



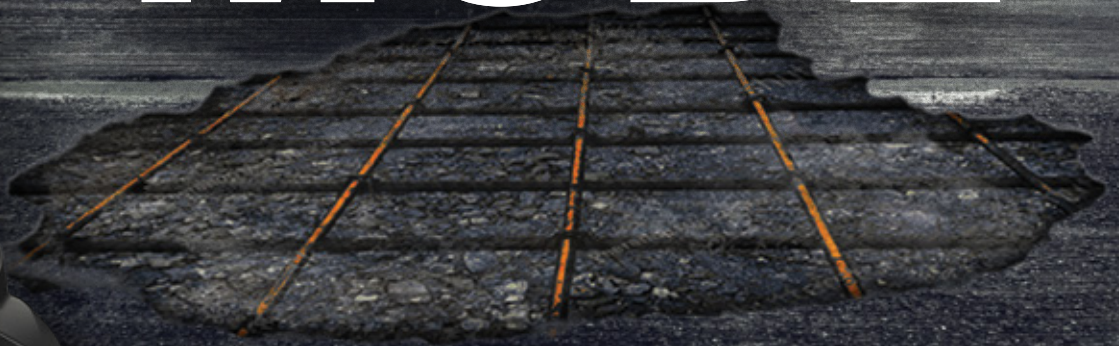
# Pacific Northwest Trenchless Review

*Forward With Trenchless Technology*

# 2023

**Restoring Water System Confidence**  
**HDD Intersect Installation in Alaska**  
**Construction Data Collection**

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# Call for Abstracts

**SUBMISSION DEADLINE: JUNE 30, 2023**

The North American Society for Trenchless Technology (NASTT) is now accepting abstracts for its 2024 No-Dig Show in Providence, RI at the Rhode Island Convention Center April 14-18, 2024. Prospective authors are invited to submit a 250-word abstract outlining the scope of their paper and the principal points of benefit to the trenchless industry.

**The abstracts must be submitted electronically by June 30, 2023 on the NASTT website:**

[nastt.org/no-dig-show](https://nastt.org/no-dig-show)



*The No-Dig Show is owned by the North American Society for Trenchless Technology (NASTT), a not-for-profit educational and technical society established in 1990 to promote trenchless technology for the public benefit. For more information about NASTT, visit our website at [nastt.org](https://nastt.org).*

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# MESSAGE FROM THE NASTT CHAIR

Matthew Wallin, P.E., Chair, NASTT

## The 2023 No-Dig Show Comes to the Pacific Northwest!

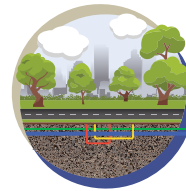
The NASTT 2023 No-Dig Show will be held right here in the PNW Chapter's backyard in Portland, OR April 30 – May 4. The city of Portland is a perfect location for our industry to come together to celebrate and educate with the theme, *Green Above, Green Below*. It is important that our industry is a steward of our precious natural resources, and we welcome the opportunity to provide a forum for learning about the latest in innovative trenchless products and services that help us all accomplish that lofty goal. Learn more about all the No-Dig Show has to offer at [www.nastt.org/no-dig-show](http://www.nastt.org/no-dig-show).

In the coming months we have many additional events planned to bring the underground infrastructure community together. Our ever-popular NASTT Good Practices Courses are being held both virtually and in-person throughout the year. Visit [nastt.org/training/events](http://nastt.org/training/events) to find a course that fits your schedule.

This fall we are excited to head to Edmonton, Alberta for the 2023 No-Dig North conference, October 23-25. No-Dig North is hosted by the Canadian Chapters of NASTT and offers three full days of

training, education, and networking. This is a must-attend event for trenchless training and networking in Canada and nearby portions of the US. Visit [www.nodignorth.ca](http://www.nodignorth.ca) for details!

If you have attended a NASTT event (national or regional) you probably left feeling excited and eager to get more involved. I ask that you consider getting engaged in one of the many NASTT committees that focus on a wide variety of topics. Some of our committees that are always looking for fresh ideas and new members are the Training and Publications Committee, the individual topic Good Practices Course Sub-Committees, the Educational Fund Auction Committee, the No-Dig Show Planning Committee and the No-Dig Show Technical Program Committee. There are many opportunities for you to consider where your professional expertise can be put to use through networking with other motivated volunteers. With education as our goal and striving to provide valuable, accessible learning tools to our community, we are proud of our continued growth as both an organization



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and as an industry. Our volunteers and committee members are what keep us moving in the right direction.

For more information on our organization, committees, and member benefits, visit our website at [nastt.org](http://nastt.org) and please feel free to contact us at [info@nastt.org](mailto:info@nastt.org).

We look forward to seeing you at a regional or national conference or training event soon! And we hope you are planning to join us in Portland for the 2023 No-Dig Show April 30-May 4.

*Matthew Wallin*

Matthew Wallin, P.E.  
Chair, NASTT Board of Directors

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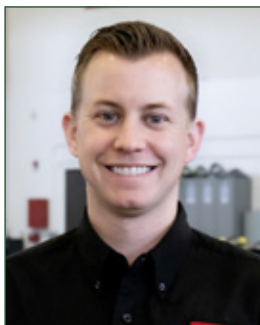
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**GLEN WHEELER**  
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glenw@jwfowler.com

Glen Wheeler joined J.W. Fowler Co. (JWF) as their first college intern over 10 years ago, serving as a Field Engineer Intern for a King County, Washington Earth Pressure Balance Tunnel Boring Machine crossing under the ship canal in downtown Seattle. After graduating from the Colorado School of Mines with a B.S. in Mining Engineering, Glen joined JWF full time as a field engineer in the tunneling and trenchless division. Throughout his career, he has continually assumed more responsibilities, leading to his current role of Chief Tunnel Engineer overseeing a staff of intern, field, and tunnel engineers.

Glen has led the technical development of some of JWF's most challenging tunnels including microtunneling, open face shield tunneling, pipe ramming, pilot tube boring, hard rock tunneling, earth pressure balance tunneling, and other underground projects across the United States.

As the author of several white papers and articles, Glen has been active in sharing his experience and expertise with the trenchless industry. He has spoken at several NASTT events, to the Wash. Dept. of Transportation, to Oregon State University engineering students, and to other industry association groups about the challenges and achievements of trenchless technology.



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rylee.archuleta@  
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Rylee has 8 years of civil engineering experience specializing in sanitary and storm sewer design and inflow and infiltration study and reduction planning. She is an employee at Leeway Engineering Solutions in Portland, Oregon where she is currently managing multiple projects related to trenchless rehabilitation design. Rylee obtained a B.S. in Civil Engineering from the University of Portland and is licensed as a Professional Engineer in Oregon and Washington. She is an active member of NASTT and currently serves as the secretary of the PNW chapter. In her free time, Rylee enjoys spending time outdoors, whether it's biking, skiing, or enjoying a cool drink on a sunny patio – preferably with her dog in tow.



**BRIAN GASTROCK PE -  
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AT LARGE**  
**Coffman Engineers, Inc.**

brian.gastrock@coffman.com

Brian Gastrock, PE has been a member of NASTT since 2007 and brings more than 21 years of civil engineering experience working on condition assessment, design, and construction management projects. His experience includes sliplining, CIPP, pipe bursting, coatings, HDD, pipe ramming, auger boring, and pilot tube guided boring for water, wastewater, and storm drain projects. Brian has extensive experience implementing trenchless solutions; helping clients realize the cost and construction impacts of trenchless alternatives.

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**BRENDAN  
O'SULLIVAN -  
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Brendan O'Sullivan is a Principal Engineer and Trenchless Technologies Technical Practice Leader for Consor working out of Portland, Oregon. He has 18 years of experience in the consulting industry serving Municipal clients throughout the United States. Brendan graduated from the University of Portland with a bachelor's degree in civil engineering in 2004 and serves in a variety of roles for infrastructure projects that focus on pressure pipelines, gravity conveyance, and trenchless technologies (rehab and new installation) for water and wastewater projects. He is a licensed professional engineer in Oregon, Washington, Texas, and Tennessee.



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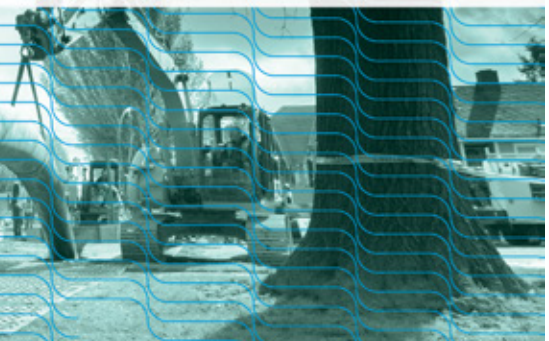
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# Restoring Water System Confidence with CIPP

By: Brendan O’Sullivan, PE, Consor

After multiple delays in project construction, associated with project permitting and railroad coordination, the City of Salem, Oregon’s (City) public works department completed the rehabilitation and replacement of a critical water main in their distribution system in the summer of 2020. The approximately 560-foot long section of existing 30-inch diameter welded steel water main is an important backbone of the water distribution system, providing redundancy and supply to the industrial area in the south-central neighborhood of the City. Installed in 1947, and identified for rehabilitation/replacement in the City’s 2007 Water Master Plan, the water main is reaching the end of its design life. With a portion of the water main having been repaired to address pinhole leaks in 2012, the City has been proactive in wanting to address the water main deterioration before a failure resulted in the release of chlorinated potable water into an environmentally sensitive ecosystem. Approximately 200 feet of the water main is located in Pringle Creek, a fish habitat for listed aquatic species. Utilizing CIPP lining technology and open cut construction methods the City rehabilitated and replaced the ageing steel waterline in and adjacent to the creek, including a portion that runs beneath a 100-foot wide railroad bridge and associated easement.

In July of 2017, the City of Salem contracted Consor (consultant, previously known as Murraysmith, Inc.) through an existing Continuing Services Agreement, to provide engineering services (design, bidding, and construction) for the rehabilitation and replacement of the

critical water main. Referred to as the Oxford Intertie the pipeline connects two distribution networks providing redundancy and looping to the water system. The Oxford Intertie, a welded steel pipe, has experienced corrosion related condition issues in the form of pinhole leaks on the section of pipe within the creek. Having addressed the deficiencies in the past and a consensus amongst City staff that the failure of the pipeline was unacceptable, pipe rehab/replacement was the only option. In addition to potential pipeline deficiencies, it was known by City maintenance staff that the existing isolation valves on either end of the intertie were not fully seating. Given the age of the valves, 1947, it was decided that new valves would be installed, and the existing valves decommissioned as part of the project design. The valve installation work was performed by City crews in advance of proposed contractor performed

construction activities to ensure proper water main isolation.

## DESIGN

Design efforts began in August 2017, with a site reconnaissance by City and Consultant staff. During this site visit it became apparent most of the water main located in the creek, running perpendicular to a Union Pacific Railroad (UPRR) bridge, would have to be rehabilitated rather than replaced due to the constraint imposed by the UPRR railroad bridge and its operation. The seven-track railroad bridge provides approximately 5.5 feet of clearance above the creek bed and was constructed predominantly of wooden timber. Additionally, the railroad would remain in service during any construction activities, making the option of open cut construction under the bridge very high risk and likely non-permittable. All



*Fish Salvage Operation*

sections of the water main located in the creek banks and upland would be replaced in kind with 30-inch diameter, Class 52 ductile iron (DI) pipe via open cut construction.

For the rehabilitation of the water main, in the creek and perpendicular to the railroad bridge, there were only two viable techniques that could potentially achieve the fully structural rehabilitation independent of the existing welded steel pipe. These techniques were cured-in-place pipe (CIPP) pressure pipe liners and sliplining. Any technology used would need to meet the NSF / ANSI Standard 61 for drinking water. Based on operational parameters and known, or assumed, characteristics of the existing water main, City engineering staff performed a hydraulic analysis of the intertie. It was determined the cross-sectional area of the existing water main needed to be maximized to the greatest extent possible to continue providing a similar level of service. When considering the structural requirements for the project and those provided by the potential lining options, it was determined CIPP was the only product that could meet all of the project requirements.

The liner for this project was designed to yield minimum liner thicknesses in accordance with AWWA's M28 – Rehabilitation of Water Mains Manual, ASTM F1216 – Standard Practice For Rehabilitation of Existing Pipelines and Conduits by the Inversion and Curing of a Resin-Impregnated Tube, and ASTM F2019 – Standard Practice For Rehabilitation Of Existing Pipelines and Conduits by the Pulled in Place Installation of Glass Reinforced Plastic (GRP) Cured-In-Place Thermosetting Resin Pipe. To increase bidding competition, the design team's approach was to allow both UV and thermal set curing products/techniques and let the local CIPP market dictate the selected method. Regardless of curing techniques either method would be required to meet the same minimum design standards for physical properties per ASTM F1216 and F2019.

A critical factor for the success of the CIPP liner installation would be the CIPP end connections. These connections would include the transition in pipe materials and the CIPP liner end-seals. To make the

Table 1: Design Parameters:

Parameter	Minimum Value
Design Safety Factor	2
Minimum Design	50 years
Ovality	2%
Soil Density	120lbs/cubic inch
Live Load	Max 10-feet water above invert
Soil Modulus	700 psi
Internal Operating Pressure	80 psi
Internal Vacuum Condition	8.7 psi

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connections between the existing welded steel and the new ductile iron piping, custom fabricated reducing couplings, with electrically isolating boots manufactured specific to the outside diameters of the project pipes, were specified. The CIPP liner would line through the pipe material transition and terminate in the new ductile iron pipe and internal mechanical joint style seals would be used for the ends-seals. With new DI pipe at the connections, there is a potential for exfiltration of chlorinated water into the creek in the near term by way of cracks in the mortar lining of the pipe. These cracks are typically sealed/closed through the autogenous healing process which begins once the lining is completely saturated. This process can take weeks to months to occur. To mitigate this short term risk of exfiltration, without sacrificing pipe longevity, the mortar lining material was removed from the DI pipe at the termination of the CIPP liner and the end seals will be connected directly to the barrel of the DI pipe with exposed metal surfaces first being prepared and coated with an NSF-61 approved epoxy product to protected against corrosion.

The record drawing for the existing steel water main consisted of a single line diagram and contained no information regarding pipe dimensions or lining and

coating materials. From the City's previous experience of repairing the water main in 2012, it was known that the water main had a bitumastic coating. To overcome this information gap, the project incorporated the exploratory work required to verify the water mains' physical characteristics into construction efforts. This exploratory work consisted of physical measurements and CCTV inspection. The contractor

was responsible for verifying internal and external pipe diameters of the water main piping prior to ordering of CIPP liner, internal mechanical joint end seals, and connection fittings. To ensure feasibility of this approach, the design team performed outreach to local contractors and suppliers during design to understand availability and approximate lead times for potential project materials to verify construction



*Liner Installation*

could be completed within the allowed in-water work window specified by state regulatory agencies.

## ENVIRONMENTAL CONCERNS

Based on previous experience of the project team in and around Pringle Creek, it was known that the creek was/is a habitat for multiple EPA listed aquatic species. For the reach of creek impacted by this project, the listed species included Steelhead and Lamprey Ammocytes. Due to the potential presence of fish, the design included the installation of creek isolation structures, associated bypass piping, and provisions for fish salvage operations performed by qualified fish biologists.

## PERMITTING

Based on desktop research, the project team confirmed two permits would need to be obtained for the project. A Joint Permit Application (JPA) and a Railroad Right-of-Way (ROW) Permit. The JPA is submitted to the Oregon Department of State Lands (DSL) and The Army Core of Engineers (USACE) and is then coordinated and reviewed by a dozen state and federal agencies. The permitting effort is performed under DSL's Aquatic Resources Management Program (ARM).

The mission of the ARM is to conserve, restore and protect the waters of this state and the ecosystem services they provide through implementation of the State's removal-fill and wetlands planning and conservation laws. The ARM program also manages State-owned waterways to preserve the public trust rights of navigation, fishing, and recreation.

The submission of the completed JPA occurred in January 2018 after the project footprint and potential impacts were determined at the 60-percent design level. The permitting review process was anticipated to take 90 to 120 days. The result of the review process was the issuance of a DSL Removal/Fill (RF) Permit. This permit authorized the temporary removal and fill of approximately 45.3 cubic yards of material below the ordinary high-water mark (OHWM) of Pringle Creek. The permitted RF work had to be completed during the in-water work period for the creek established by ODFW

between June 1 and October 15. However, due to delays between the consulting agencies, the RF permit was not issued until July 2018, approximately 60 days longer than anticipated.

The City's bidding procedures dictate that projects shall have all necessary permits in hand before publicly advertising a construction project. The issuance of the permit in July didn't provide the necessary time for bidding, contracting and completion of construction activities within the 2018 in-water work period. Therefore, the decision was made to postpone the project advertising until Spring of 2019. This decision required the City to obtain an extension of the RF permit to accommodate the new construction schedule.

Although there is an existing easement agreement between the City and UPRR for the existing waterline, a ROW permit from UPRR was required for the placement of the creek bypass piping. Due to constraints on both ends of the project and under the rail bridge the existing easement was deemed inadequate in size for the anticipated creek bypass piping. The ROW permits allowed the project to occupy additional area within the UPRR ROW, outside the existing easement, without the procurement of additional easement(s). The ROW permitting was coordinated in

advance with the local UPRR Yardmaster and once a contractor was on board the permit application was submitted through the UPRR headquarters. One condition of the permit that was known in advance and incorporated into the contract documents was the requirement of railroad flaggers when construction activities (excavation and CIPP install) were performed within 25-feet of the ROW. Once the permit fees were paid and necessary insurance certificates provided the permit was issued for the project in June 2019.

## CONSTRUCTION, TAKE 1

The project was advertised for construction on February 15, 2019 with a bid opening on March 12, 2019. Five bids were received from general contractors. After a thorough review of the bid packages for completeness and accuracy, the contract was awarded to the second lowest bidder. This was a result of anomalies noticed in the bid package of the apparent low bidder that rendered their bid non-responsive.

The Prime Contractor mobilized to the site on June 5, 2019 and began pothole excavation efforts to expose the existing water line to verify the pipeline the following day. The steel watermain had an interior diameter of 29 inches, a 5/8-inch wall thickness, and was lined and coated with a bitumastic

Table 2: Bid Results

Bidder No.	Bid Amount
Contractor #1	\$498,900 (non-responsive)
Contractor #1	\$512,315
Contractor #1	\$519,694
<b>Engineer's Estimate</b>	<b>\$531,460</b>
Contractor #1	\$548,864
Contractor #1	\$578,449

Table 3: Approved Liner Properties

Initial Flexural Strength	39,160 psi
Long-Term Flexural Strength	30,457 psi (77% retention)
Initial Modulus Elasticity	2,973,273 psi
Long-Term Modulus of Elasticity	2,320,603 (78% retention)
Thickness	8.3 mm

Construction Activity			2019												2020											
	Start	Finish	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S								
Mobilization	6/5/2019		★																							
Water Main Exploration	6/5/2019	6/7/2019	■																							
Submittal Reviews	6/7/2019	6/21/2019	■																							
Material Lead Time	6/22/2019	9/9/2019	■	■	■	■	■																			
Water Main Rehab and Replacement	8/12/2019	9/18/2019				■	■																			
UPRR Bridge Project Construction	9/3/2019	9/20/2019				■	■	■																		
UPRR Bridge Project Delays	9/21/2019	11/29/2019					■	■	■	■	■															
CIPP Liner Lead Time	3/1/2020	5/15/2020											■	■	■											
Re-Mobilization	8/12/2020																	★								
Water Main Rehab and Replacement	8/13/2020	9/9/2020																■								

Construction Schedule

material. With the physical characteristics confirmed, the CIPP liner properties could be determined, approved, and ordered. The 29-inch interior diameter of the steel pipe was established to be 1 inch smaller than the interior diameter of the DI pipe.

Due to the rigidity of the CIPP pressure liner this diameter difference would create a significant annular space between liner and host pipe. This condition increases the risk of a thinner CIPP liner, introduces a potential failure point, a created a potential warranty issue since the offset exceed the liner manufactures tolerances for installation. To mitigate these risks, custom DI spools were fabricated with a thicker mortar lining to match the diameter of the existing water main. With the final details of the CIPP end connections confirmed all custom materials were ordered with the longest lead time being eight weeks for the CIPP liner. The CIPP liner approved for installation was a UV cured, vinyl resin impregnated glass fiber reinforced tube.

In preparation for in-water work, the prime contractor, mobilized a crane on August 12 to begin the installation of the creek isolation structures/cofferdams and associated bypass piping. Before the upstream cofferdam installation could be completed, certified fish biologists performed fish salvaging operations using seining and electrofishing capture techniques. Salvage operation confirmed the presence of one of the listed species for Pringle Creek, Lamprey Ammocetes.

Of the 176 salvaged fish, 8.5 percent were listed species. Utilizing an assorted size of sandbags and corrugated HDPE pipe, creek flows were diverted through the project area effectively isolating the project work area. With the project area isolated, CIPP liner was scheduled to begin installation the week of September 9.

As the site was being prepared for the delivery and installation of the CIPP liner and DI pipe, the City was informed by UPRR representatives on August 20 that the railroad ROW had to be vacated to facilitate a bridge replacement project that would begin the first week of September and have a three-week construction duration. On August 22 the creek bypass piping and the lower cofferdam were removed, thereby vacating the ROW. On September 3 the UPRR contractor began mobilizing to the site with an anticipated final completion date of September 20th. This anticipated completion date was not achieved. Due to utility conflict issues that arose on the west side of the bridge limits and challenges with pile driving in the creek, the bridge replacement project experienced delays that resulted in final completion being achieved at the end of November.

With the delays of the bridge replacement project, it was determined that the completion of the water main project in the 2019 construction window/in-water work window was impossible. The City issued an Engineering Directive to the contractor to remove the upper cofferdam

and secure the site for construction to resume the project in June 2020. On Thursday, September 18 the upper cofferdam was removed, and the site was secured. The bridge project delays and subsequent engineering directive resulted in an extra \$70,000 in costs for the City and the loss/disposal of the CIPP liner since the new construction window would be well beyond the 6-month shelf life of the liner.



Liner End Seal

## CONSTRUCTION, TAKE 2

After the construction delays experienced in 2019 the project got back on track in 2020. The CIPP liner passed through US Customs controls and was received in July. In preparation for the CIPP installation fish salvage, cofferdam installation and creek bypass pump/piping installation occurred the week of August 17. Of the 254 salvaged fish, 16 percent were listed species. The CIPP liner was successfully installed and cured on August 26 and mechanical end seals, using custom WEKO Step-Seals by Miller Pipeline Corp., were installed on August 27. The final connections were made on August 28 and the rehabilitated water line was successfully pressure tested at 125 psi on September 2. In early September, the Contractor worked to complete final creek bed and surface restoration while the City performed disinfection and bacteriological testing activities. With passing results, the intertie waterline was placed back into service by September 11.

Often the critical path of an infrastructure project, the permitting

“The permitting process is crucial to completing a project on time.”

process is crucial to completing a project on time. All the lessons learned on this project are related to the permitting efforts. The four lessons learned by the City and design team on this project were:

- 1) Understand the permitting agency drivers & tailor applications accordingly. This is easier said than done as each as each reviewer will have different perspective and interpretation of relevant law, codes, or guidance accordingly.
- 2) Assume worse case review duration, then add float to the schedule in the event of delays.
- 3) Interagency coordination can be a big challenge that owners and engineers have no control over. Constant communication can't overcome all situations and one must be flexible.

- 4) Even with advanced communication, constant coordination, and a permit in hand a project may still be derailed by internal needs/desires of a railroad company. †

### ABOUT THE AUTHOR:



*Brendan O'Sullivan is a Principal Engineer and Trenchless Technologies Technical Practice Leader for Consor working out of Portland, Oregon. He*

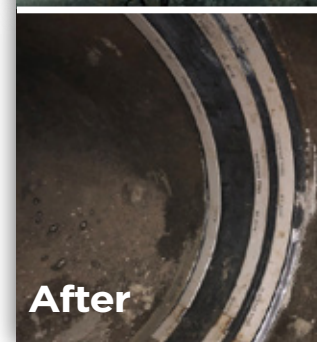
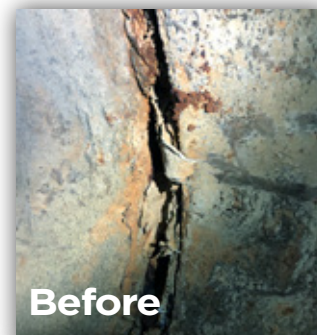
*has 18 years of experience in the consulting industry serving Municipal clients throughout the United States. His is Member-at-Large on the PNW-NASTT Board of Directors*

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# Two Challenging HDD Sites Require Intersect Installation Method in Alaska

By: Brian Gastrock, PE – Coffman Engineers, Inc.

## 1. INTRODUCTION & PROJECT DESCRIPTION

In southcentral Alaska, a few hours' drive from Anchorage, a natural gas line was installed in 2020 to connect new natural gas wells to the existing distribution grid. The project included new compressor equipment, along with approximately two miles of pipeline. The route included crossing two separate creeks with wide valleys having vertical relief of more than 130 feet from top of valley to creek/river bottom. Both sides of the valley were relatively similar in elevation as shown in Figures 1 and 2.

The new 10-inch gas pipeline helps to supply natural gas to Anchorage, the largest demand of natural gas in Alaska. The alignment was proposed by the client and was generally along property lines. Coffman Engineers (Coffman) provided engineering services on the pipeline, compressor pad, and cathodic protection system. The design was completed in less than three months after Coffman was approached by the client in February 2020 with construction scheduled for summer 2020. To facilitate the creek/river crossings, HDD was selected as the preferred method of installation by the client.

Two HDD alignments were proposed as part of the project. The northern alignment was approximately 1,750 feet long and the southern alignment was approximately 1,950 feet long. For the purpose of this paper, the northern alignment crossed a creek, and the southern alignment crossed a river. The northern alignment also crossed a road, and a large wetland. The north alignment valley was relatively wide, approximately 1,100 feet, with approximately 50 to 70 feet

of elevation change from the top of the valley to the river bottom. The southern alignment valley was approximately 900 feet wide with more than 130 feet of vertical relief from the top of the valley to the river bottom. The southern alignment also crossed property that was designated moose habitat and the larger river is a salmon bearing river, popular for local fishermen and tourists.

Collecting geotechnical data along the alignment was determined early as a necessity to help reduce the risk of claims by an HDD contractor. Access to the alignments required additional coordination by the geotechnical engineer and subcontractor. The southern alignment required the use of helicopters to lift the drilling equipment into place to perform one of the geotechnical investigations. Originally, the design team proposed two soil borings at the bottom of the valley but one was completed due to cost and the fast-track schedule.

The horizontal alignment proposed by a general mainline contractor with HDD experience required adjustment in the initial design phase. After the design team evaluated the contractor's proposed alignment, it was determined the required entry and exit angles were more than 40 degrees to accommodate the horizontal alignment. The design team's evaluation included minimum depth of cover under the river of approximately 80 feet to increase the overburden pressure and aid in containing the drill mud in the borehole. The initial evaluation was based on general geologic knowledge of the area as the geotechnical investigation had not been completed at the time of the evaluation. The plan view alignments generally followed existing property lines and easements but did not account for

the topography and vertical relief of the southern alignment.

## 2. SITE ACCESS & GEOTECHNICAL INVESTIGATIONS

The overall project encompassed nearly two miles of pipeline, crossing a variety of terrain. A majority of the alignment is generally flat through undeveloped areas. Occasional driveways, grassy fields, and sparse forests are the typical surface features along the direct-bury portion of the alignment. HDD was quickly determined as the most logical installation method for the river crossings to reduce permitting efforts and expedite construction timeline.

The southern HDD alignment was adjusted significantly, namely it was relocated through protected habitat for moose. This required the client to meet with the property owner and describe the HDD process and mitigation measures in the event of hydrofracture. After the HDD process was described to the property owner, the easement and agreement were approved relatively quickly.

Existing subsurface information along the alignment did not exist beyond the general knowledge of local geotechnical engineers. Initially, the client asked if geotechnical investigations were necessary. Coffman met with the client to express the risks associated with not contracting with a geotechnical engineer to evaluate the subsurface conditions. The client agreed to perform six soil borings, three for the north HDD, and three for the southern HDD. Initially, four borings were recommended for the south HDD, but due to the limited site access to the bottom of the valley only one boring in the middle of the alignment was approved and required



a helicopter to lower the drill rig to the site. The south HDD site access did not have road or trail access, and a helicopter lift was required to lower the geotechnical drill rig to perform the investigation.

The geotechnical investigation was completed in less than three weeks and suggested the proposed HDD alignments and profiles were feasible based on the subsurface conditions encountered. On the southern HDD alignment, possible cobbles were identified in the geotechnical investigation, at the HDD pilot bore profile depth. On the northern HDD alignment, a boring was not performed near the existing roadway crossing. These two circumstances led to issues during construction.

### 3. HDD PROFILE AND SOIL CONDITIONS

The original profile proposed by the general mainline contractor to the client was to drill 15 feet under each of the rivers.

Coffman met with the client early to explain the risk of inadvertent returns (frac-out) associated with the proposed drill profile geometry and ground surface topography at each site. After discussions with the client, it was agreed to adjust the alignment and profiles to help reduce the risk of frac-out during installation.

During the initial design evaluation, it was determined the northern HDD alignment had less risk of frac-outs due to the elevation change being approximately 70 feet from entry point to creek channel. While the south HDD had an elevation change of approximately 130 feet from the entry point to the river channel. The northern HDD alignment geotechnical investigation found very dense to hard silt 20 feet below the river but to accommodate the geometry of the entry and exit angles and to provide adequate depth of cover beneath the toe of the slopes, the profile was designed at 50 feet below the creek channel. The final profile

issued for construction is shown in Figure 1.

The initial southern HDD profile was developed prior to completing the geotechnical investigation to aid in finalizing the geotechnical site investigation. To reduce the risk of formational fluid loss and frac-out due to the anticipated free draining gravel below the river, the first designed profile was approximately 80 feet below the river. After the geotechnical investigation was completed a very dense silty sand was found between 33 feet and 58 feet below the river. Using this information, the profile was shallowed up to 45 feet below the river channel in an attempt to stay in the bottom third of the very dense soil layer. It was at this point that discussions with the selected HDD contractor led to revising the profile design to include an “S-curve” in the eastern portion of the profile. The revised drawing also added approximately 70 feet to the overall length of the HDD alignment to a design length of 2,020 feet in length. The revised profile of the south HDD is shown in Figure 2.



Figure 1. North HDD

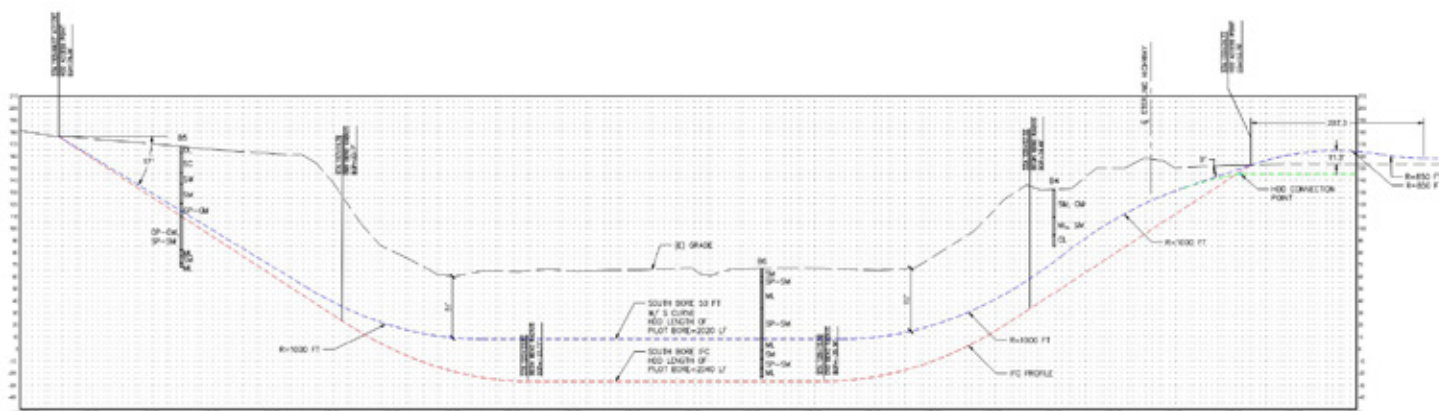


Figure 2. South HDD

#### 4. HDD MUD PRESSURE & PULL CALCULATIONS

The design calculations from the Horizontal Directional Drilling Good Practices Guidelines (4th Edition) were used for the project. Using the equations from the Good Practices Guidelines and a unit weight of drill mud of 9 pounds per gallon (lb/gal), the north HDD pullback calculations estimated the maximum startup pull force required was 33,200 pounds (lbs) of force at the end of the second vertical curve/beginning of bottom tangent. This startup force had the lowest factor of safety (FoS) of 16.8 relative to the maximum allowable pull force. Table 1 summarizes the calculated pull loads, startup loads, and corresponding FoS.

The south HDD pullback calculations estimated the maximum startup pull force required was 45,200 lbs of force at the exist point. This startup force had the lowest factor of safety (FoS) of 12.3 relative to the maximum allowable pull force. Table 2 summarizes the calculated pull loads, startup loads, and corresponding FoS.

To evaluate the risk of hydrofracture, the Good Practices Guideline equations were used to calculate the FoS against hydraulic fracture. The maximum and minimum allowable mud pressures were also calculated using the Good Practices Guidelines equations, 5-75 and 5-79, as shown below.

Using the two equations below and the values provided by the geotechnical engineer and identified in the Good Practices Guidelines (bore radius, radius of the Plastic Zone, soil friction angle, soil cohesion and soil unit weight), an FoS for each of the six geotechnical borings was calculated. Our design team also interpolated the subsurface soils to the toe of the slopes at the bottom of the valley as the depth of cover at these

Table 1. North Bore Pull Force Calculations

Location	Calculated Loads (lbs)	Startup Loads (lbs)	Factor of Safety (FoS)
Entry Point	15,028	30,055	18.5
End of Straight Tangent/Beginning of Vertical Curve	15,025	30,051	18.5
End of Vertical Curve/Start of Second Vertical Curve	14,680	29,359	19.0
End of Second Vertical Curve/Beginning of Straight Section	16,618	33,236	16.8
End of Straight Section/Beginning of Third Vertical Curve	15,891	31,782	17.5
End of Third Vertical Curve/Beginning of Straight Tangent Section	14,639	29,278	19.0
Exit Point	14,592	29,184	19.1
Maximum Allowable Pull Force $F_s = 557,308$ lbs			

Table 2. South Bore Pull Force Calculations

Location	Calculated Loads (lbs)	Startup Loads (lbs)	Factor of Safety (FoS)
Entry Point	16,780	33,560	16.6
End of Straight Tangent/Beginning of Vertical Curve	16,534	33,068	16.9
End of Vertical Curve/Start of Second Vertical Curve	19,086	38,172	14.6
End of Second Vertical Curve/Beginning of Straight Section	21,363	42,726	13.0
End of Straight Section/Beginning of Third Vertical Curve	20,411	40,822	13.7
End of Third Vertical Curve/Beginning of Straight Tangent Section	22,789	45,577	12.2
Exit Point	22,641	45,282	12.3
Maximum Allowable Pull Force $F_s = 557,308$ lbs			

$$p_{max} = u + [\sigma'_0 \cdot (1 + \sin \varphi) + c \cdot \cos \varphi + c \cdot \cot \varphi] \cdot \left( \left( \frac{R_0}{R_{pmax}} \right)^2 + \frac{\sigma'_0 \cdot \sin \varphi + c \cdot \cos \varphi}{G} \right)^{\frac{-\sin \varphi}{1 + \sin \varphi}} - c \cdot \cot \varphi \quad [5-75]$$

and

$$p_{min} = \frac{7.48 \cdot \gamma_{mud} \cdot h_{bore}}{144} + L_{bore} \cdot \left[ \left( \frac{\mu_p \cdot v}{1000 \cdot (d_{bore} - d_{pipe})^2} \right) + \left( \frac{\tau_y}{200 \cdot (d_{bore} - d_{pipe})} \right) \right] \quad [5-79]$$

locations were less than the midpoints of the valley-bottom. The calculations were completed at the profile elevations and not for each soil unit encountered. The highest FoS calculated was 11.8, for the east side of the south HDD profile. This boring encountered “weak” bedrock approximately 20 feet below the ground surface so the FoS calculated was in alignment with the investigation. While calculating the risk of hydrofracture in bedrock can be unconservative due to existing fractures in the bedrock, discussions with the geotechnical engineer during the investigation gave confidence to reduced hydrofracture risk. The geotechnical engineer had prior experience with the bedrock layer encountered on other projects and expressed the risk of hydrofracture in other layers of the HDD profile were higher, as described in the following paragraph.

The lowest FoS was calculated at the midpoint of the south HDD profile. This boring was the lowest point of the profile and encountered the highest mud column, and therefore the highest annular pressures. The calculated FoS was 2.4. After performing the calculations using the geotechnical data, the client was provided the calculations and agreed with the HDD designs.

## 6. CONSTRUCTION

The HDD designs were bid on by multiple contractors, including two local Alaskan contractors. The low-price contractor was one of the Alaskan contractors and as part of their bid they had planned to purchase a larger 250-ton drill rig. The drill rig arrived on schedule and the contractor began construction on the south HDD in the summer of 2020. Coffman was not provided detailed information regarding the specifics of HDD durations, pull forces encountered, or HDD operations. Coffman was informed by the client the design calculations and subsurface conditions were generally the same as encountered during construction.

The contractor set up the drill rig on the west side of the alignment and made the anticipated progress for the first few days. At approximately 200 feet into the pilot bore, the contractor encountered cobbles and drilling was forced to stop

after several attempts were made. After multiple attempts, including the installation of pneumatically installed conductor casing and a washover casing failed, the contractor elected to mobilize to the north HDD and attempt the south HDD at a later date. The specifics of why the attempts failed were not shared with Coffman.

In the fall of 2020, the HDD contractor mobilized their equipment to the north HDD alignment, with the drill rig located at the north end of the alignment, per the design. The first 100 feet of the profile

*“To facilitate the creek/river crossings, HDD was selected.”*

was located under an existing roadway. The road was the primary access to rural residents and closing the road was not feasible. Within the first 50 feet of the pilot bore, the contractor encountered



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Figure 3. South HDD Plan view with geotechnical boring locations

very loose soils. This was not anticipated during the design phase as the soils were non-native backfill located under a primary roadway owned by the Department of Transportation (DOT). The DOT’s design standards of 95% minim compaction of structural soils was assumed during the design. The loose soils resulted in loss of steering control and frac-outs at the toe of the south side of the roadway embankment.

The client and contractor contacted Coffman to evaluate the use of a low

compressive strength sand-cement slurry that would be installed using injection grouting techniques. The method was approved and within a day a local batch plant was able to provide the sand-cement slurry to the site. The slurry was installed via the pilot borehole and allowed to set before resuming drilling. After the slurry set, the contractor attempted to resume drilling. Again, there were frac-outs at the toes of the slope and the contractor again experienced loss of steering control. It was at this time the client released

the HDD contractor and entered into a contract with another larger, national HDD contractor.

The new HDD contractor proposed a pilot hole intersect for the south HDD as well as a larger drill rig (500-ton capacity) than what was available in Alaska. Additionally, the new HDD contractor proposed drilling in winter conditions to reduce the risk of frac-outs having the top 8 to 10 feet of soil frozen during drilling operations. The intersect method proved successful on the northern HDD alignment with no known additional frac-outs during installation. Additionally, the north HDD was completed without difficulties using the larger rig.

## 7. PROJECT RESULTS

This project gave the client the advantage of using HDD to cross sensitive areas (creeks and moose habitat) while minimizing project impacts. The project did experience additional time and cost to contract the second HDD contractor. The project was also delayed a few extra months due to the first contractor unable to complete the HDDs, but in the end the pipeline was generally installed per the design. ✚

## 8. REFERENCES

- Bennett, D., and Ariaratnam, S. (2017)  
 – Horizontal Directional Drilling Good Practices Guidelines, North American Society for Trenchless Technology (NASTT), Fourth Edition, USA



Figure 4. Staging area 1



Figure 5. Staging area 2

### ABOUT THE AUTHOR:



**Brian Gastrock, PE** has been a member of NASTT since 2007 and brings more than 21 years of civil engineering experience working on condition assessment, design, and construction management projects. His experience includes sliplining, CIPP, pipe bursting, coatings, HDD, pipe ramming, auger boring, and pilot tube guided boring for water, wastewater, and storm drain projects. Brian is Treasurer of the PNW-NASTT Chapter.

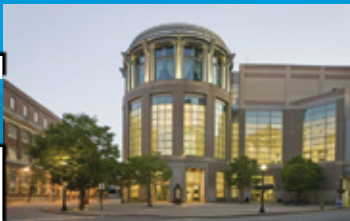
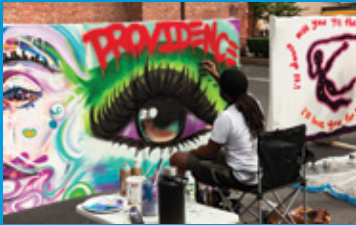


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# Construction Data Collection on an 1160-foot Crossing of the Lake Washington Ship Canal with HDD

By: Jake Andresen, MS, P.E. Staheli Trenchless Consultants  
Kimberlie Staheli, Ph.D, P.E. Staheli Trenchless Consultants

## INTRODUCTION

Seattle Public Utilities (SPU) determined that an existing cast iron force main was discharging sewage into the Shilshole Bay Waterway in 2017. This canal leads from Puget Sound to Lake Washington and is a USACE governed waterway. After bypass pumping through a temporary above-ground pipe was setup, SPU assembled a team to design a replacement pipeline installation. Horizontal Directional Drilling (HDD) was the selected method for installing the new force main beneath the ship canal which was then tied into the existing force main at new vault locations on either side of the Ship Canal. The project required a 12-inch flow diameter and the product pipe installed was a 16-inch outer diameter HDPE. The HDD construction was completed in March of 2019 and this case study discusses the data collected by the owner’s representative (Staheli Trenchless Consultants) during the drilling operation.

## CONDUCTOR CASING INSTALLATION

The HDD was installed beneath an active rail line (BTRR). To drill beneath the RR, a conductor casing was specified at the entry location, extending 10 feet on either side of the railroad tracks. Approximately 40 feet of 42-inch steel conductor casing was installed at a grade of approximately 18 degrees from the horizontal using the pipe ram method. The casing was rammed forward approximately 18 feet prior to

*“The 1200 foot pullback was completed with one mid-pull fusion weld.”*

cleaning out the soil plug. The 20-inch diameter pipe ram hammer required between 250 and 363 blows per foot of casing advancement in the first 15 feet. The production rate decreased to 1000 blows per foot at 18 feet of casing advancement at which point the spoils were removed from the first 42-inch casing section while leaving a plug at the front. During the ramming of the second 20 foot casing section, the pneumatic hammer required 330 to 1400 blows per foot to advance the casing. The stress of the ramming did

result in a split in the casing wall when the head of the casing had been advanced 25 feet. The 14-inch split section of casing was removed and ramming continued. The high blow counts required per foot matched the anticipated heavily glaciated soils.

## PILOT BORE

The pilot bore was drilled with a Universal 250x400 drilling rig with 250,000 pounds of pull and thrust capacity, using 5-inch outer diameter steel drill



Figure 1: 20-foot section of 42-inch steel casing setup on guide rails and 20-inch pneumatic hammer

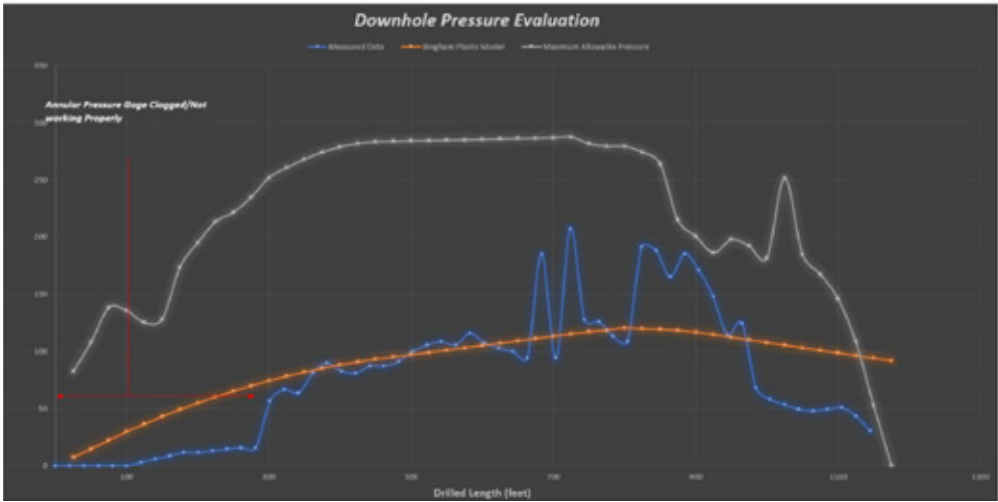


Figure 2: Pilot bore annular pressure data

pipe in lengths of 20 feet. A gyroscope was used as the guidance system to track the location of the downhole drilling assembly. Due to the design and housing requirement of the gyroscope, a 5-inch minimum diameter drill pipe was recommended by the gyroscope manufacturer. The drill bit used on the pilot bore included roller cutters in anticipation of dense to very dense glacial soils recorded in nearby geotechnical borings advanced to over 80 feet below the Shilshole Bay Waterway and within

50 feet of the alignment. The gyroscope tool contained an annular pressure gauge capable of recording the drilling fluid pressure within the bore. During the pilot bore first portion of drilling, the annular pressure gauge was malfunctioning, and the contractor experienced a loss of the drill fluid into the formation with no visible inadvertent return. After this occurrence, the contractor tripped the rods out of the hole, increased the viscosity and suspension capacity of the drilling fluid, and cleaned the housing of the annular

pressure tool. The malfunction was attributed to clay soil particles clogging the pressure sensor. The contractor then proceeded to complete the pilot bore with no further issues. In two instances, the contractor observed spikes in the annular pressure and was able to adjust the advance rate immediately to avoid further incidences of fluid loss during pilot bore drilling.

## REAMING

The contractor elected to complete two 24-inch reaming passes to create a 24-inch bore into which the 16-inch HDPE pipe would be pulled. The reamer used for the first reaming pass is shown in Figure 3.

## PIPE PULLBACK

The contractor installed the 16-inch HDPE pipe in one shift. The pullback was completed with one mid-pull fusion weld. The entire 1200 foot pipeline was not fused prior to pullback because the 1200 feet of laydown space required would have caused loss of access to many residents along W. Commodore Way (Figure 4).



Figure 3: (Left) Ream Pass 1: 24-inch reamer installed on drill string and (Right) 24-inch Short Barrel Fly Cutter



Figure 4: (Left) 16-inch HDPE pipe staged in roadway and (Right) Use of loader to manage pipe layout during pullback

Figure 5 shows the estimated and measured pull loads. It should be noted that the measured pull loads are measured at the drill rig itself and therefore include the loading applied to the drill string, leading reamer, and the product pipe.

## ACHIEVABLE BENDING RADIUS

The target alignment during the pilot bore stage included two vertical curves. The first vertical curve target was a radius of 800 feet and the second vertical curve target was 1,000 feet. During the construction of the pilot bore when constructing the first vertical curve, the guidance technician noted that 800 feet was not achievable and that the tightest curve that could be achieved was 1,000 feet. To generate a bend, the soil must have sufficient reaction strength to force the stiff steel rods into a curve. In this case, the soil was not able to maintain the attempted 800-foot bend radius curve. The alignment was adjusted and the pilot bore proceeded. This provides an opportunity for comparison to available guidelines for minimum achievable bend radius.

Note that the proposed 800 feet is greater than both of the general industry guidelines for achievable radius from these equations in Table 1 of 500 and 750 feet but was not achievable in the field, this is attributed to the soil not having sufficient ability to force the stiff steel pipe into a curve, as significant force must be applied which increases as the bend becomes tighter (increased strain energy). This force to bend the pipe is generated from a support reaction with the surrounding soil. For any given soil, there is a limiting support reaction force that can be maintained without shear failure, and therefore a limiting radius that can be achieved.

At the conclusion of this project, an as-built survey was conducted of the installed pipeline. The as-constructed radii are shown in Table 2.

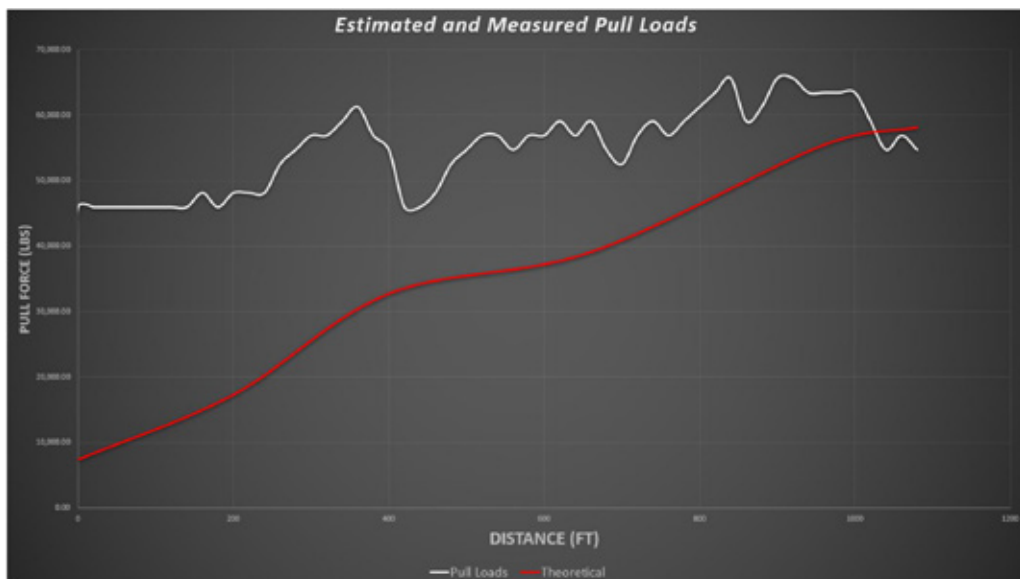


Figure 5: (Left) Estimated and measured pull loads during product pipe pullback

## PRODUCTION RATES

Staheli Trenchless performed a time/motion study during the HDD construction to document the construction activity and associated time required to achieve the project milestones. The time during which the drill phases were being completed is compared to the amount of time actually constructing the bore to develop a construction efficiency metric during each period. This data may be useful in developing schedules in the future when estimating relationship between advance rates and time required on-site. The contractor generally implemented a 10-hour workday. For each phase of the construction, the average time required to advance 1 foot only takes into account the time actually spent operating the drilling rig. The efficiency is based upon the total number of days spent on a drilling phase and accounts for downtime when the drill rig was not being operated.

The time required to complete each major phase of the drilling (pilot bore, reaming, pullback) is summarized in Table 3. †

### ABOUT THE AUTHOR:



**Jake Andresen, MS, P.E.**, is a Senior Engineer and Risk Management Inspection field lead for Staheli Trenchless Consultants (STC). He holds a Masters in Geotechnical Engineering from Arizona State University and manages STC's specialty inspection staff.

### ABOUT THE AUTHOR:



**Kimberlie Staheli, Ph.D., P.E.**, is the President of Staheli Trenchless Consultants. As a researcher with the USACE, Dr. Staheli was the author of some of the first research studies on HDD and trenchless construction mechanisms in the United States. She is passionate about using her 30 years of field and design experience to mitigate trenchless risk.



Table 1: Common equations for achievable radius with steel pipe

<i>Common Equations for Limiting Bend Radius</i>	<i>Application to 5-inch Drill Steel</i>	<i>Recommended Radius</i>
$1200 * OD_{Steel\ Pipe}$	$1200 * 5" * \frac{1ft}{12in}$	500 feet
$1800 * OD_{Drill\ Pipe}$	$1800 * 5" * \frac{1ft}{12in}$	750 feet

Table 2: Comparison of achievable radius of pipeline installed with HDD

	<i>Target</i>	<i>Pilot Bore Survey</i>	<i>HDPE Pipe As-Built Survey</i>
Vertical Curve #1	800	1000	900
Vertical Curve #2	1000	1000	1000

Table 3. Production rates and efficiency

<i>Project Phase</i>	<i>Minutes/FT</i>	<i>Efficiency</i>
Pilot Bore	1.39	0.67
Reaming	1.56	0.50
Pullback	0.33	0.64
Total	3.28	0.58



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# Geotechnical Baseline Reports for HDD: When, Why, and How

By: Michelle Macauley, PE, Macauley Expert Services  
Nick Strater, PG, Brierley Associates

## 1. INTRODUCTION

As HDD installations get longer, larger, and increasingly complicated, the cost and risk of these installations also increases. The intent of the Geotechnical Baseline Report is to identify project-specific risk and balance the risk between the Contractor performing the work and the Owner. The goal of a well-written GBR is to establish measurable baseline statements describing ground conditions and behaviors in advance of bidding and construction. As shown on the following figure, the opportunity to minimize risk is greatest earlier in the project, whereas the cost associated with change is greatest later in the project cycle.



*Risk Management over the Course of a Project*

The purpose of the baseline statements is two-fold: during the bid process the baseline statements allow the Contractors a common basis of interpretation for their bid and during construction the baseline statements provide a contractual mechanism to evaluate potential differing site condition claims. In principal, if the subsurface conditions

and ground behaviors encountered during construction are consistent with the baselines, the Contractor “owns” the condition. If specific subsurface conditions or ground behaviors exceed the baselines, and the Contractors’ means and methods, schedule, or material costs are impacted, additional compensation from the Owner may be warranted.

More and more frequently Owners are requesting Geotechnical Baseline Reports (GBR’s) for Horizontal Directional Drill (HDD) projects. However, ASCE’s Geotechnical Baseline Reports for Construction was written specifically for the tunneling industry and does not address ground conditions and behaviors that influence HDD construction. As such, challenges arise for Owners and Design Engineers when adapting the ASCE guidelines to HDD projects. Owners and Design Engineers must evaluate the ground conditions for an HDD project and develop baseline statements that are project-specific, are realistic and measurable, capture the ground conditions that impact project cost and risk, and influence project success.

Other challenges with baseline reports for HDD projects come from Owners that are unfamiliar with the purpose, process, contracting implications, and cost of a well-written GBR. Often Owners see the GBR as a report that relieves the Owner of risk; this is not the intent. A well-written GBR is a report that is written by the Design Engineer with input, collaboration, and consensus from the Owner and that ultimately becomes part of the Contract documents.

The following section will address the challenges of writing baseline statements for HDD projects, the ground conditions

*“The opportunity to minimize risk is greatest earlier in the project.”*

and behaviors to baseline, how to craft measurable baseline statements, and suggestions on how to convey this information to a Contractor pool that may be unfamiliar with the GBR concept. Example baseline (both well and poorly written) statements from previous projects are presented.

## 2. CHALLENGES OF WRITING A GBR FOR HDD PROJECTS

The GBR must not conflict with other sources of geotechnical data, and typically takes precedence over all other geotechnical-related documents, including data reports and design memoranda. This should be clearly expressed within the Contract Documents. For the baseline statements to be effective, they need to be specific to the construction process (HDD) and must be measurable during construction. Some additional challenges of writing GBRs for HDD projects are summarized below:

### **Owners that are unfamiliar with the contractual importance of GBRs:**

Owners often incorrectly assume that the purpose of the GBR is to eliminate their risk. As noted, the purpose of the

GBR is not to relieve the Owner of risk, but rather to balance this risk between the Contractor and Owner. Where overly conservative or broad baselines are applied, the bid prices can be expected to increase, but the potential for claims will decrease. To accurately represent the Owner's desired risk and budget profile, the Owner's representatives should be involved during the baseline selection process and drafting of the baseline statements.

### **Contractors that are unfamiliar with the contractual importance of GBRs:**

In many cases, small to medium-sized HDD contractors may not be familiar with the purpose, or the contractual implications of the GBR. Unless clearly presented, the GBR may be mistaken for a generic geotechnical report used to present data (e.g. a "test boring report"). A project which seeks to use a GBR should incorporate a pre-bid conference allowing review and explanation of the document and drilling subcontractors

should be required to attend. Where appropriate, a pre-contract meeting may be warranted to further discuss the baselines and the expectations for measuring the baselines.

### **Baselines for HDD projects can be challenging to measure during construction:**

Baselines that cannot be effectively measured may result in confusion and unnecessary conflict. Unlike large diameter tunneling, the HDD construction process typically does not allow direct inspection of the subsurface conditions encountered. Therefore, the Owner needs to provide full-time oversight of the HDD activities, which should be completed by field personnel experienced in HDD drilling and reaming methods. Ideally the field staff should also be familiar with the purpose and content of the GBR. In some cases, secondary subsurface explorations such as supplemental sampling and testing may be required to accurately document the conditions encountered.

### **Owners often desire a "turn-key" GBR:**

Each GBR should be specifically written for the geologic setting, the project conditions, and in concert with the Owner's risk profile. Engineers should use caution if approached by an Owner that only desires to review the GBR with the Final Design Package. Because GBRs are part of the Contract Documents and are typically higher in the contractual hierarchy than other geotechnical documents, Owners should assemble a GBR team that includes legal professionals familiar with trenchless construction. Early engagement between the Owner, legal counsel, and the trenchless engineer will ensure that the Owner is aware of the legal ramifications of the GBR and that the baselines capture the trenchless risks for the project along with the Owner's desired risk profile. Owners should be actively involved in the GBR process from inception, through development of baseline statements, to supporting construction monitoring of conditions.



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### 3. WHAT TO BASELINE?

The baseline report must be specific to the subsurface conditions and potential risk specific to the project. The risks may be identified based on previous drilling in the project vicinity, project-specific explorations and laboratory test data, and engineering geologic judgement.

Potential conditions that may warrant baselines for soil, bedrock, and groundwater are summarized in Tables 1 through 3, below. It should be noted that these are not considered comprehensive lists.

### 4. WHAT NOT TO BASELINE

In general, the following items are not baselined:

- Performance rates, such as drilling and reaming rates;
- Drill fluid components;
- Equipment size and capacity;
- Presence or length of conductor casings (this should be required as part of the design if needed);
- Drill tool types and drilling methods;
- Construction schedule;
- Direction of HDD construction; and
- Number and location of inadvertent fluid returns

### 5. HOW TO CONVEY BASELINE CONDITIONS

To better define baseline conditions, and to reduce complexity, the geologic units may be grouped into engineering units exhibiting similar behavior or responding similarly to HDD construction methods. Geologic units are often based on divisions of geologic time and on general geologic processes. Engineering units are based on how the ground (soil or bedrock) is anticipated to behave during construction. Ground that is expected to behave in a similar way may be grouped together into a single engineering unit. An example of an engineering unit comprised of soil and bedrock is provided below.

Rarely do the ground conditions along an HDD drill path remain constant for the entirety of the alignment. There are several ways to convey the variability of

Table 1 – Potential Sources of HDD Risk – Soil

Geotechnical Condition	Potential Risk
Gradation	Encountering unanticipated soil gradations (i.e. gravel instead of silt) may result in reduced penetration rates, require tool replacement, and require additional drill fluid additives.
Density and strength	Encountering unanticipated soil density and strength may impact penetration rates, steering accuracy, and the pressure confining capability, which is related to risk of inadvertent drill fluid returns (“frac-out”).
Frequency of cobbles and boulders	Cobbles and boulders may become obstructions during drilling and reaming, which may redirect the drill string, impact line and grade control, and necessitate tooling changes.
Soil Abrasivity	Encountering abrasive soils will increase tool wear, which may impact the rate of penetration and increase costs associated with tool rehabilitation and replacement.
Swelling Clays	Swelling clays may restrict borehole annulus, increasing annular pressure and risk of frac-out. They may also result in increased costs associated with larger reamer diameter, additional drill fluid additives required to stabilize swelling behavior, and time spent swabbing the borehole.
Stickiness	Encountering soils with high stickiness may result in clogged tooling (i.e. drill fluid jets and pressure probes) requiring drill string retraction and reducing penetration rate.

Table 2 – Potential Sources of HDD Risk – Bedrock

Geotechnical Condition	Potential Risk
Top of Bedrock	Encountering unanticipated bedrock may require retooling and may dramatically impact penetration rates and project costs. A variable top of bedrock may require modification of drill path to avoid unnecessary drill rod and tool stress. Variability in the bedrock surface elevation can impact the ability to achieve the desired penetration angle of the pilot hole and to effectively seat conductor casings.
Bedrock Weathering	Encountering weathered bedrock may adversely impact line and grade control and may increase tool wear.
Bedrock Strength	Encountering high-strength bedrock will reduce impact penetration rate and may exacerbate abrasive conditions.
Bedrock Abrasivity	Encountering abrasive bedrock will increase tool wear, which will impact the rate of penetration and increase costs associated with tool rehabilitation and replacement.
Rock Quality Designation (RQD)	Encountering low RQD (high natural fracture density) material may increase tool wear and the potential for drill fluid loss.
Fracture Characteristics	The orientation and aperture of fractures may adversely impact pilot hole steering, alter the behavior of reamers, and increase the potential for drill fluid loss.
Faults, Shears	Fault and shear zones may adversely impact pilot hole steering, behavior of reamers, and increase the potential for drill fluid loss.

Table 3 – Potential Sources of HDD Risk – Groundwater

Groundwater Condition	Potential Risk
Elevated groundwater pressures	Encountering elevated groundwater pressures (i.e. artesian conditions) may result in borehole collapse and drill fluid dilution.
Lack of groundwater	Lack of groundwater may result in loss of drilling fluid to the formation and require additional drilling fluid.
Unanticipated groundwater chemistry	Man-made and natural contaminants (i.e. hydrocarbon and saltwater) can adversely impact drill fluid behavior.

the ground conditions along the drill path to the Contractor. It is especially important to convey the changing ground conditions to the Contractor if the variability between ground conditions is distinct, the risks associated with the various ground conditions are different, and the changing ground conditions impact construction. The anticipated variability of anticipated ground conditions along the HDD alignment can be conveyed to the Contractor graphically (i.e. with a figure), in text, or in a table. The following table (Figure 3) is an example of one way to convey the changing ground conditions and where each unit will be encountered along the drill path. In this project the ground conditions were grouped into engineering units based on anticipated construction behavior.

Note that the example above establishes a reasonable degree of accuracy. This will vary by project depending on the degree of certainty associated with the geologic interpretation.

In many cases, baselines are shown in bold to distinguish them from general discussion and commentary. This helps to avoid conflict regarding what is and is not technically a baseline.

## 6. EXAMPLE BASELINE STATEMENTS

It is important to recognize that the selection of baseline values need not be based strictly on data but may also incorporate local experience. This allows the author to accommodate data gaps and sampling limitations. Two examples are below:

Example baseline statement developed to address boulders:

*"The HDD design borepath geometry is expected to encounter glacial till. Although not encountered by the project test borings, the glacial till deposits are known to contain boulders, which may impact HDD tooling requirements, line and grade of the bore, and penetration rates.*

***For baselines purposes the contractor shall expect to encounter a total of 10 boulders during drilling and reaming,***

### 4.1.2 Unit 2- Hard Clay

Unit 2 consists of hard, high plasticity clay and weathered shale. This unit can be excavated with a bladed drill bit. This unit will exhibit swelling characteristics, will require special drilling mud additives to control the swelling ground during drilling and may require oversizing the maximum reamer diameter to accommodate the swelling soils.

***For baseline purposes the Plasticity Index (PI) is 40, the percent fines is 95%, and the swell potential is very high.***

Figure 2. Example of an Engineering Unit

***For baseline purposes, the alignment will be drilling through the following:***

Unit	Begin Station	End Station
<b>Unit 1</b>	<b>0+00</b>	<b>2+80</b>
<b>Unit 2</b>	<b>2+80</b>	<b>3+15</b>
<b>Unit 3</b>	<b>3+15</b>	<b>3+50</b>
<b>Unit 4</b>	<b>3+50</b>	<b>10+75</b>
<b>Unit 3</b>	<b>10+75</b>	<b>10+90</b>
<b>Unit 2</b>	<b>10+90</b>	<b>11+40</b>
<b>Unit 1</b>	<b>11+40</b>	<b>12+90</b>

***\*\*\* Note: Stationing should be considered accurate +/- 25-feet***

Figure 3. Conveying Variability of Conditions Along an Alignment Using a Table

***ranging in size from 12 inches to 5 feet, as measured along the pilot-hole axis. Individual boulders shall be documented based on drilling behavior and shall be measured once per individual occurrence."***

Example baseline statement developed to expand on a limited rock strength data set:

*"The HDD design borepath will encounter bedrock consisting of gneiss. Limited laboratory testing completed for the project resulted in unconfined compression strength values ranging from 8,000 to 20,000 psi. Local experience suggests that stronger bedrock may be present. For baseline purposes, the contractor shall expect*



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to encounter gneissic bedrock having an average compressive strength of 18,000 psi, and a maximum compressive strength of 30,000 psi.”

Baseline statements should be concise and precise. While many baseline values are based on laboratory test results, an effective baseline statement does not provide a wide range of possible values. For example, consider the following statement:

*“Standard Penetration Test (SPT) values ranged from 0 blows per foot (bpf) to 50/4 inches bpf, with an average of 11 bpf. For baseline conditions, SPT values are expected to range from 0 bpf to refusal.”*

This statement does not provide the Contractor with definitive information. An improvement may be to baseline the durability of the unit (i.e. medium durability) or separate the single geologic unit into engineering units that can be baselined with a narrow range.

Additionally, baseline statements should not over-rely on statistical analysis of laboratory test results, but rather provide a narrow range of values or (preferably) a single value. The following table (Figure 4) is an example where simplification would be appropriate to convey a compressive baseline strength.

## 7. EXAMPLE GBR OUTLINE FOR HDD CONSTRUCTION

While every GBR will be project specific, there are commonalities to each report. Below is an example outline from a previous GBR.

### 1. Introduction

- 1.1 Project Information
- 1.2 Purpose and Organization
- 1.3 Limitations

### 2. Project Overview

### 3. Geologic Data and Project Setting

- 3.1 Data Sources
- 3.2 Regional Setting and Geology

### 4. Ground Characterization and Groundwater Conditions

- 4.1 Engineering Units
- 4.2 Groundwater Conditions
- 4.3 Engineering Units Along Alignment

	Density pcf	Compressive Strength psi
Number of samples	12	12
Mean	139	3458
Median	138	3438
High	152	5839
Low	129	1497
Baseline Average	142	3438

Figure 4. Example of an Overly Statistical Baseline Table

### 1.1 Limitations

This GBR presents baselines with respect to certain subsurface conditions that are expected to be encountered during construction that may influence the contractor’s rate of progress, tooling selection, tool wear, or approach to bidding the project. The GBR establishes a contractual basis for allocation of geotechnical risk during performance of the work; it does not define the single correct interpretation of geotechnical conditions. This baseline report represents the conditions the contractor should assume for bidding purposes and for which the contractor is responsible for during construction.

Figure 5. Example of a Limitations Clause

## 5. Design and Construction

### Considerations

Similar to the recommendations in the ASCE guidelines for GBRs, it is important to clearly state the hierarchy of the GBR relative to the GDR and other Contract documents. Often it is also important that the limitations section does not attempt to unduly limit the scope of the GBR or caution the contractor from relying on the information in the GBR. An example limitations clause is provided below in Figure 5.

## 8. CONCLUSIONS

GBRs for Horizontal Directional Drilling projects should be project

specific, concise, and clearly convey the key geotechnical risks that may impact HDD construction. Owners should be involved throughout the GBR process and fully understand the contractual obligations of their GBR. A well written GBR provides baseline statements that are understandable, measurable, and defensible. †

## 9. REFERENCES

ASCE Geotechnical Baseline Reports for Construction: Suggested Guidelines, 2007, Edited by Randall J. Essex., 72 pp. Tunnelingonline.com, What’s the Deal with GBR’s, Contributing Author, June 6, 2019

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