Cove Borehole Summary

Borehole	Depth (m)	Approximate Location	Description	Primary Geotechnical Concerns
CC BH1	17.98	Top of slope, near entry location	Sand with gravel, poor to good mafic lithic tuff	Unstable borehole, loss of drilling fluid
CC BH2	18.19	Mid slope, 40 m away from entry	Loose sand with gravel, very poor to excellent mafic lithic tuff	Unstable borehole, loss of drilling fluid
CC BH3	17.98	Bottom of slope, 70 m from entry	Topsoil, silty sand with gravel, very poor to excellent mafic lithic tuff	Unstable borehole, loss of drilling fluid

The sand and gravel, identified as compact to dense, were not expected to provide structural borehole support and would require casing. Concerns of borehole collapse while reaming, with the probability of gravel falling down the borehole, led to a recommendation of temporary construction casing. The tuff had a rock quality designation (RQD) that ranged from very poor to good. RQD signifies the degree of jointing or fracturing in a rock mass, measured in percentage. RQD is a determination of number of broken pieces per a set run in the core log. Low RQD values would suggest a material which is already broken or would easily break apart. RQD values ranged between 0% and 93% (at depth).

HDD ALIGNMENT

The designed subsea alignment could not be altered as the maneuverability and placement of the cable is limited when utilizing subsea cable laying equipment based on the profile of the sea floor and plastic bends were not achievable. Due to these reasons, any change in alignment of the HDD was restricted by the subsea alignment.

The HDD alignment was selected to maintain the crossing within the existing NB Power property boundaries and crossing the shore as close to perpendicular as practicable.

Figure 3. Photo of the core run from CC-BH1: 1.3 to 7.3 m showing very poor RQD

Figure 4. Photo of the core run from CC-BH1: 7.3 to 11.4 m showing RQD increasing with depth

The design followed a northwest to southeast alignment, with the entry point situated approximately 120 meters northwest of the low tide shoreline, at the top of the slope, and the exit point approximately 10 meters to the southeast of the low tide shoreline. The straight-line alignment length for the crossing was 127 m. Figure 5 shows the HDD alignment.

HDD DESIGN DETAILS

The entry and exit angles were selected to maximize the depth underneath the surface of the slope, limiting the probability of hydraulic fracture to surface, and aiming to maintain the borehole within drillable geotechnical strata. In addition, site restrictions on the entry location and exit point below water surface limited the alignment of the crossing. The entry angle of 18 degrees was selected as it was the maximum the HDD rig could achieve. This angle also minimized the amount of casing which would be required when compared to a lower entry angle crossing.

Rather than following a traditional HDD profile which includes the drill path reaching a baseline tangent and then curving back to surface, the profile terminated angling downwards at 7 degrees. This was selected to approximate the slope of the bedrock ledge below water surface, while allowing the HDD to maintain depth underneath the ground surface. During pullback the pipe would be floated on the water surface and then sunk to align with the exit and bore path trajectory.

The standard practice in the HDD industry, for steel pipe installations, is to utilize 1,200 times the nominal pipe diameter as the radius of curvature (ROC). For the HDPE product pipes used for this project, the rule of thumb was not applicable. In this case, the radius

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Figure 5. Chocolate Cove Topography along HDD Alignment

of curvature was based on constructability and the steering capabilities of Marathon's equipment. The radius of curvature was selected to be 300 m, which is well within the allowable bending radius of the HDPE pipes (recommended to be a minimum of 50 times the nominal pipe diameter).

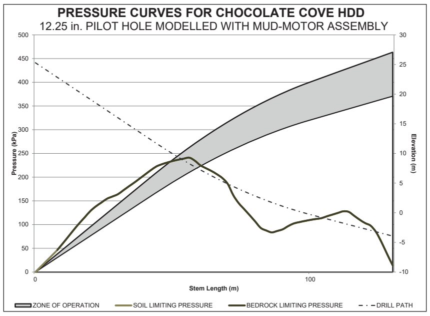
The achievable depth of cover for this installation was limited by the required entry and exit points and the required exit angle. Optimal HDD depth is based on several parameters including geological formation, required overburden pressure to contain the drilling fluid annular pressure, buried facilities in the area, watercourse/roadway geometric parameters, topography, and space limitations. Due to the restrictions outlined, the maximum achievable depth of the crossing was 10.1 m, while the shallowest cover was 3.5 m (apart from the entry and exit locations). The majority of the crossing was situated within 0% to 35% RQD bedrock. The design profile is shown in Figure 6.

ANNULAR PRESSURE ANALYSIS

Annular Pressure (AP) modeling was developed to model the expected drilling pressure required to drill the pilot hole along the proposed Chocolate Cove bore. This pressure has been modeled accurately as confirmed by field values from many HDD installations using pressure monitoring tools. CCI also modeled the potential overburden or confining pressure and used this information to approximate the risk of frac out.

The ability to accurately assess the risk that the expected annular pressure during drilling of the HDD results in a hydraulic fracture that could reach the water body is highly dependent on the homogeneous nature of the formation, level of fracturing in the bedrock, and type/consistency of the overburden. It is also important to note that the information provided by borehole investigations is accurate at that location but may vary significantly some distance away. A vertical borehole may not identify the vertical fractures that can significantly affect AP and fracture potential modeling. This potential inaccuracy is accounted for by utilizing conservatism in the modeling assessment, and by acknowledging the AP pressure model as a process to reduce drilling fluid releases based on the quality of information provided.

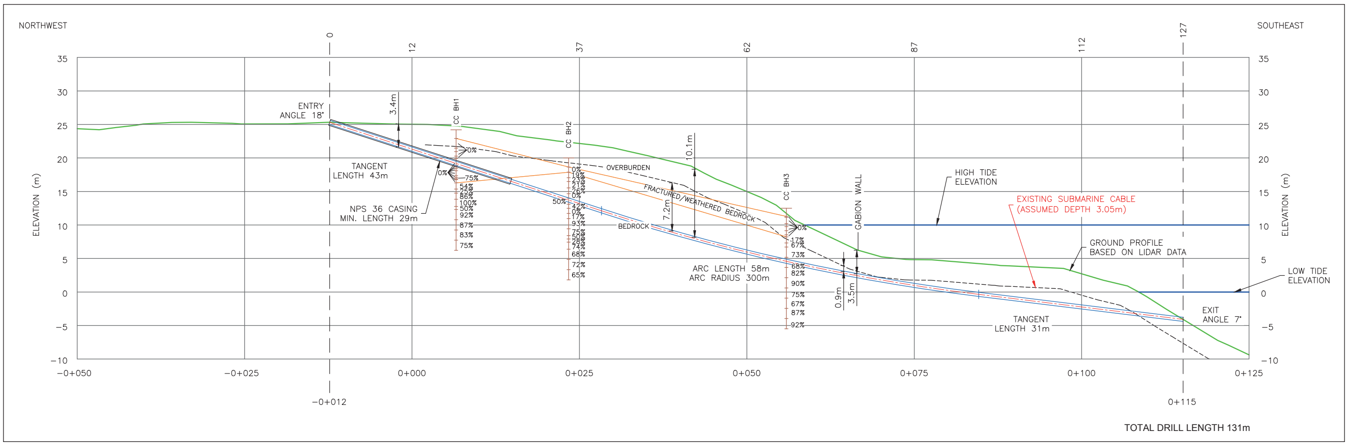
Figure 7. Chocolate Cove Annular Pressure Chart Showing HDD Drilling Pressure Zone of Operation



Similar to any theoretical modeling, the annular pressure analysis is a tool to better understand the probability of hydraulic fracture to surface during HDD. The graphical representation below shows the calculated drilling pressure and an operational window with a 25% increase to show an estimated drilling zone for the contractors. Based on this model, drilling pressures exceeding the fracture pressure don't necessarily indicate a fracture will occur, but rather higher risk that a hydraulic fracture may occur in those areas. The annular pressure chart for the Chocolate Cove HDD is shown in Figure 7.

According to the AP chart for the Chocolate Cove HDD, the expected fracture containment pressure (strength of the formation) was lower than the expected drilling pressures for the majority of the drill path design, indicating a high probability of hydraulic fracture to the surface and waterbody. Due to the limited available workspace, alignment constraints, and required exit point depth target, this risk could not be mitigated any further with design. The regulators' major concern for the whole project was to limit any potential hydraulic fracture to the land segment.

Figure 6. Chocolate Cove HDD Profile



STRESS ANALYSIS

Comprehensive stress analyses were completed for the installation and operation of the 16-inch and 3.5-inch HDPE conduits. These analyses were completed in compliance with ASTM F1962 (Standard Guide for Use of Maxi-Horizontal Directional Drilling for Placement of Polyethylene Pipe or Conduit) and PRCI PR277-144507 (Installation of Pipelines by Horizontal Directional Drilling). The analysis included an evaluation of required pull force, the applied tensile stress, long-term ring deformation, and safety factors against collapse. The results of the stress analysis suggested that the maximum pull force for the bundled installation would be 25,000 lbs. (with safety factor). Buoyancy control was not utilized in the pullback analysis. The safety factor against collapse and tensile stresses were well within the ASTM allowable tolerances.

HDD CONSTRUCTION

The HDD for Chocolate Cove was executed by first installing a 20 m section of NPS 36 steel casing to isolate the sand with gravel and the 0% RQD bedrock. Casing was installed to a depth where the rock quality was suitable for HDD operations.

Pilot hole operations utilized a 12 ¼-inch drill bit with an 8-inch mud motor combination. Steering wire was laid on surface to provide accurate tracking of the downhole bit. Pilot hole operations were stopped approximately 30 m from exiting the ground surface at the bottom of the crossing. At this point, the downhole tooling was tripped back to surface.

A 24-inch reamer was attached to the drill string, with a bullnose piece at the front to center the string within the pilot hole and maintain the alignment of the borehole. The crossing was then forward reamed (push reamed) from entry to exit. This methodology promoted drilling fluid and cuttings circulation back to the entry location. The traditional HDD method of completing the pilot hole in its entirety prior to reaming would have allowed the borehole to drain fluid and cuttings to the Bay of Fundy and would have allowed a constant release of drilling fluid and cuttings to the bay for the entire reaming process. The reaming process was slower than anticipated which lead to delays in the project schedule.

Prior to line pull, the exit location was excavated by an excavator located on a barge to mitigate the risk of cobbles/boulders falling into the hole.

The product pipes were fused and prepared in a protected cove, a short distance away from the HDD location. Once the ream was completed, the pipes were floated and placed into position on the water surface, utilizing tugboats. Divers entered the water to attach a cable to the drill bit to bring the drill string to surface in order to attach to the product pipe. There were very limited safe operational windows available each day for divers and tugs to complete the work due to fast-moving tides and weather conditions. The weather impact led to delays of pipe pullback as the divers and tugs could not safely work in the water. The product pipes were successfully installed within the borehole prior to HDD rig mobilization to the next crossing.

WILSON'S BEACH ENGINEERING DESIGN AND CONSTRUCTION

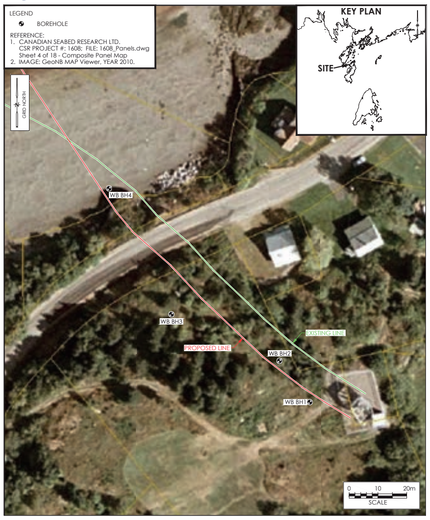
SITE DETAILS

Wilson's Beach is located approximately 3 km east of Chocolate Cove and is separated from Chocolate Cove by the Bay of Fundy. The top of the slope is approximately 28 m above sea level (low tide, 0 m geodetic). The slope of the hill drops 28 m vertically over 125 m horizontally (approximately 22.4% slope or 12.5 degrees). The tide at the crossing location rises approximately 10 m at its maximum, reaching 10 m, geodetic. Approximately 75 m to the south east of the low tide marker, Route 774 runs northeast to southwest.

SITE-SPECIFIC GEOTECHNICAL CONDITION

Stantec completed a borehole program at Wilson's Beach that consisted of four boreholes (designated as Boreholes WB BH1, WB BH2, WB BH3, and WB BH4). Boreholes were drilled approximately every 30 m from the top of slope to the bottom of slope. Borehole locations are shown in Figure 8 and Table 2 summarizes the findings of the borehole program.

Figure 8. Wilson's Beach Borehole Locations



The borehole program identified shallow amounts of overburden materials consisting of topsoil, sands, and some gravel. These formations would be excavated at the time of construction and would not pose a risk.

The RQD values of the shale encountered below the shallow overburden materials ranged from 0% to 75%, at depth. Images of the low RQD samples (near surface) are seen on page 15.

Geotechnical conditions at the exit point were investigated by a marine based approach including divers, electric resistivity tomography, and excvations. The investigation suggested formations were sand with some cobble.

HDD ALIGNMENT

Similarly to Chocolate Cove, the alignment was selected based on maintaining the design within NB Power property boundaries and crossing the shore as close to perpendicular as practicable.

The design followed a southeast to northwest alignment, with the entry point situated approximately 150 meters

Table 2. Wilson's Beach Borehole Summary

Borehole	Depth (m)	Approximate Location	Description	Primary Geotechnical Concerns
WB BH1	10.36	Top of slope, near entry location	Topsoil, very poor to excellent shale	Loss of circulation
WB BH2	10.67	Mid slope, 27 m away from entry	Topsoil, silty sand, very poor to fair shale	Loss of circulation
WB BH3	21.95	Mid slope, 62 m from entry	Topsoil, gravel, very poor to fair shale	Loss of circulation
WB BH4	9.30	Bottom of slope, 110 m from entry	Silty sand, shale	Loss of circulation

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Figure 9. Photo of the core run from WB-BH3: 0 to 3.7 m showing poor RQD

Figure 10. Photo of the core run from WB-BH3: 3.7 to 6.9 m showing increasing RQD with depth

Figure 11. Wilson's Beach Topography along HDD Alignment

southeast the low tide shoreline, at the top of the slope, and the exit point approximately 20 meters to the northwest of the low tide shoreline. The straight-line alignment length for the crossing was 170 m. Figure 11 shows the HDD alignment.

HDD DESIGN DETAILS

Wilson's Beach had site constraints similar to those at Chocolate Cove. Thus, the HDD was similarly designed to maximize the depth underneath the surface of the slope, limit the probability of hydraulic fracture to surface, and to maintain the borehole within drillable geotechnical strata. The entry angle of 18 degrees was selected as it would allow drill path to progress quickly through the lower RQD formations. A horizontal exit was chosen to provide additional depth throughout the bottom of the crossing while maintaining an exit elevation which was feasible for subsurface divers and equipment. The subsea profile was uneven, which provided difficult terrain to lay the subsea portion. In order to find a profile suitable, the alignment required a sharp point of inflection in the alignment to adhere to the subsea cable laying requirements. A 300 m design radius was also utilized at this location.

The achievable depth of cover for this installation was limited by the constrained entry and exit points as well as the required exit angle. Due to the restrictions outlined, the maximum achievable depth underneath the road was 11.3 m. A shallow depth of cover continued throughout the crossing profile. The majority of the crossing was to be completed within 0% to 50% RQD bedrock. The design profile is shown in Figure 12.

ANNULAR PRESSURE ANALYSIS

According to the AP chart for the Wilson's Beach HDD, the expected fracture containment pressure (overburden strength) is lower than the expected drilling pressures for the majority of the drill path design, indicating a high probability of hydraulic fracture to the surface and waterbody. The annular pressure chart for the Wilson's Beach HDD is shown in Figure 13. Similarly to Chocolate Cove, the workspace and alignment constrictions did not allow the risk of fracture to be mitigated any further with design modifications. The regulators' major concern was again to limit any potential hydraulic fracture within the land segment.

STRESS ANALYSIS

Results of the stress analysis suggested that the maximum pull force would be 29,000 lbs. (with safety factor) for the bundle pullback. Buoyancy control measures were not utilized in the assessment. The safety factor against collapse and tensile stresses were well within the ASTM allowable tolerances.

HDD CONSTRUCTION

The HDD for Wilson's Beach was executed by first excavating the 0% RQD materials near the entry location to approximately 3.4 m depth, where the rock quality was suitable for HDD installation.

Similar operations as implemented at Chocolate Cover were completed for the Wilson's Beach HDD. Marathon experienced loss of circulation and a fracture to the watercourse during drilling operations. Loss of circulation (LOC) materials were pumped downhole to seal any fractures and to maintain forward progression. The fractures were successfully plugged, and construction proceeded.

Figure 13. Wilson's Beach Annular Pressure Chart

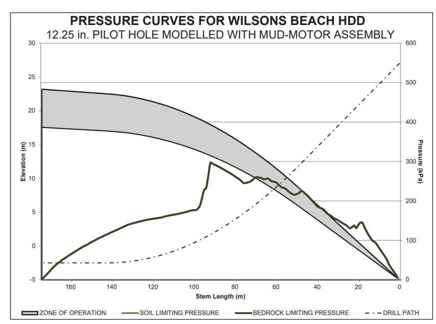
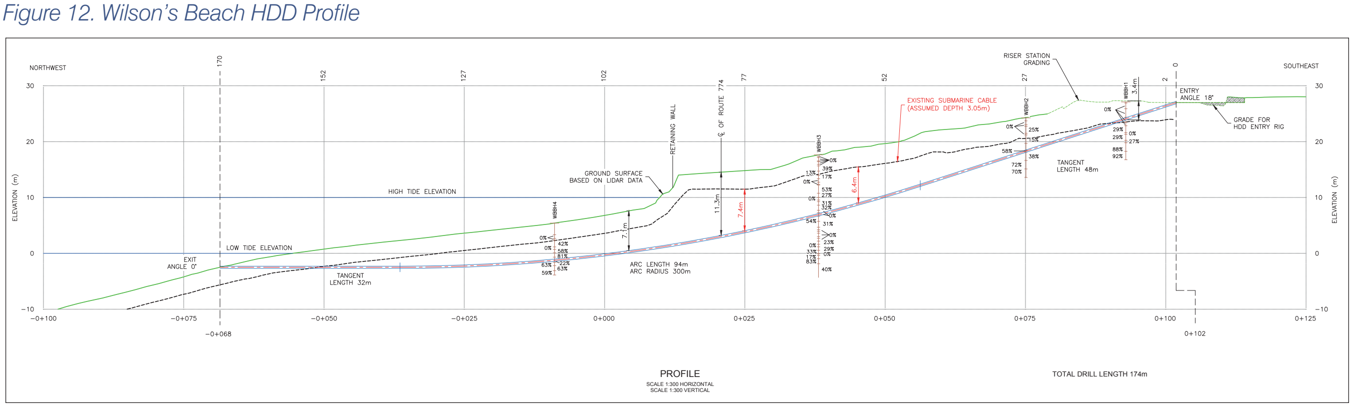
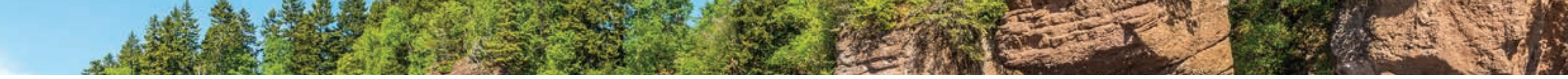


Figure 12. Wilson's Beach HDD Profile





Upon drilling to the exit location, Marathon intercepted approximately 5 to 10 meters of sand over the bedrock and could not maintain the ability to steer up or maintain the borehole wall. The pullback was not completed, as the excavator could not reach deep enough from the barge position and working in the loose materials would have been unsafe for the divers.

LITTLE WHALE COVE ENGINEERING DESIGN AND CONSTRUCTION

SITE DETAILS

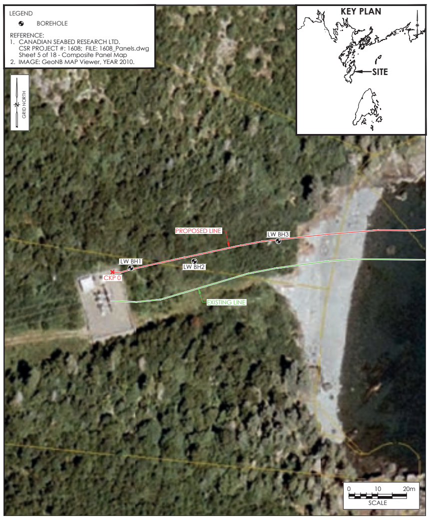
Little Whale Cove is located on the opposite side of Campobello Island to Wilson's Beach, approximately 5 km west. The top of the slope is approximately 22 m above sea level (low tide, 0 m geodetic). The slope of the hill follows an approximately 14.6%, 8.3-degree, downward slope. A similar max tide elevation is present at this crossing.

SITE-SPECIFIC GEOTECHNICAL CONDITIONS

Stantec's borehole program at Little Whale Cove consisted of three boreholes (designated as Boreholes LW BH1, LW BH2, and LW BH3). Access to the shore was not possible for this investigation. Boreholes were completed within 50 m of the top of slope. Borehole locations are shown in Figure 14. Table 3 summarizes the findings of the borehole program.

The borehole program identified shallow overburden materials consisting of topsoil, cobbles, and boulders. These formations were planned to be excavated at the time of construction such that the drill could directly enter the bedrock interface.

Figure 14. Little Whale Cove Borehole Locations



The RQD values of the shale ranged between 0% and 85%, at depth. Images of the low RQD samples are seen below.

Geotechnical conditions at the exit point were similarly investigated by a marine approach. Formations were suggested to be large-sized boulders, approaching the size of a small car. The mitigation measures implemented are discussed in the construction section below.

HDD ALIGNMENT

Similarly to the other crossings, the alignment was selected based on maintaining the design within NB Power property boundaries and crossing the shore as close to perpendicular as practicable.

The design followed a southwest to northeast alignment, with the entry point situated approximately 160 meters southeast the low tide shoreline, at the top of the slope, and the exit point approximately 27 meters to the northwest of the low tide shoreline. The straight-line alignment length for the crossing was 177 m. Figure 17 shows the HDD alignment.

HDD DESIGN DETAILS

The HDD was designed to maximize the depth underneath the surface of the slope, limit the probability of hydraulic fracture to surface, and to maintain the borehole within drillable geotechnical strata. The entry angle of 18 degrees was selected as it would allow drill path to progress quickly through the lower RQD formation and into a more competent bedrock. A horizontal exit was chosen due to the sloping subsea profile and immediate turn requirement to connect the HDD to the subsea section. A 300 m design radius was also utilized at this location.

The depth of cover for this crossing was generally greater than the other crossings. Near the shoreline, the depth was approximately 6 m, and ground cover reduced as exit was approached. The design profile is shown in Figure 18.

Figure 15. Photo of the core run from LW-BH1 0 to 5.2 m showing poor RQD near surface

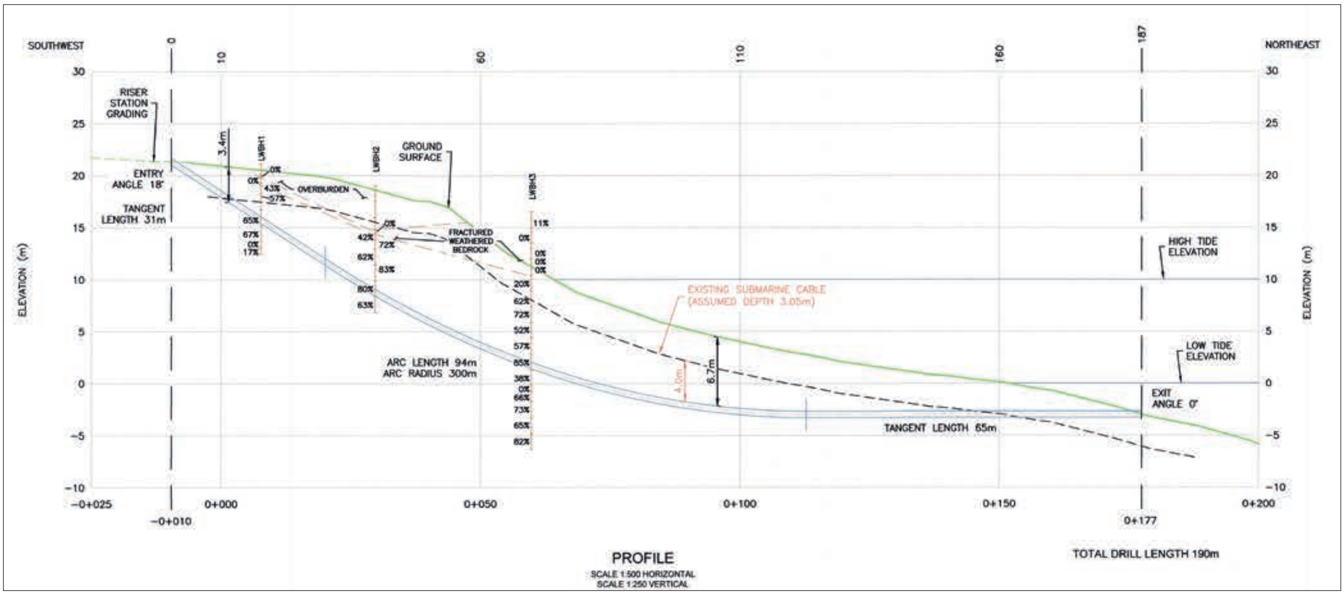
Figure 16. Photo of the core run from LW-BH1 5.2 to 9.0 m showing medium RQD and no recover zone

Table 3. Little Whale Cove Borehole Summary

Borehole	Depth (m)	Approximate Location	Description	Primary Geotechnical Concerns
LW BH1	8.99	Top of slope, near entry location	Topsoil, boulder and cobbles, very poor to fair felsic lithic tuff	Borehole collapse, loss of circulation
LW BH2	12.19	25 m from entry	Topsoil, sand with cobbles and boulders, very poor to good felsic lithic tuff	Loss of circulation
LW BH3	22.86	50 m from entry	Topsoil, very poor to good felsic lithic tuff	Loss of circulation

Figure 17. Little Whale Cove Topography along HDD Alignment

Figure 18. Little Whale Cove HDD Profile



ANNULAR PRESSURE ANALYSIS

According to the AP chart for the Little Whale Cove HDD, the expected overburden fracture containment pressure (overburden strength) is lower than the expected drilling pressures for the majority of the drill path design, indicating a high probability of hydraulic fracture to the surface and waterbody. The annular pressure chart for the Little Whale Cove HDD is shown in Figure 19.

STRESS ANALYSIS

The results of the stress analysis suggested that the maximum pull force would be 34,000 lbs. (with safety factor). The safety factor against collapse and tensile stresses were well within the ASTM allowable tolerances.

HDD CONSTRUCTION

The large-diameter boulders identified were expected to cause complications for the HDD near the exit point. The boulders

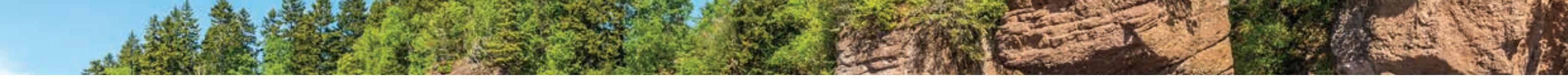
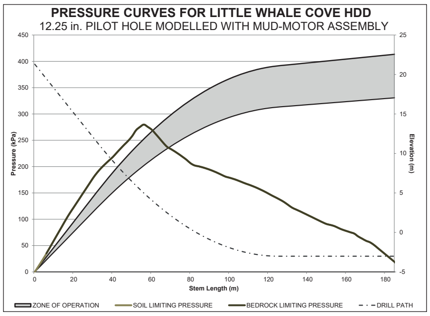


Figure 19. Little Whale Cove Annular Pressure Chart



would have to be moved in order to allow the drill to come to surface. Additionally, there was an increased risk of a boulder falling onto the product pipe or drill string during pullback operations.

Ultimately, due to delays in crossing durations of the other crossings and access restrictions, the scheduled arrival of the marine cable ship was deemed to be too close to the anticipated completion date of the Little Whale Cove HDD. NB Power elected to complete an open cut of the slope to ensure that the conduit would be in place and not result in any additional incurred costs associated with delaying the marine ship. The open cut methodology was a difficult option due to the work on the steep slopes, rock excavation requirements, and inability to achieve additional depth with open cut to mitigate future erosion above the pipe.

LONG EDDY POINT ENGINEERING DESIGN AND CONSTRUCTION

SITE DETAILS

Long Eddy Point (Long Eddy) is located on Grand Manan Island, approximately 14 km southeast of Campobello Island and the Little Whale Cove HDD. The topography at Long Eddy Point is more drastic than the previous crossings, and the elevation profile drops 20 m vertically over 60 m horizontally (33.3% slope, 18.43 degrees).

Figure 20: Long Eddy Point Borehole Locations

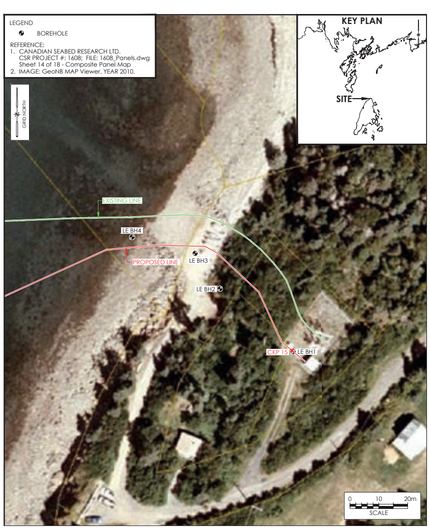


Table 4. Long Eddy Point Borehole Summary

Borehole	Depth (m)	Approximate Location	Description	Primary Geotechnical Concerns
LE BH1	30.48	Top of slope, near entry location	Silt, very poor to excellent basalt	Borehole collapse, loss of circulation
LE BH2	15.24	45 m from entry	Topsoil, very poor to fair basalt	Loss of circulation
LE BH3	15.49	55 m from entry	Cobbles and boulders, very poor to fair basalt	Loss of circulation
LE BH4	11.89	70 m from entry	Cobbles and boulders, very poor to fair basalt	Large cobble and boulder presence, loss of circulation

Figure 21. Photo of the core run from LE-BH1 17.4 to 21.3 m showing medium RQD at depth

Figure 22. Photo of the core run from LE-BH1 21.3 to 25.9 m showing good RQD near the top and poor near the bottom

Figure 23. Long Eddy Point Topography along HDD Alignment

SITE-SPECIFIC GEOTECHNICAL CONDITIONS

Stantec's borehole program at Long Eddy Point consisted of four boreholes drilled between the entry location and the high-tide mark. The borehole locations are shown in Figure 20 and Table 4 summarizes the findings of the borehole program.

The borehole program identified shallow overburden materials consisting of topsoil, cobbles, and boulders. These formations were planned to be excavated at the time of construction such that the drill could directly enter the bedrock interface.

The RQD values of the shale ranged between 0% and 83% and varied with depth. Images of the varying RQD samples are seen above.

The borehole program did not include any borings or other investigations in the Bay of Fundy. Due to the crossing length, this left approximately 250 m of the crossing without known geotechnical conditions. The borehole located at the base of the slope showed cobbles and boulders, which were expected to continue throughout the final section of the crossing. This would pose significant risk for loss of circulation, borehole collapse, and inability for forward progression.

HDD ALIGNMENT

The NB Power easement for the land section at Long Eddy Point required the HDD be rotated approximately 19 degrees from the feasible underwater alignment of the marine section. The HDD centerline required inclusion of two horizontal curve segments to align with the available right of way and marine centerline.

The entry point was situated approximately 216 meters southeast of the low tide shoreline, at the top of the slope, and the exit point approximately 60 meters to

Figure 24. Final Long Eddy Point HDD Profile

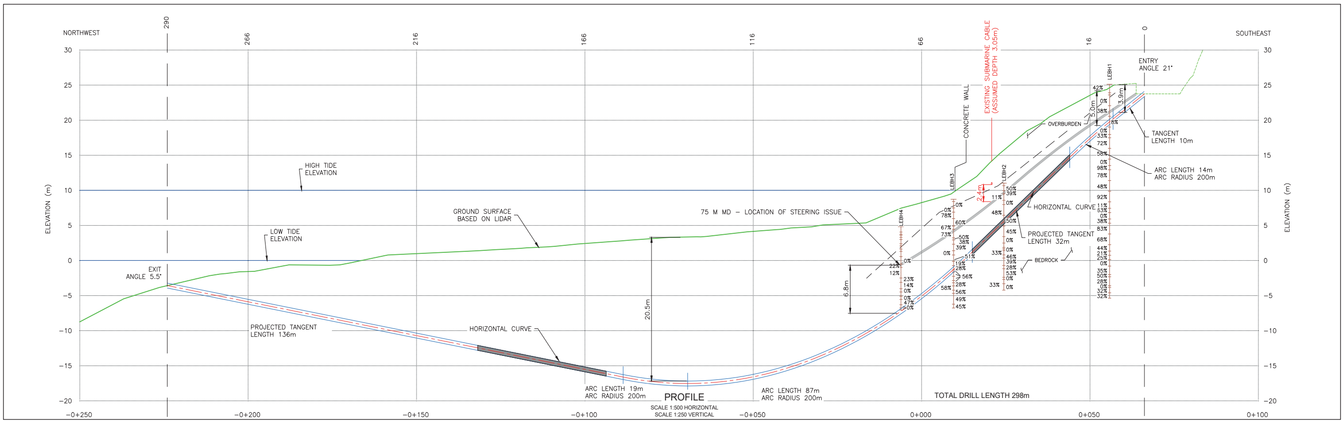
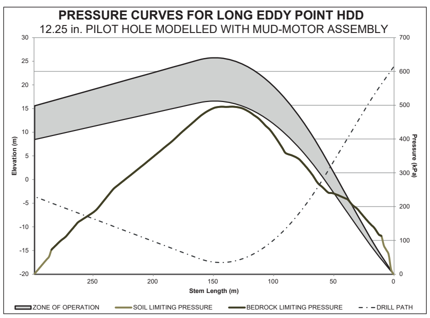


Figure 25. Long Eddy Point Annular Pressure Chart



the northwest of the low tide shoreline. The straight-line alignment length for the crossing was 290 m. Figure 23 shows the HDD alignment.

HDD DESIGN DETAILS

Two HDD designs were developed for the Long Eddy crossing. The first drill path entered at 20 degrees, following the topography down and exiting at the identified exit point above. This attempt during construction was not feasible, as will be discussed in the section that follows. The second design for Long Eddy utilized a higher-than-standard entry angle of 21 degrees in order to gain depth underneath the topography, which sloped downwards at 18 degrees. A 10 m straight tangent was included to enable the contractor to gain sufficient steering reaction before entering a drop curve, which increased the build rate to 25 degrees. This additional angle increase allowed the drill to attain more depth of cover than the original drill path.

A 200 m design radius was elected to be utilized to allow for separate vertical and horizontal bends, rather than introducing any complex steering geometry (compound curves).

ANNULAR PRESSURE ANALYSIS

According to the AP chart for the Long Eddy Point HDD, the expected fracture containment pressure was lower than the expected drilling pressures for the majority of the drill path design, indicating a high probability of hydraulic fracture to the surface and watercourse. The annular pressure chart for the Long Eddy Point HDD is shown in Figure 25.

STRESS ANALYSIS

The results of the stress analysis suggested that the maximum pull force would be 64,000 lbs. (with safety factor). The safety factor against collapse and tensile stresses were well within the ASTM allowable tolerances.

HDD CONSTRUCTION

Marathon's first attempt to complete the crossing intersected the bedrock/cobble layer at approximately 75 m measured depth. It was not possible to progress the drill past this location. A design change was completed, deepening the crossing to remain in the bedrock for a longer length. Loss of circulation and borehole instabilities on the second attempt caused hinderance to the execution of the crossing.

The scheduled date of the marine cable lay ship was fast approaching. In an attempt to prevent further delays to the marine portions, a plan was made with the NB Power to abandon the second attempt and excavate from the exit side to the depth of where the first drill had failed. Marathon ran tooling into the first borehole and forward reamed the existing section of pilot hole. Product pipe was installed from the shore to the entry location, while the section from the shore to the subsea was open cut.

PROJECT SUMMARY AND LESSON LEARNED

NB Powers' 69 kV submarine cable replacement project included four separate locations planned for HDD installations from the substation to approximately 4 m underneath the low tide elevation (0 m geodetic). One crossing was successfully completed as designed. One crossing was modified to suit the geotechnical conditions experienced at site. Two crossings could not be completed, one due to timing restrictions and the second to unforeseen geotechnical conditions.

The geotechnical investigation boreholes were not located over the alignment, and were only completed on land, thus lacking information for the final half of each crossing. Due to the land-only approach, the site conditions differed greatly than what was shown in the boreholes provided. Additional investigation beforehand in the water portion of the crossing could have led to a better understanding of the expected stratigraphy.

Additional buffer between the required completion date of the HDD crossings and the cable lay could have been included to account for any schedule delays and could have led to the completion of the final crossing.

REHABILITATION OF THE LESLIE STREET SANITARY TRUNK SEWER AND MAINTENANCE HOLES

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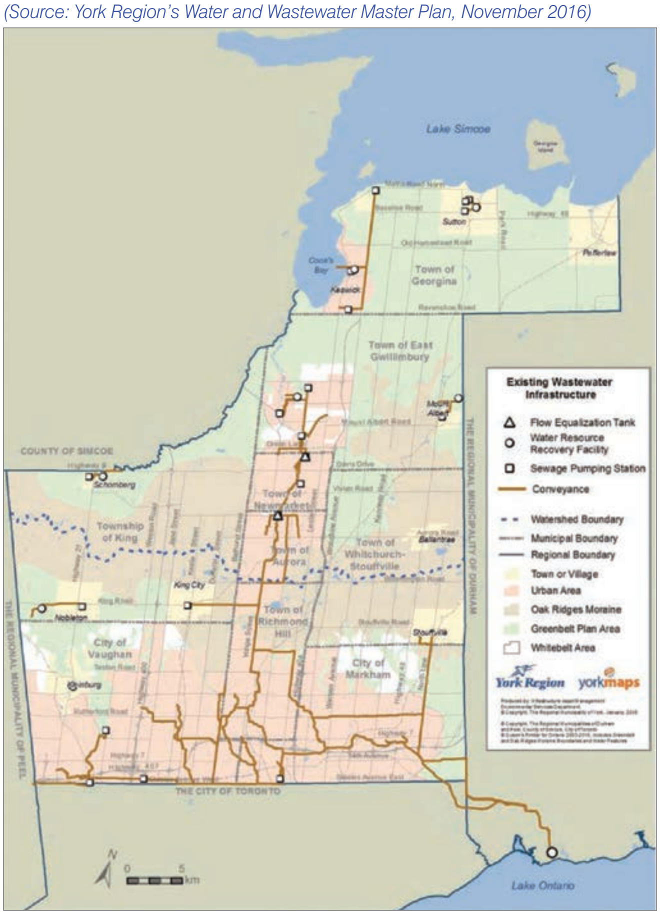
Mark Ortiz, Project Manager, Regional Municipality of York, Newmarket, Ontario

ABSTRACT

Sewage from Regional Municipality of York's largest sewage pumping station, with an average flow rate of approximately 1 m^3/sec, is conveyed downstream via twin forcemains, for a length of 2.5 km. After crossing the highest point in the topography, the sewage is relayed downstream, by gravity, via a 2,100 mm diameter sewer, 79 m long, ~ 4.5 m in depth, referred to as the Leslie St. Sanitary Trunk Sewer (STS). Commissioned in 1979, recent inspections on the sewer displayed exposed reinforcement bars and protruding aggregates; for a continuous defect of 34 m length, that makes up 43% of the sewer's total length.

Maintenance hole (MH) inspections also revealed similar structural defects. With no redundancy available, the sewer was identified in dire need of rehabilitation. The sewer is located on a congested utility corridor, with two transmission towers (500 KV and 200 KV) and a 900 mm gas main ~ 4 m from the sewer. Given the existing flows and site constraints, sliplining was identified as a cost-effective method to rehabilitate the sewer and installing HDPE liners for MH rehabilitation. To prevent migration of corrosion to the next downstream sewer post rehabilitation, the design includes changing turbulent flow to laminar conditions to limit stripping of hydrogen sulfide (H_2S). The existing site conditions pose construction challenges such as strategically locating construction staging areas, and managing flows during rehabilitation without affecting the normal operation of the pumping station. Learning objectives include addressing construction challenges during design, such as flow control and construction footprint management, and presenting construction activities as they proceed. The construction contract was awarded in July 2020 and the construction is expected to be completed by June 2021.

Figure 1. York Region's Baseline Wastewater System
(Source: York Region's Water and Wastewater Master Plan, November 2016)



INTRODUCTION AND BACKGROUND

The Regional Municipality of York (or York Region) is located in the Greater Toronto Area directly north of the City of Toronto. Covering ~ 1,800 square kilometres (km), York Region is bordered by Lake Simcoe and Simcoe County to the north, Peel Region to the west and Durham Region to the east. York Region is an upper tier municipality that provides regional services, including conveyance and treatment of wastewater for its nine, lower tier, local municipalities. York Region owns, operates, and maintains over 214 km of large diameter, gravity trunk sewers with an average diameter of 1,500 mm and an average age of 24 years. Figure 1 depicts the baseline wastewater systems in York Region.

Leslie St. Sewage Pumping Station is the largest sewage pumping station (SPS) owned and operated by York Region. Wastewater collected from the municipalities of Vaughan, Richmond Hill, and Markham is discharged into the Leslie St. SPS. Wastewater collected at Leslie St. SPS, is relayed downstream via twin forcemains, 1,200 mm in diameter, and approximately 2.5 km in length. After negotiating the highest point in topography, the twin forcemains discharge into a 2,100 mm diameter, circular, reinforced concrete pipe (RCP), (C76, Class 3). This gravity sewer is referred to as the Leslie St. Sanitary Trunk Sewer (STS); the sewer was commissioned in 1979 and constructed via the open cut technique. The Leslie St. STS starts at MH 43-04, also known as the discharge chamber, and extends for a length of approximately 80 m, at ~ 5 m depth and at 0.17% gradient, to connect to MH 49-JC, also referred to as the Junction Chamber. A 1,500 mm RCP sanitary sewer, referred to as the Richmond Hill Collector, also connects to MH 49-JC. Downstream of MH 49-JC, the pipe increases to a 2,400 mm RCP sanitary sewer known as the South-East Collector (SEC) sewer. Without any redundancy available, the wastewater from Leslie St. STS is further relayed downstream by a network of SEC sewers and eventually discharges into the Duffin Creek Water Pollution Control Plant (WPCP).

In 2006, routine inspections were conducted on the Leslie St. STS. The video inspection from 2006 didn't reveal many details about the condition of the sewer, owing to the limited visibility during the inspection. In the 2016 inspection, substantial deterioration due to hydrogen sulphide (H_2S) corrosion was noted. The inspection revealed exposed reinforcement, surface spalling and protruding aggregates, resulting in allotting the sewer a structural grade of 4. Under York Region's guidelines for the frequency of inspections, a sewer with a structural grade of 4 requires inspection each year to closely monitor the rate of degradation. Thus, based on the observed condition in 2016, this sewer section was inspected in 2017 and again in 2018. The 2017 and 2018 inspections resonate with the 2016 inspections, i.e., with reinforcement visible at the surface and protruding aggregates defects; both the defects were observed/recorded as a continuous defect that were observed to be distributed along the length of the sewer. Inspection of MH 43-04 revealed structural deterioration in the form of protruding aggregates and reinforcement visible at the surface. Condition assessment of MH 49-JC revealed infiltration stains and a circumferential crack.

To ameliorate the observed conditions of the Leslie St. STS, York Region's Capital Planning and Delivery (CPD) set out a contract for detailed design and contract administration to rehabilitate the STS and the two associated MHs. Jacobs has been contracted by York Region to provide engineering services to complete preliminary and detailed design, contract administration, site inspection, project management and related services for Rehabilitation of the Leslie St. STS.

PROJECT SCOPE

Project Location

The Leslie St. STS is located within Infrastructure Ontario (IO) owned land, leased to Hydro One Networks Inc. (HONI) corridor, just south of the Highway 407 Express Toll Route, east of Leslie St. and West of Highway 404. A private golf practice centre is located to the east of the STS. The project site is across the road from a local high school. Two high voltage transmission towers (500 KV and 200 KV) and overhead hydro cables are located in close proximity to the sewer and MHs under consideration. A 900 mm, vital gas main, runs in parallel and in close proximity to the sewer alignment; the offset distance of the gas main from the STS is approximately 4.5 m. See Figure 2 and Figure 3 for the project location map and an aerial view of the site features, respectively.

Figure 2. Leslie St. STS, Project Location Map

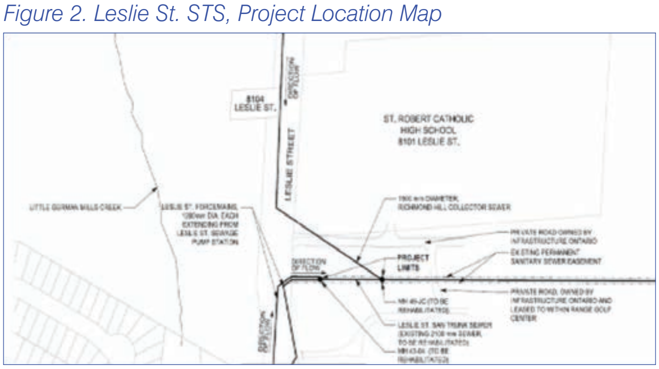
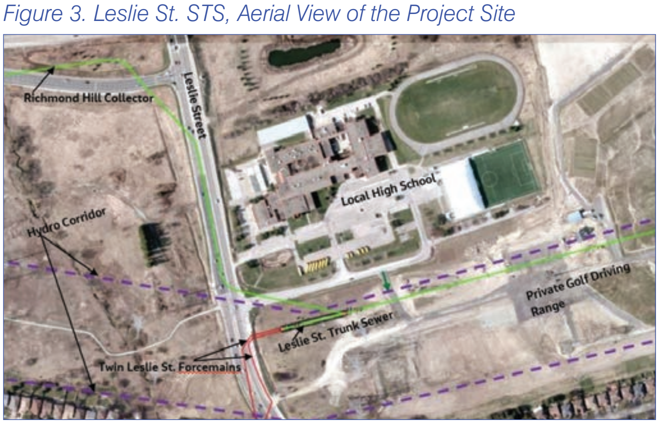


Figure 3. Leslie St. STS, Aerial View of the Project Site



Project Goals

The following are the key objectives of this project:

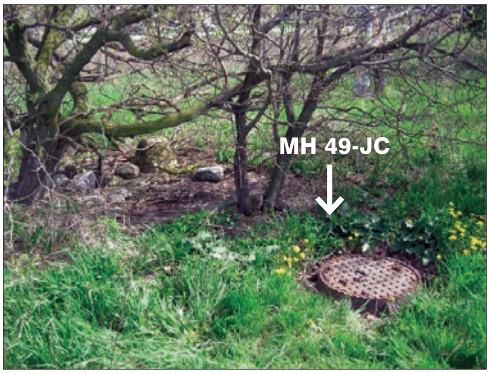
- Structural rehabilitation of Leslie St. STS and associated MHs (i.e., MH 43-04 and MH 49-JC) and extend its Remaining Useful Life (RUL) by 50 years.
- Maintain capacity during construction activities: All flows from Leslie St. SPS are conveyed downstream via the Leslie St. STS, with no redundancy. It will be critical to maintain all flows within Leslie St. STS, during construction activities.
- Minimize operational impact to the Leslie St. SPS during construction: Leslie St. SPS is the largest SPS operated by York Region. All flows arriving at Leslie St. SPS are conveyed to the Leslie St. STS via twin forcemains. One of the key objectives is to maintain the daily/normal operation of the Leslie SPS while conducting rehabilitation of Leslie St. STS.
- Corrosion resistance: The sewer section under consideration has been subject to severe structural deterioration owing to H_2S induced corrosion. The extent of deterioration is further exacerbated due to the forcemains discharging into Leslie St. STS; generally, the sewage conveyed from sewage pumping stations and forcemains is depleted in oxygen in comparison to the gravity sewers. The flow from the forcemains negotiate a drop of approximately 1 m, at MH 43-04, resulting in turbulent flow conditions, resulting in H_2S stripping. Thus, the ability of the selected rehabilitation product to resist H_2S is important.

Project Constraints

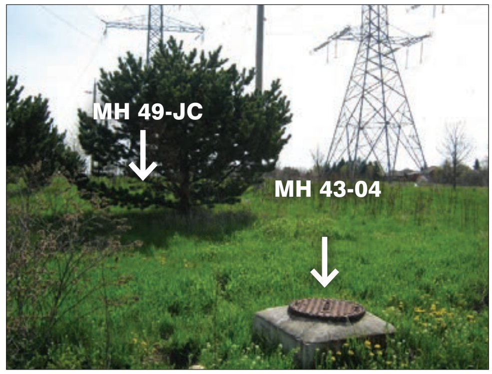
The following are the project-specific constraints that were taken into consideration by Jacobs during the design phase:

- Flow control and maintaining capacity during rehabilitation: Analysis of the Supervisory Control and Data Acquisition (SCADA) obtained from the Leslie St. SPS shows that the flows from the twin forcemains are discharged to MH 43-04 at an average rate of approximately 1.08 m^3/sec. With no redundancy or opportunities for flow diversion, all flows from the Leslie St. STS are relayed approximately 821 m downstream, to the next available MH 49-01, into a 2,400 mm diameter trunk sewer. Thus, all flows within Leslie St. STS will need to be maintained always during construction.
- Coordination with utilities, stakeholders, and construction footprint management: The sewer from MH 43-04 to MH 49-JC is located within the Infrastructure Ontario (IO) owned, leased to HONI and in close proximity to high voltage, transmission line towers and underneath overhead hydro cables. A 900 mm natural gas main, is in the vicinity of the Leslie St. STS's alignment. An alternative access path, directly off Leslie St. is presently blocked with large boulders. Refer to Figure 4 for existing site conditions. Several permits and approvals were required to be obtained from the key stakeholders, prior to conducting site investigations and onset of construction.
- Proximity to private property and school: Construction activities will be near local high school and a private golf centre. Thus, it is important to build and maintain public trust during construction. Restrictions, such as avoiding the use of the road during the school's pick-up and drop-off times, have been applied, i.e., no construction vehicles can access the road from 7:30 am to 6:30 pm on weekdays.

Figure 4. Existing Site Photographs



MH 49-JC, Tree Growth Over the Top of Slab of the MH



MH 43-04 and MH 49-JC, in Proximity to Hydro Tower

Project Site Encumbered with Overhead Cables and High-Voltage Transmission Towers in the Vicinity

Existing Paved Access Road, Located Between the School and Project Site

Sewer Inspection and Condition Assessment

After the 2016 CCTV inspections on the Leslie St. STS revealed a structural grade 4 deterioration, CCTV inspections were conducted again in 2017 and 2018 to closely monitor the degradation in the sewer's fabric. Review of the 2017 inspection revealed reinforcement visible at the surface over a total length of approximately 24 m, i.e., 30% of the inspected pipe length, alongside an intruding sealing ring at one of the joints was observed. Based on the observed defects in the 2017 inspection the structural condition grade for sewer section 43-04 to 49-JC is Grade 4, that is, poor. Review of the 2018 CCTV inspection resonated with the findings of the 2016 and 2017 CCTV inspections; structural deterioration in the form of exposed reinforcement and surface spalling were observed.

The intruding sealing ring continues to be present in the 2018 CCTV inspection. Without action to address these structural defects collapse is considered likely in the foreseeable future. See Figure 5 for a snapshot of the observed conditions of the sewer during 2017 and 2018 CCTV inspections.

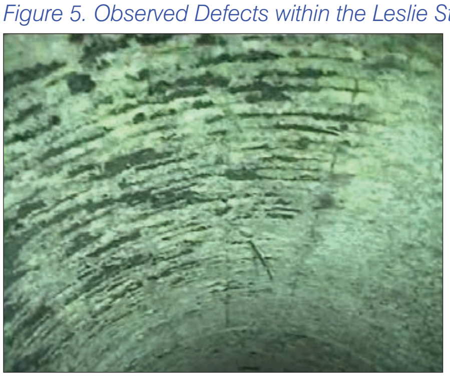
Laser inspections were conducted on this sewer and it revealed corrosion of the pipe fabric throughout the sewer, with values ranging from 29 mm to 101 mm. In the areas where reinforcement was visible, maximum corrosion was observed: 101 mm at 7.3 m from MH 43-04. As the reinforcement corrodes in the presence of sulphuric acid, the structural integrity of the sewer is called into question. Reference to standard C76-02 (the 2002 version of C76) indicates that cover to reinforcing should be 25 mm. This is likely to be a minimum value, but it is slightly concerning that the recorded maximum corrosion in the sewer exceeds this value. No debris was found in the sewer during the sonar inspections. Refer to Figure 6 for a visual of the corrosion observed during the laser inspection.

Maintenance Holes Configuration and Inspection and Condition Assessment

Upstream MH 43-04 is approximately 2,100 mm X 6,000 mm, cast in place MH, while MH 49-JC is 5,100 mm X 9,000 mm, cast in place chamber; refer to Figure 7 for typical details of the two chambers.

National Association of Sewer Service Companies (NASSCO), Level 2, Manhole Condition Assessment and Certification Program (MACP) inspection of MH 49-JC and MH 43-04 was conducted in 2017. The condition assessment of MH 43-04

Figure 5. Observed Defects within the Leslie St. STS, during 2017 and 2018 CCTV Inspections

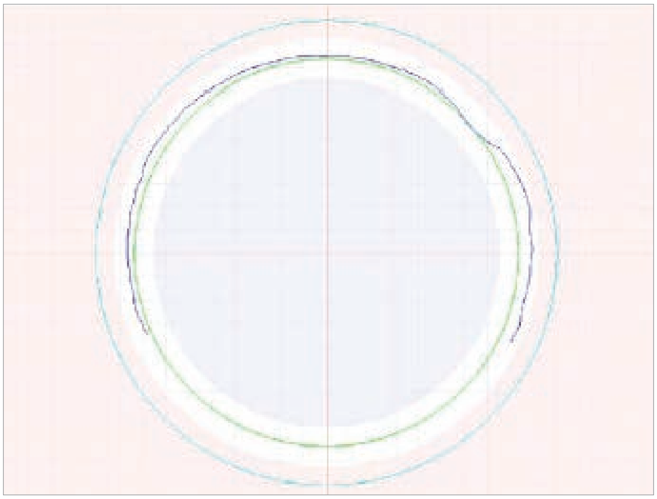


Exposed Reinforcement Bar Visible in the Sewer's Obvert, During the 2017 CCTV Inspection

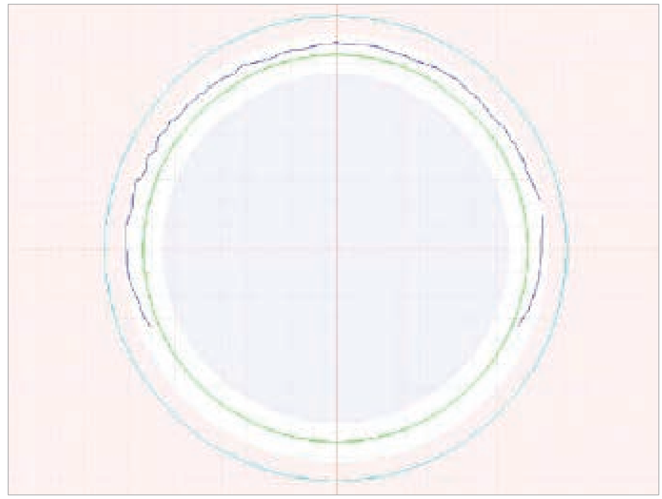
Exposed Reinforcement Bar Visible in the Sewer's Obvert, During the 2017 CCTV Inspection

Exposed Reinforcement Bars at the MH and Pipe Interface, During the 2018 Inspection

Figure 6. Corrosion Observed in the Sewer During the 2017 Laser Inspection



Corrosion to a Depth of 87 mm, Observed at 4.4 m from MH 43-04



Corrosion to a Depth of 101 mm (Maximum Corrosion), Observed at 7.3 m from MH 43-04

revealed surface spalling with protruding and visible aggregate within the chimney and chamber sections continuous over the full depth. The inspection noted spalling of the cone with some visible reinforcement. Based on the observed conditions and the MACP standards, the MH is classified a structural condition grade 4. With the absence of operational defects, the MH is allotted an operational grade 1. MH 43-04 will require structural rehabilitation due to the presence of cracks, protruding surface aggregates, visible surface reinforcement across a considerable length of the MH and on the underside of the roof slab. It is therefore prudent to conduct structural rehabilitation of this MH.

The condition assessment of MH 49-JC revealed a circumferential crack at a depth of 0.62 m within the chimney section. Debris was also noted settled on the benching below the access cover and frame. Otherwise the MH is generally shown to be in fair condition with no evidence that the concrete corrosion due to high levels of H_2S, as exhibited in MH 43-04, upstream, has migrated to this point. This is consistent with the conditions observed in the 2017 CCTV survey of sewer section 43-04 to 49-JC, which showed the surface damage and loss of concrete within the pipe, decreasing further downstream from MH 43-04 towards MH 49-JC, and the images of MH 49-JC taken from the 2016 CCTV survey. Based on the observations made and in accordance with the MACP standard, the MH is classified as Structural Condition Grade 2, fair condition and an operational grade 1, good condition. Although in a fair condition, in order to attain a 50 years RUL, a structural rehabilitation of this MH will be undertaken.

APPROACH TO SEWER AND MH REHABILITATION

Jacobs worked with York Region to apply a Multi-Criteria Analysis (MCA) approach to identify the most cost-effective and reliable technique to restore the structural and operational integrity of the Leslie St. STS. The MCA approach aims to assess the relative suitability of the various rehabilitation options against the physical properties and site-specific circumstances of the sewer. The MCA approach has been utilized to prioritize the most applicable techniques and permits selection of the most feasible and cost-effective rehabilitation technology. The following techniques were reviewed and assessed for their suitability for rehabilitation of the Leslie St. STS:

- Sliplining with Glass Reinforced Polymer (GRP)
- Full Pipe Sliplining
- GRP Panel Lining
- Geopolymer Spray Lining
- Compression Fit Lining
- Panel Lining
- Point Repairs and Joint Seals

Some of the crucial evaluation criteria and relative weightings selected for analysis of applicable products for rehabilitation of the Leslie St. STS are discussed in the following bullets:

- Bypass pumping required during installation: Twin forcemains from LSSPS discharge flows directly into MH 43-04. Flows will need to be maintained through the rehabilitation. This criterion was allotted a high weightage in the evaluation.
- Resistance to H_2S: The presence of H_2S in sewers is a known issue. The ability of a rehabilitation technique to resist H_2S was, therefore, given a high weighting.
- Additional access points or easement required: The MHs in this study area are large and are adequate for insertion of most rehabilitation products. Hence, this criterion was allotted a low weighting.
- Existing pipe surface preparation required: The surface preparation of the sewer to accept the given rehabilitation option is important, as it is critical to achieving proper installation within the host pipe. This factor was, therefore, allotted a high weighting.
- Proven history of similar projects at this diameter: It is important to select only those techniques for consideration that have a track record on sewers of a similar size. This criterion was given a high weighting.
- Improvement of the structural condition of existing pipe: There is considerable loss of concrete cover due to the fact that the reinforcement bars are visible at the surface. Hence, it is imperative that the recommended rehabilitation product provides a fully independent structural solution; this criterion was allotted a high weighting.
- Longevity of installation (useful life): At least 50 years of RUL is expected from the rehabilitation product. Hence, this criterion received a high weighting.
- Impact on hydraulic capacity: Most of the sewer rehabilitation techniques inevitably result in some diameter reduction; however, capacity is not a concern for the Leslie Street sewer and, hence,

this criterion has been scored with a medium weighting.

- Availability of experienced and qualified local installers (with 10 or more years of experience): Most techniques evaluated are well established. Hence, this criterion has been scored with a medium weighting.
- Potential installation problems: Each of the rehabilitation techniques has its own method- or product-related potential installation issues, which need to be carefully monitored and considered during the installation process. The potential impact of these issues on performance and longevity, should they occur, and the effort and cost required to correct these defects can vary greatly. Hence, this criterion was allotted a high weighting in the evaluation.
- Material delivery issues: Local availability of liner materials is preferred to prevent delays in the project schedule; hence, this criterion was allotted a high weighting.
- Minimal ongoing maintenance liabilities: Turbulent conditions were observed in MH 43-04 that could result in damage and wear to some rehabilitation products, increasing the need for future maintenance activities. Minimal maintenance requirements are preferred; hence, this criterion was allotted a high weighing.

The performance of each of the potential rehabilitation technologies was evaluated against the criteria listed above to assess its suitability to address the system problems and site-specific issues and constraints identified for rehabilitation of the sewer. Point repairs and joint seals are not considered structural rehabilitation techniques; therefore, these were not considered in the evaluation. Through a comprehensive analysis and detailed scrutiny of the physical properties of the sewer, the existing flow conditions, access requirements, site constraints and opportunities, effectiveness of the product against corrosion, resistance against infiltration and the bypass requirements, sliplining was identified as the most appropriate and cost-effective technique for rehabilitation of the Leslie St. STS.

From the CCTV and laser inspections, it is evident that the sewer has undergone significant loss of concrete with exposed reinforcement bars visible. For sewer sections that are constructed by the open cut technique such as the Leslie St. STS, it is recommended that the sewer rehabilitation product is designed for the fully deteriorated condition.

The condition assessment of MH 43-04 identified surface damage and wear as a result of H_2S corrosion. During the MH inspections, reinforcement is visible at the surface across much of the chamber walls, below the roof slab. Prior to application of a MH rehabilitation product it will be necessary to remove all loose and deteriorated concrete to a depth of approximately 25 mm below the top layer of reinforcing steel, along the entire depths of the MH. All damaged reinforcement will need to be removed and replaced with new and the chamber walls will need to be structurally rebuilt with a PVC/HDPE corrosion resistant liner. On completion of installation of the sliplining pipe, the chamber roof slab will need to be reconstructed and with the application of the HDPE/PVC corrosion resistant lines, to the inside surfaces of the chamber as well as underside the roof slab. To allow for a more robust rehabilitation solution, application of HDPE/PVC corrosion resistant liners will be implemented on MH 49-JC. MH rehabilitation designs also include replacement of steps and other appurtenances with corrosion-resistant FRP. To permit non-man entry rescue in accordance with York Region's confined space entry procedures, openings within safety grates will be aligned below the cover and frame and all ladders will be continuous from rim to invert.

FLOW CONTROL DURING REHABILITATION

To determine the flow spectrum in the Leslie St. STS, flow information recorded on the SCADA installed at Leslie St. SPS was evaluated by Jacobs. At the Leslie St. SPS, a combination of two 1,200 kW and four 750 kW pumps in different configurations are used for pumping flows downstream. Using SCADA information, for a period of one-year (September 20, 2017 to September 20, 2018), flows discharged from Leslie St. SPS towards MH 43-04 into the Leslie St. STS, were analyzed.

From the SCADA information, calculations were performed to determine the combined flow rates into the Leslie St. STS, from the two forcemains; Table 1 summarizes the results. These flows were averaged over the 24-hours period.

Table 1. Combined Flow Rates into Leslie St. STS, from the Forcemains

Flow Type	Flow Rate (m3/sec)
Average	1.08
95th percentile	1.42
10th percentile	0.97
Maximum	2.17

Identification of Low Flow Conditions

The chosen method of rehabilitation is sliplining; unlike other sewer rehabilitation techniques, this technique can be performed in live flow conditions. The amount of flow remaining in the sewer, during installation depends upon the individual contractor's preference and method of work. Based on our previous sliplining experience, the flows within a sewer typically range from between 20–50% pipe depth, but may be as high as 80–90% pipe depth. For the purposes of this assessment, Jacobs has evaluated the installation scenario at a 30% pipe full depth, equivalent to the existing flow conditions, observed from the 2017 and 2018 CCTV inspections. The flow rate corresponding to a 30% pipe full depth (or 630 mm) is 1.51 m^3/sec; due to dynamic nature of the system, this value is slightly higher than that calculated using Manning's equation (see Figure 8).

Instantaneous flow rates (i.e., measured flows from SCADA at Leslie St. SPS) from September 2017 through September 2018 were evaluated to identify the period during which the flows were equal to or below 30% pipe full depth (i.e., below 1.51 m^3/sec). Analysis revealed that during the entire 12 months of flow information from SCADA, the lowest instantaneous flows were recorded from November 24, 2017 to week of January 5, 2018, but instantaneous flows frequently exceeded the 30% flow depth; this can be attributed to the off-seasonal storm events, off-seasonal thawing cycle observed during the previous winter.

Diurnal Flow Patterns

From evaluation of the SCADA, a period of low flow was identified during the winter months. To evaluate a typical diurnal profile for the Leslie St. STS, a suitable period of low flow conditions (dry weather observed during winter) was identified.

From Figure 9, the maximum depth of flow in the Leslie St. STS, during low flow/dry weather conditions, falls just around 30% of pipe full. A few instances where the flow depth was above 30%, this aberration can be attributed to off-seasonal storm events and off-seasonal snow melts. Detailed analysis of the flow data shows that flows typically drop to around 25% pipe full depth between the hours of 2:00 am to 7:00 am each night during dry weather/

Figure 8. Depth-Flow Relationship According to Manning's Equation

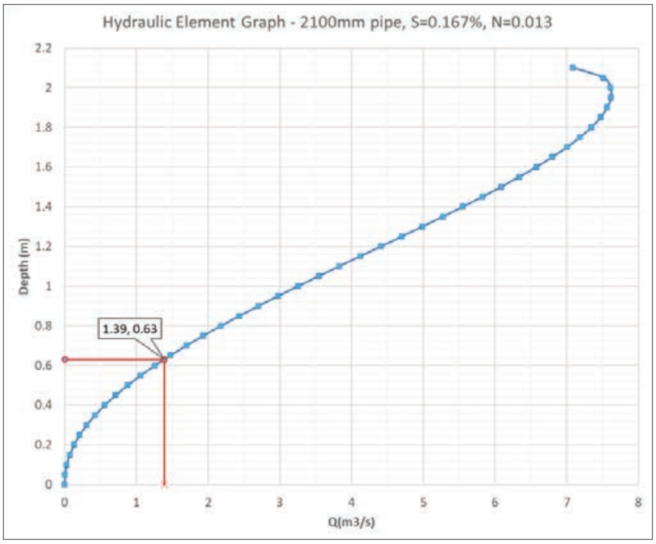
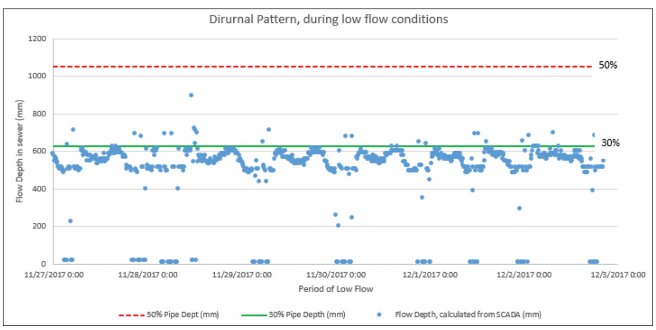


Figure 9. Flow Depth During Dry Weather/Low Flow Conditions, Calculated from SCADA Flows



low flow conditions. At no time do the levels in the sewer cross or reach 50% pipe full depth. Review of the velocities associated with the SCADA flow rates are such that sliplining can be undertaken without the need for bypass. Thus, no bypass installation will be required to maintain flows below 50% pipe full depth.

Based on the results of the modeling and SCADA evaluation efforts, anticipated flows within the sewer during the rehabilitation work are such that sliplining can be undertaken in live flow conditions, without the need for installation of external bypass pumps. Additionally, to account for unforeseen conditions during construction, an evaluation of the Leslie St. pumps data determined that there is sufficient residual pressure available within the forcemains to overcome the headloss associated with pumping flows for an additional ~ 90 m, should the need for temporary bypass of flows around the working area to MH 49-JC arise; thus, the need for installation of external bypass pumps is eliminated.

During the rehabilitation of MH 43-04 and MH 49-JC, temporary sewer plugs with integrated flume pipe will be installed in the incoming forcemains and the 1,500 mm diameter Richmond Hill Collector sewer. A flexible hose will be used to transport residual flows through the structure, under, over or around existing features to discharge into the outgoing pipe a short distance downstream of the working area. To allow safe working conditions during rehabilitation, the contractor will be required to consider a factor of safety of 2 while calculating the maximum pressure on the sewer plugs or the chosen flow control measure. A temporary bulkhead will be constructed at the entrance of the downstream sewer to prevent any backflow.

DESIGN CONSIDERATIONS

Site Access

There is no direct vehicular access to the chambers or a route along the sewer and MHs. However, the area is flat and is accessible via two existing access roads located north and south of the project location (refer to Figure 2). The access road to the north, between the local high school and the project site, is paved and originates at a signalized intersection on Leslie Street; this is a private road owned by IO. Constructed before the local golf driving centre to provide access to the school grounds, it is also used to provide access the golf centre and to stormwater facilities east of Leslie St. and as an alternate access to the hydro corridor, allowing the access off Leslie St. to be closed.

A gravel access road off Leslie Street to the south of the sewer provides access to the hydro corridor. However, this access is currently blocked to prevent unwarranted access to the hydro corridor. For the purposes of accessing the project site during construction, the daily entry and egress to the site will be undertaken from this south, gravel access road; this type of daily access improves safety and minimizes disruption to the stakeholders in the vicinity, i.e., the patrons at the golf centre and the school.

For the purposes of large construction equipment or materials to be dropped off with the initiation of construction and to be picked up, post completion of construction, the contractor shall make use of the north access road, while exiting the project site, whilst the entry will continue to be from the south access road. To accommodate school's requirements, no access during the school hours will be permitted, i.e., no use of the north access road from 7:30 am to 6:30 pm, on weekdays. Safety and ease of construction access were built into the design by considering construction vehicle access routes, safe turn-around, length, width, number of axels, gross weight of the largest construction vehicle/equipment anticipated to be present on the site, and laydown and staging areas.

Construction Footprint Management

The sewer and associated MHs are located within HONI corridor and near transmission line towers and overhead power cables; York Region possesses a 6 m wide total sewer easement around the centre line of the pipe. The temporary construction staging areas at 43-04 and 49-JC and temporary access roads have been sized appropriately to accommodate the proposed construction vehicles, activities and for storage of materials and equipment while also taking into considerations the needs of HONI, vital gas main, and golf centre for access to their facilities and infrastructure for their routine.

Requirements Stipulated by HONI:

HONI stipulates that all construction activities be conducted at a distance greater than 10 m from the base of any given transmission tower. The existing exclusion distance between MH 49-JC and the nearest transmission tower is approximately 14 m. To allow the rehabilitation works at MH 49-JC, this exclusion distance will be temporarily reduced to 11 m, during construction. Temporary construction fences, 1.8 m high with pad locks, will be erected along the perimeter of all staging and laydown areas. In addition, to prevent knocking off the towers, jersey barriers

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in conformance with the Ontario Provincial Standard Drawings (OPSD) 911.140, around the base of the towers, will be erected prior to the start of construction. The contract documents also require a signaler or flagman to be provided for work within the 10 m exclusion zones of the existing towers. The flagman shall warn the operator each time any part of the vehicle or equipment or its load may approach the minimum distances. No grading/ excavation work will be carried out using mechanical methods. Excavations using hydro vacuum truck system or manual excavations will be considered. All metallic components will be grounded to prevent nuisance shocks.

Requirements Stipulated by the Vital Gas Main Owner

A 900 mm, steel coated, vital gas main. runs in parallel alignment to the Leslie St. STS, on the north side. The separation distance between the Leslie St. STS and the gas main is 4.5 m. Prior to commencement of rehabilitation of the sewer and MHs, permit from the owner of the gas main was obtained. For approval of the permit, the following to be implemented during construction:

No mechanical excavation within 1 m of their gas main: The current recommended rehabilitation method is a trenchless solution with minimal excavation requirements. The downstream chamber, MH 49-JC, is a rectangular chamber measuring, 5,100 mm X 9,000 mm and is approximately 4 m deep. To gain access into the sewer, top slab of MH 49-JC, shall be removed. The depth of MH 49-JC from the ground level to the sewer's obvert is approximately a meter only. Excavations, to expose and remove the chamber's roof slabs to allow sliplining installation and perform MH rehabilitation of MH 49-JC, will be limited to 1.0–1.5 m, which is feasible by manual excavation or VAC methods.

Loading restriction: Owner of the vital gas main, has restricted the weight of construction vehicles up to a maximum of 10,000 kgs/axle. No construction vehicles will be allowed to cross over the gas main. The only time when the vehicles/equipment shall cross the gas main is during the access over the existing asphalt road. Jacobs had performed calculations for determining the maximum superimposed load which worked out to be less than the maximum requirement of 10,000 kgs/axle. Additionally, to build in a factor of safety, use of low ground pressure construction equipment/vehicles while working directly over the existing sewer and forcemains. has been specified in the contract. The ground pressure of vehicles intended to traverse over the sewer, gas main, and the forcemains, is restricted to 40 KPa.

Rehabilitation Design Considerations

The MCA for evaluation of sewer rehabilitation alternatives of sewer section MH 43-04 to MH 49-JC identified sliplining with CCFRPM or GRP as the preferred method for rehabilitation. For installation of the sliplining pipe it will be necessary to remove the existing chamber roof slabs of MHs 43-04 and 49-JC to provide clear unobstructed access to the sewer. Recommendations for treatment of the existing MHs will therefore be limited to rehabilitation of the chamber walls, channels, and inverts together with reconstruction of the roof slabs and provision of new access hatches and covers and frames. To allow for a robust, long-term corrosion resistant solution for rehabilitating the MHs, corrosion protection HDPE/PVC liners, including the underside of the newly cast roof slab will be installed during construction. MH 43-04 and 49 JC are big chambers, large enough to permit installation of the sliplining pipe without the need for construction of temporary launching and receiving pits. This significantly reduces the impact and disturbance to stakeholders and the existing environment.

Figure 10. Construction Activities on the Project Site

Setup of Contractor's Laydown Area

MH 43-04 and MH 49-JC, in Proximity to Hydro Tower Steel Plates Installed, Above the Forcemains, at Discharge Chamber, MH 43-04

Top Slab Exposed at Discharge Chamber, MH 43-04

“Given the scope of work, the construction sequencing will be completed within a single contract. A single contract for Leslie St. STS project will provide simplicity and ease of contractor management.”

Generally, it is preferred to slip-line from the downstream MH in the upstream direction, since that provides better control of the pipe during installation and prevents washing away of the slipliner in the downstream sewer. Hence installation will take place from 49-JC in an upstream direction.

Currently, the wastewater from the forcemains negotiates a drop of approximately 1 m before hitting the bench of MH 43-04; this jump/drop results in turbulent flow that allows for H_2S stripping, accelerating the effects of corrosion. To address the root cause of the H_2S stripping, i.e., the turbulent flow discharge, during construction, re-benching of MH 43-04 to allow for a laminar flow regime in the sewer and thus prevent migration of the H_2S corrosion effects into the next downstream sewer, post rehabilitation. The new benching will be better sloped to allow for a laminar flow regime. A corrosion resistant coating will be applied to line the new benching.

Access design for the rehabilitated MHs considers compliance with York Region’s design guidelines and standards for access and the requirements for safe access of confined space entry policy. These may be summarized as follows: all MH manhole covers to be 1,200 mm X 1,200 mm minimum, having a single leaf operated hatch cover. All hatch covers will be bolt down lids installed with continuous rubber gasket (modular/multi section gaskets are not permitted) and stainless-steel bolts.

Construction Phase

Given the scope of work, the construction sequencing will be completed within a single contract. A single contract for Leslie St. STS project will provide simplicity and ease of contractor management. An estimated construction period of approximately six to eight calendar months is anticipated for the completion of the rehabilitation of the Leslie St. STS, along with the construction of temporary access roads, temporary laydown, and staging areas. To ensure selection of the most appropriate contracting teams based on proven experience and qualifications and innovative packaging of contract works to maximize efficiency and reduce unnecessary costs, a pre-qualification of the construction contractor was undertaken. In the subsequent and final step in tender process, the contractor was awarded the construction tender on July 21, 2020. Since contractor’s mobilization on the site, to date, site preparation activities – including construction of temporary access roads, installation of laydown and staging areas construction, installation of erosion and sedimentation control measures – have been completed; refer to Figure 10. The presentation will be updated with the construction activities, as they proceed on the site.

CONCLUSIONS

Flows from York Region’s largest wastewater pumping station are conveyed downstream by the Leslie St. STS, without redundancy. Recent inspections on the sewer, revealed structural deterioration in the form of exposed reinforcement and protruding aggregates, allotting the sewer a structural grade 4. Given the physical properties of the sewer, high sewage flows, lack of redundancy and the site characteristics, sliplining was identified as the recommended technique to restore the structural integrity of the sewer and extend the RUL by at least 50 years.

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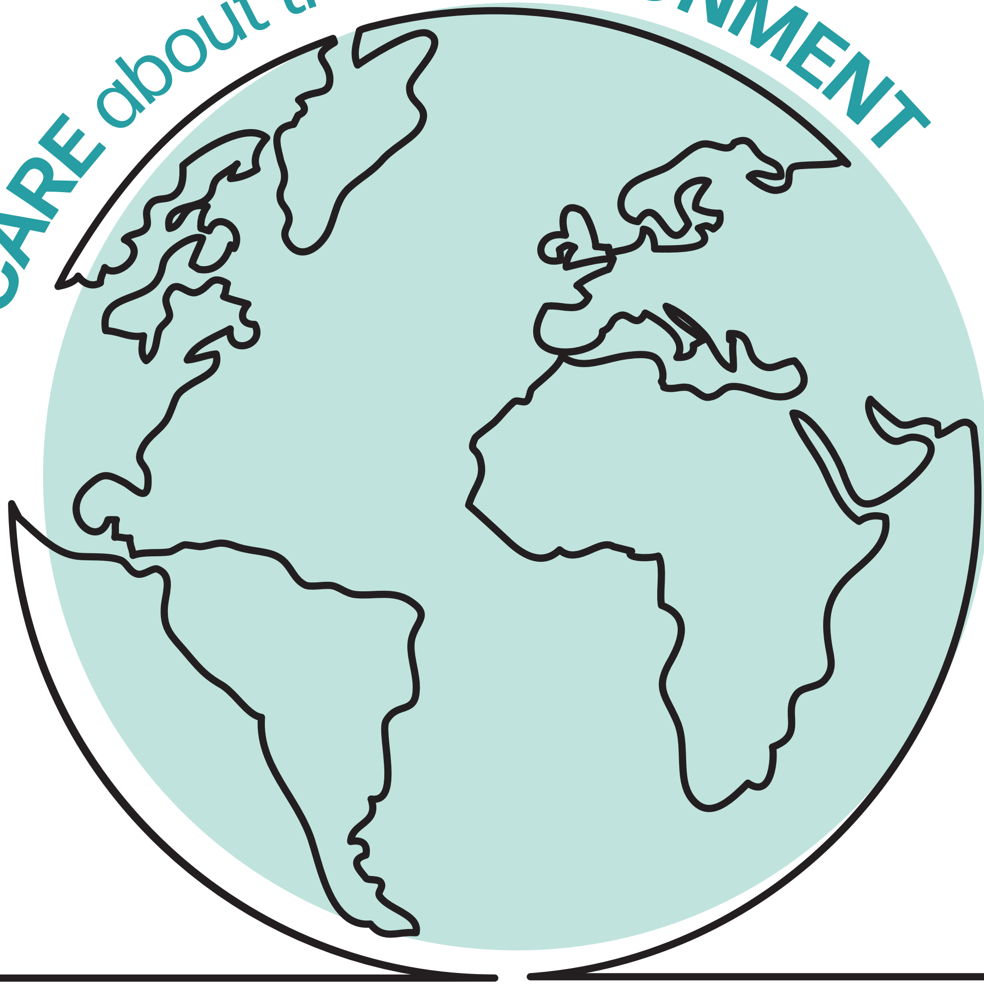

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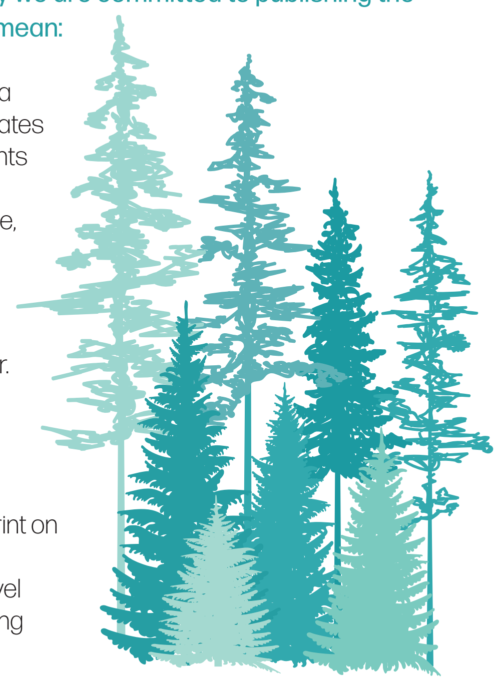

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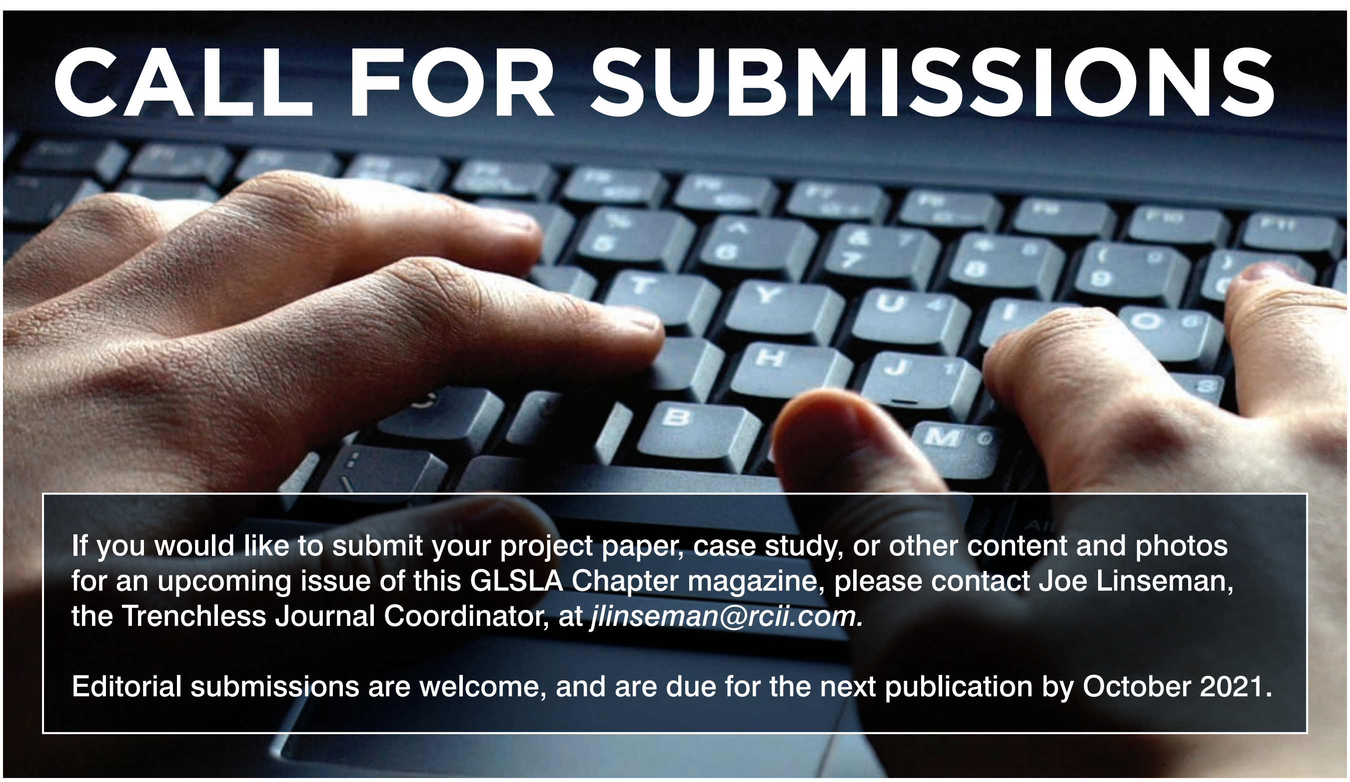

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