PACIFIC NORTHWEST CHAPTER

Pacific Northwest Trenchless Review

2019

Innovation in the Pacific Northwest

INSIDE:

- Moving Forward with Pipe Ramming
- Relocating a Sewer Under a Highway
- Inadvertent Returns of Drilling Fluids
- Details on NASTT Events and More!

MARCH 18, 2019 | 5:30-7:30PM | CHICAGO, ILLINOIS

In conjunction with NASTT's No-Dig Show

2019 NASTT SPEAKEASY

EDUCATIONAL FUND AUCTION

NASTT'S 18TH ANNUAL EDUCATIONAL FUND SPEAKEASY AUCTION & RECEPTION

Join us in a Chicago Speakeasy! The Annual Educational Fund Auction helps raise money for very worthy causes. Since 2002, **NASTT has raised nearly $1.1 Million** and used those funds in support of our many educational initiatives. Due to your generosity, NASTT is able to provide targeted trenchless training courses to the industry, publish trenchless resources manuals and sponsor university students' attendance at NASTT's No-Dig Shows, as well as award scholarships.

EXCITING AUCTION ITEMS

Come to the auction and bid on great items like trips, tickets, electronics, industry items and more!

HAWAIIAN VACATION RAFFLE

The winner of this raffle will receive a dream Hawaiian vacation, a $5,000 value! Tickets are $25 or five for $100 with a maximum of 1,000 tickets being sold.

Raffle generously sponsored by Vermeer

COSTUME CONTEST

Show us your style! Speakeasy style that is! Come dressed in 1920s gangster or flapper attire at the auction's Eighth Annual Costume Contest! Prizes will be awarded- don't miss out!

50/50 RAFFLE

A great way to win some cash for yourself and help out our student chapters! The winning ticket will be drawn immediately following the live auction and you must be present to win. The winner splits the cash pot with the students.

Auction Reception sponsored by

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Bennett trenchless engineers

FOR MORE INFORMATION VISIT NASTT.ORG/NO-DIG-SHOW/AUCTION

Pacific Northwest Trenchless Review

WINTER 2019

Featured...

Alderwood

Advances in pipe ramming are largely led by innovations in the field, as engineers stretch the limits

08

Connector

Sewer main relocation allows WSDOT to move forward with a big new freeway interchange project

15

Returns

A commonly accepted method for predicting IRs has lagged behind other design calculations for HDD

19

Also...

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Growing with the Industry

Brendan V. O'Sullivan - *Chair, NASTT PNW Chapter*

Welcome to the eighth edition of the Pacific Northwest Trenchless Review! With the number and types of trenchless projects increasing in scope and complexity, the future is bright for the trenchless industry. The North American Society of Trenchless Technology (NASTT) is at the forefront of trenchless education, and the Pacific Northwest Chapter (PNW-NASTT) is here to further the mission to "advance trenchless technology and to promote its benefits for the public and the natural environment by increasing awareness and knowledge through technical information dissemination, research and development, education and training" throughout Alaska, Idaho, Oregon, and Washington.

Looking back… PNW-NASTT kicked the year 2018 off by hosting a Cured-in-Place Pipe (CIPP) Best Practices Short Course in Anchorage, Alaska. It was a great success! We had 32 attendees from the municipal and consultant sectors. I want to personally thank Brian Gastrock for his hard work as our boots on the ground for this event coordination.

We had great turnout for our Annual General Meeting at the 2018 No-Dig Show in Palm Springs, California, and hope to see more faces from the PNW-NASTT this coming March at NASTT's 2019 No-Dig Show in Chicago/Rosemont, Illinois. We will be holding a chapter meeting at 4 p.m. on Sunday, March 17, at the Donald E. Stephens Convention Center prior to the Conference Kick-off, so mark your calendars now.

Looking to the future, the PNW-NASTT kicked off the new year with our biennial Trenchless Symposium and Short Course at the World Trade Center Portland in Portland, Oregon, on January 16th and 17th.

Our chapter plans to increase educational opportunities throughout our geographic area and bring more exciting NASTT best practices courses to the region. We are also exploring other educational opportunities, be it webinars, lunch & learns, or presentation/networking events.

As the trenchless industry continues to grow, existing techniques are refined, and new techniques are developed, it will be crucial to continue the NASTT mission. This can only be accomplished with the involvement of those in the industry, and we are looking forward to welcoming new members to our chapter as we move into 2019! If you want to get involved, please contact me at Brendan.O'Sullivan@murraysmith.us or (503) 225-9010.

Regards,
Brendan V. O'Sullivan
Chair, NASTT PNW Chapter

Congratulations, Mr. O'Sullivan

PTR Communications extends its congratulations to the Pacific Northwest Chapter's Brendan O'Sullivan for winning the 2019 Ralston Young Trenchless Achievement Award, which recognizes young individuals within NASTT who have demonstrated excellence in trenchless technology.

A Senior Engineer at Murraysmith in Portland, Brendan has served as Chair of NASTT's PNW Chapter. (As of January, he is Immediate Past Chair, with Carl Pitzer taking the position of Chair.)

Brendan will be presented with the award at the 2019 No-Dig Show's Gala Awards Dinner, March 19 in Chicago.

No-Dig Show Ahead

Frank Firsching - *NASTT Chair*

Hello, Pacific Northwest Chapter members! As 2018 closes, I am pleased with the progress we've made in our organization during my term as Chair of the Board of Directors. We are stronger than ever, and that is due in large part to our Regional Chapters and dedicated members.

The 2018 No-Dig Show in Palm Springs was very successful on all accounts. The exhibit hall featured close to 190 exhibitors, and we welcomed over 2,000 attendees from all over the world who came to experience the world-class technical sessions and networking events that our Show is known for. The Educational Fund Auction was, once again, the trenchless social event of the year, raising nearly $100,000 for our educational programs. Thank you all for your generous support.

Plans are under way for NASTT's 2019 No-Dig Show, which will be held March 17-21 in Chicago. We hope you will make plans to join us.

"Plans are under way for NASTT's 2019 No-Dig Show, which will be held March 17-21 in Chicago. We hope you will make plans to join us."

Our No-Dig Show Program Committee members volunteered their time and industry knowledge to peer-review the 2019 abstracts. These committee members ensure that the technical presentations are up to the standards we are known for. Thank you to the Pacific Northwest members who have volunteered for this important task this year, including Dan Buonadonna, Jack Burnam, Michelle Macauley, Kimberlie Staheli and Diana Worthen.

The Pacific Northwest Chapter is also home to some of our Track Leaders. Track Leaders are Program Committee members that have the added responsibility of managing a track of the technical program and working with the authors and presenters to facilitate excellent presentations. I would like to extend a special thank-you to the Chapter members that will also serve as Track Leaders in 2019: Jack Burnam and Kimberlie Staheli.

The North American Society for Trenchless Technology is a society for trenchless professionals. Our goal is to provide innovative and beneficial initiatives to our members. To do that, we need the involvement and feedback from our professional peers. If you are interested in more information, please visit our website at nastt.org/volunteer.

Our continued growth relies on the grassroots involvement of our regional chapter advocates. Thank you again for your support and dedication to NASTT and the trenchless technology industry.

Regards,
Frank Firsching
NASTT Chair

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Upcoming NASTT Conferences, Courses and Events

March 17

NASTT's Introduction to Trenchless Technology – Rehabilitation
Donald E. Stephens Convention Center
Chicago, Illinois

March 17

NASTT's Introduction to Trenchless Technology – New Installations
Donald E. Stephens Convention Center
Chicago, Illinois

March 17-21

NASTT's 2019 No-Dig Show
Donald E. Stephens Convention Center
Chicago, Illinois

March 20

NASTT's Gas Good Practices Course
Donald E. Stephens Convention Center
Chicago, Illinois

March 20-21

NASTT's Pipe Bursting Good Practices Course
Donald E. Stephens Convention Center
Chicago, Illinois

March 20-21

NASTT's New Installation Methods Good Practices Course
Donald E. Stephens Convention Center
Chicago, Illinois

March 20-21

NASTT's CIPP Good Practices Course
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March 20-21

NASTT's Laterals Good Practices Course
Donald E. Stephens Convention Center
Chicago, Illinois

March 20-21

NASTT's HDD Good Practices Course
Donald E. Stephens Convention Center
Chicago, Illinois

May 7-9

Trenchless Technology Road Show
Sheraton Vancouver Airport Hotel
Richmond, B.C., Canada
Information: trenchlessroadshow.ca

September 26

World Trenchless Day
Information: worldtrenchlessday.org

October 28-30

No-Dig North 2019
Telus Convention Centre
Calgary, Alberta, Canada

For more information and the latest course offerings, visit:
nastt.org/training/events

PIPE RAMMING:
Understanding the Forces that Drive the Industry Forward

Kimberlie Staheli, Ph.D., P.E.
Staheli Trenchless Consultants

Armin Stuedlein, Ph.D., P.E.
Oregon State University

Paul Richart, P.E.
Alderwood Water and Wastewater District

The limits of pipe ramming technology are constantly being stretched. Rams are increasingly installed in longer lengths, in more aggressive and/or dense geotechnical conditions, below the water table, and to installation tolerances that allow gravity pipelines within the rammed casings. Advances are largely led by innovations in the field, with engineers striving to develop design parameters that model the performance of the installations. To understand the mechanisms controlling ramming and provide construction inspectors with information during construction, three extensive pipe ramming projects were instrumented during construction. The instrumentation, consisting of strain gages and accelerometers, was placed on pipe rams ranging from 36- to 84- inches in diameter. Field data was collected throughout the ramming. Detailed field notes allowed correlation of all construction activities with the data. The full bank of data remains under analysis. This article presents the details of two 84-inch pipe ram installations constructed from the same shaft.

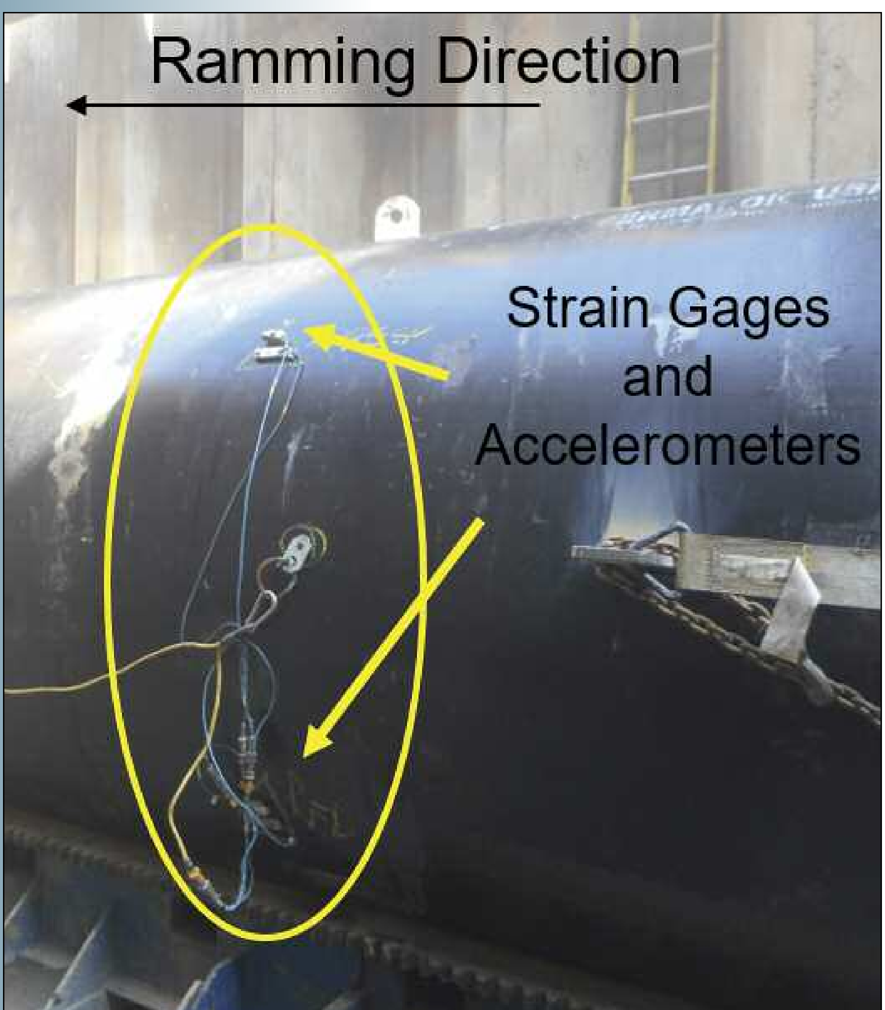


Figure 1. 84-inch outer-diameter pipe showing position of strain gages and accelerometers that were monitored during ramming operations.

CASE HISTORY AND GEOTECHNICAL CONDITIONS

The Alderwood Water and Wastewater District (AWWD) needed a 24-inch water force main and a 30-inch gravity sewer line. These pipes were to be constructed within a single casing. One 84-inch casing would go beneath State Route (SR) 527, and another beneath Silver Creek. The SR 527 crossing was 145 feet long and the Silver Creek Crossing was 35 feet long.

Significant geotechnical information was available for the design. In addition, there were construction records and reports from previous projects.

There were three soil units at the site: 1) recent fill deposits; 2) alluvium and recessional outwash deposits (post-glacial deposits); and 3) dense glacially overridden deposits. The trenchless crossings were all within the post-glacial deposits and dense glacially overridden deposits. These units were described as follows:

Figure 2. (A) Cutting shoe is attached to lead pipe section. (B) Top View – welding lubrication line in the shaft. (C) Close-up of lower section of shoe during fabrication and insertion of bullet bits.

Figure 3. (A) Grundoram Taurus pneumatic hammer. (B) Hammer Sled to hold hammer to design grade.

- Alluvium and recessional outwash deposits—"…generally silt, sand and gravel varying from medium dense to very dense, generally increasing in density with depth. These soils will exhibit moderate to high strength." (GeoEngineers, Geotechnical Design Summary Report, August, 2011a).
- Dense Glacial Deposits— "…primarily sand with varied amounts of silt and gravel that are generally very dense in consistency. These soils will exhibit high strength and generally low permeability." (GeoEngineers, Geotechnical Design Summary Report, August 2011a).

TRENCHLESS DESIGN

Many design parameters were included in specifications to manage risk. One of these was design of a temporary pipe plug to counterbalance groundwater pressure. Requirements were also established for the cutting shoe, setting limits on the diameter to thickness ratio. (Price and Staheli, 2013). The most valuable requirement was providing real-time instrumentation on the pipe ram. The contractor had to hire an instrumentation specialist that met requisite qualifications to perform instrumentation monitoring during the pipe ram. The specification outlined how each pipe was to be fitted with strain gages and accelerometers. Each pipe was fitted with four pairs of strain gages and accelerometers on the outside at locations that were 90 degrees apart and located one pipe diameter from the hammer-casing interface. Signals were collected during ramming and processed with Pile Driving Analyzer® (PDA) manufactured by Pile Dynamics, Inc. Figure 1 shows the instrumentation mounted on the pipe during construction.

DETAILS OF THE RAM

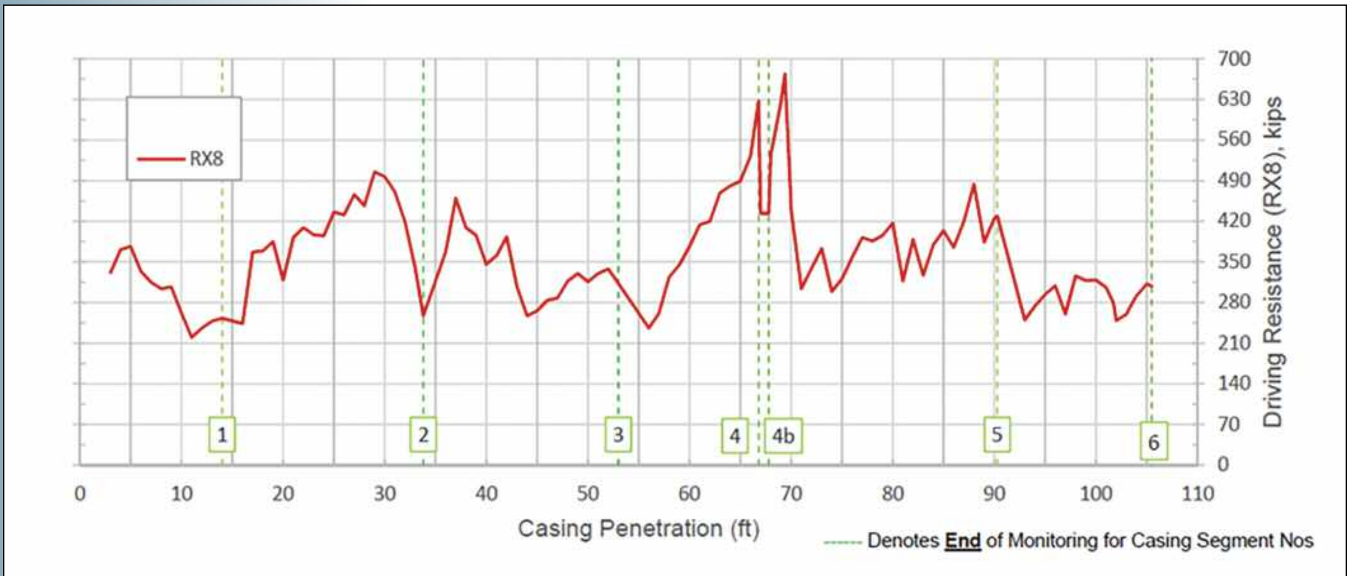
The contractor used a heavily reinforced cutting shoe that contained bullet bits. The specification required the cutting shoe design to meet a thickness ratio (d/t) of 30 (minimum). The pipe had an outer diameter of 84 inches and a 1.25-inch-diameter wall. The 4-foot cutting shoe was attached to the lead casing as shown in Figure 2, as was used for SR527 crossing.

The pipe ramming hammer used on the project was a Grundoram Taurus pneumatic hammer manufactured by TT Technologies. During pipe ramming, the hammer was supported by a sled that moved on pipe rails surveyed to design line and grade, as required by specification. The sled for holding the hammer is shown in Figure 3.

RAMMING ANALYSIS BENEATH SR 527 RAM

Six pipe segments, each 20 feet in length, were rammed beneath SR527. Parameters measured during ramming included:

- Penetration Resistance – blows/foot
- Average Hammer Operation – blows/minute
- Average Transfer Energy – kip-ft
- Average Transfer Efficiency - % (Based on the rating of the hammer)
- Average Compressive Stress – ksi



Casing Penetration v. Driving Resistance and Hammer Operation (RMDT, 2016)

- Ramming Duration – hh;mm;ss per-20 foot pipe

The Driving Analyzer® estimated the ultimate driving resistance based on the Case Method "RX8" quantity computed by the PDA (Goble et al. 1975; Rausche et al. 1985; Meskele and Stuedlein 2015a, 2015b). Figure 4 shows the ultimate driving resistance for the ram beneath SR 527.

During installation of the first segment, the hammer was operated slowly as the shoe was penetrating a grout column that had been constructed to seal the shaft during launch. While ramming through the grout, rebound of the cutting shoe was observed due to lack of reaction force that would be provided once the pipe was buried. The force shown by the PDA measurements at approximately 5 feet of penetration was primarily due to resistance at the face and the rebound. Once the pipe advanced to an embedment of 11 feet, the jet grout columns and engineered pipe plug were fully penetrated into the soil, and the force reduced from 375 to 225 kips. The force then increased linearly, indicative of an increase in frictional resistance along the circumference of the casing (both inner and outer diameter), consistent with the findings of previous research. (Stuedlein and Meskele 2013; Meskele and Stuedlein 2015b). Ramming of the first casing segment was completed at a penetration distance of 14 feet, leaving 5 feet of casing within the shaft.

> "AFTER THE RAM MOVED PAST THE BOULDER, THE PENETRATION RESISTANCE DROPPED MARKEDLY, AND RAMMING SPEEDS INCREASED."

Ramming began on the second casing and the pipe advanced approximately 2 feet when ramming force markedly increased with little forward penetration. Advance rates measured 3/8-inch per minute. The operator had concerns about over-excavation because the second casing was filling with soil with no advance. In addition, significant rebound was observed at this location. The ramming behavior was indicative of an obstruction on the leading edge, such as a large boulder. The contractor attached a come-along to each side of the casing at spring-line from the front wall of the shaft to the end of the casing to apply compressive force on the casing and eliminate rebound. This was done to improve energy transfer (Meskele and Stuedlein 2013; 2015a) to the leading edge of the casing. Once the come-along was tensioned, ramming continued for less than one minute when the pipe "jumped" forward over the next two feet. The penetration resistance decreased by 70 kips with an instantaneous decrease in resistance at the face. When the muck was excavated recovered, a fractured large boulder, approximately 17-inches in diameter was removed from the pipe. It is likely that this boulder caused the stalled casing.

During Segment 2, after the ram moved past the boulder, the penetration resistance dropped markedly, and ramming speeds increased. At completion of ramming this segment, surface settlement was noted. It is likely the face of the ram lost stability while ramming without forward movement and allowing soil into the pipe. The instrumenta-

> "At the end of Segment 3, 50 feet of soil was removed to lower penetration resistance."

tion was able to document a large drop in face resistance immediately after the pipe moved forward. Resulting settlement was significant approximately 20 feet from the shaft where a sink-hole with a depth of 40 inches Luckily, the settlement had arrested prior to ramming beneath SR 527. Figure 5 presents the penetration resistance versus penetration depth while ramming Segment 2, tracking events that occurred. Clearly, the observations made during ramming are correlated to the penetration resistance inferred from the dynamic analyses, similar to those demonstrated by Meskele and Stuedlein (2015a).

Segment 3 was rammed without incident. The only change was the introduction of lubrication to the outside of the bore. The hammer resistance was uniform and significantly less than the first two pipes. This was due to lower face resistance because the soil became less dense soil at the face. In addition, it is possible the friction coefficient decreased due to the less dense soil. From 36 to 43 feet, the penetration resistance decreased by 125 kips, even though the surface area over which the friction acted increased by one-fourth. This indicates a large reduction in face resistance. From 44 to 53 feet, the ramming resistance increased 53 kips which is indicative of a steady increase in frictional force on the inside and outside surface of the pipe. The normalized friction over this lubricated section was 0.07 tons per square foot of surface area.

At the end of Segment 3, 50 feet of soil was removed to lower penetration resistance. A plug remained in the pipe to counterbalance groundwater. Once removed, observations revealed a distinctly layered soil system. This was consistent with the geotechnical report, showing the dense glacial soils on the bottom of the pipe and the alluvial/recessional soils on the top of the pipe. Face forces on the ram likely decreased when the ram broke into the alluvial/recessional deposits which exhibited a lower density than the dense glacial soils.

The data from Segment 4 revealed the most interesting trend on the project. During Segment 4, forces increased substantially at a penetration depth between 56-65 feet and ramming rates increased from 3.5 to 14 minutes per foot. The data reflected significant resistance at the front of the pipe. In addition, the relative efficiency of the hammer dropped over 50% in 4 feet of ramming. Once soil was removed, observations within

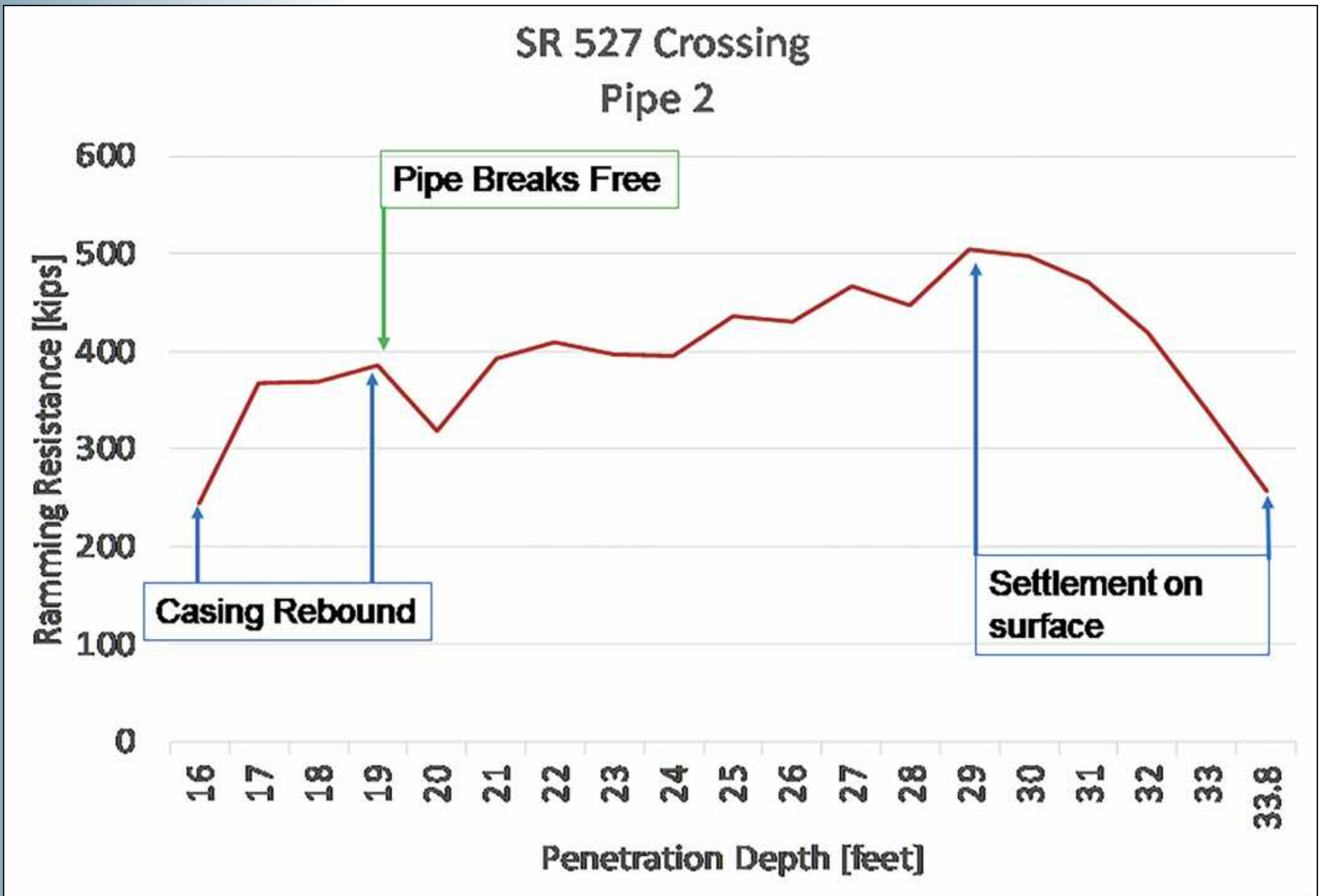


Figure 5. Casing Penetration Depth v. Ramming Resistance and events while ramming Segment 2 on the Highway 527 Crossing

the casing showed the front of the casing moving to the left. Measurements indicated the leading edge of the pipe was 24-inches south of the design alignment; however, grade was within specified tolerance.

At the beginning of Segment 5, the pipe would not move and the longitudinal weld on the casing split for a distance of approximately 6 feet from within the shaft. The weld was fixed, and ramming resumed. Ramming forces continued to be high and the hammer began jumping at the collets. The adapter piece between the hammer and collets was removed and friction pads were welded to the adapter to allow the hammer to seat into the collets. High penetration resistance continued for 2 feet when the pipe "broke free" of an apparent obstruction and penetration resistance decreased from 680 to 300 kips.

Soil was removed prior to ramming Segment 6. Soil within the pipe was sand, gravel, cobbles, and occasional boulders. Soil layering was no longer observed. One boulder was recovered that measured 15-by-19-inches. Survey indicated a bend in the pipe starting at Segment 4; however, the bend did not continue through Segment 5 and the alignment continued straight (at the tangent to the angle) after the bend. The trajectory resulted in the ram ending significantly off line; however, grade was still maintained. Therefore, the ram under SR527 continued.

Ramming Segments 5 and 6 were fairly consistent and non-eventful with the exception of the hammer-pipe interface overheating. The temperature measured at the hammer/pipe interface rose as high as 200 degrees Fahrenheit, when ramming stopped to allow the hammer to cool. Final survey indicated a deviation of 54-inches south of design alignment requiring a new manhole. Although not ideal, the ram was able to successfully carry the on-grade sewer within specified tolerance.

RAMMING BENEATH SILVER CREEK

The equipment was turned 180 degrees within the shaft and the ram beneath Silver Creek was initiated. This ram was short, requiring three 20-foot segments of casing. Sand bags were used to create an artificial plug. More concern was placed on groundwater since this fish-bearing stream could not be dewatered. Figure 6 shows the variation of driving resistance with penetration depth of the casing.

Driving resistance curves for SR527 and Silver Creek crossings have similarities that deserve attention. The first Segment on Silver Creek shows low casing penetration with significant rebound due to the lack of

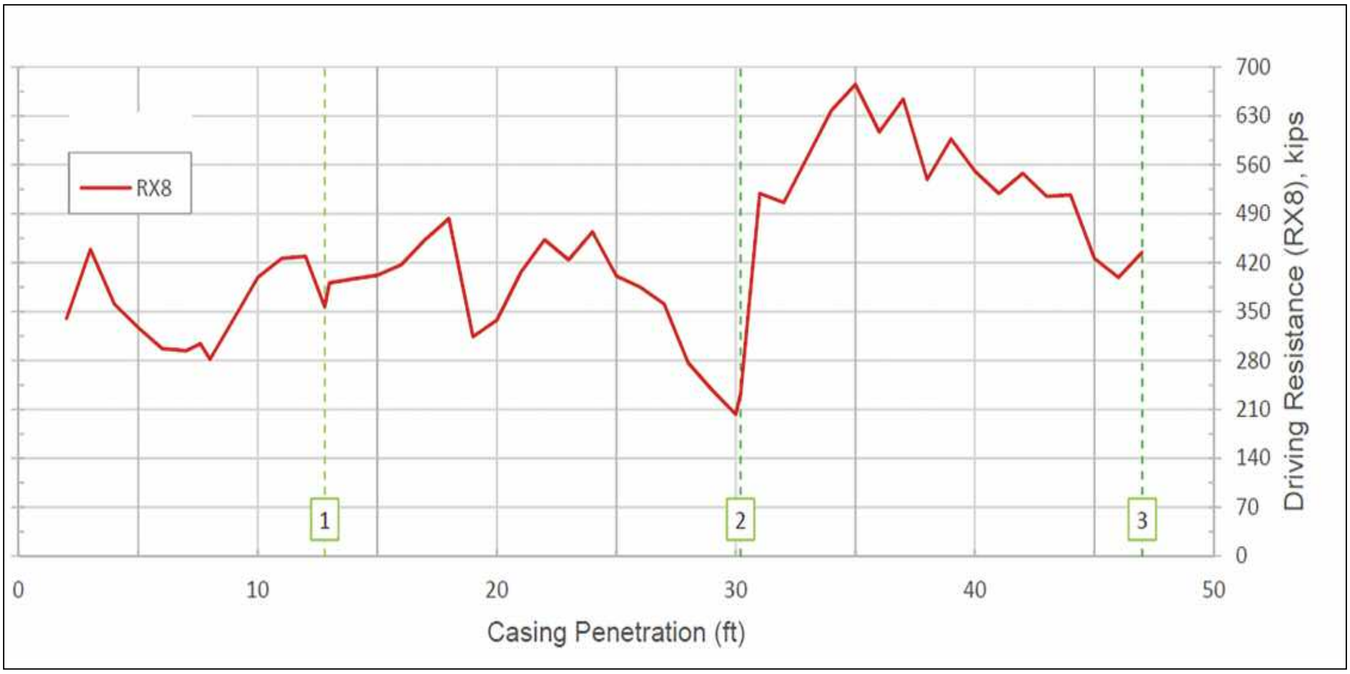


Figure 6. Casing Penetration v. Driving Resistance for the Silver Creek Crossing (RMDT, August, 2016)

friction. Both drives had high amount of face resistance at launch, followed by a reduction and corresponding slow rate of increase as the friction along the inner and outer surface of the pipe accumulate with increasing penetration length.

A strong correlation between the two drives occurred during the installation of Segment 3 when deviations in line occurred and was observed upon removal of soil. During the deviation in line, the penetration resistance increased from 210 kips to 660 kips – similar to the SR527 crossing. The increase in resistance occurs due to the increased normal stresses (and therefore interface friction) acting on the pipe circumference over a length of approximately 10 feet (over which the casing bends). The increase in resistance was followed by an overall decrease, as the pipe continued forward. The load required to bend the pipe was no longer realized. Figure 12 shows an overlay of the penetration resistance of the two rams.

CONCLUSIONS

The data acquired during these pipe rams is invaluable to determine the mechanisms that govern pipe ramming behavior. Penetration resistance was addressed in this article; however, there are many other analysis to be completed that will increase the factors that govern pipe ramming behavior. This analysis clearly showed that the force on the cutting shoe is a very significant component of the total driving resistance and changes in soil conditions during ramming results in increases or decreases in resistance as these loads change. Realizing the forces at the front of the ram make up a significant component of the total ramming resistance is a departure from current thinking within the industry which largely focusses on friction as the primary component of the driving resistance. When forming this conclusion, it should be noted that these conclusions were based on test sites with soils that were highly dense and contained gravel and cobbles with high unconfined compressive strength. It will be necessary to instrument pipe rams in soils with lower density to determine the relative component of the force on the cutting shoe compared to the frictional component of the resistance.

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Groundwater Impacts on Sewer Relocation Using Trenchless Technologies Near and Under I-105

Erik Waligorski
Carollo Engineering

Dave Christensen & Mike Benoit
City of Renton

The City of Renton recently constructed a sewer main relocation project that allows the Washington State Department of Transportation (WSDOT) to move forward with a new $116-million freeway interchange project connecting the High Occupancy Vehicle (HOV) lanes between Interstate 405 (I-405) and State Route 167 (SR 167), one of the most heavily congested interchanges in the state.

To construct this new freeway, the existing sewer main had to be relocated farther south on Talbot Hill, a southeast neighborhood of the City, and a new sewer crossing had to be constructed underneath

I-405. Given certain geographical and infrastructural obstructions, the City decided to relocate and construct both sewers using trenchless technologies instead of traditional open-cut construction.

This two-part project ultimately served as an important learning experience for both the design team and the contractor, specifically because they were repeatedly challenged by the presence of groundwater within the project site and its degrading effects on the surrounding soil. At one point, the contractor's equipment became flooded with water; at another, the contractor risked losing the face of the excavation and creating a void under I-405. However, through careful designing and productive use of available trenchless technologies, both installations overcame each obstacle and were successfully completed on time.

This article reviews the obstacles faced and lessons learned through this project and offers valuable insight to similar projects that may be completed in the future.

THE DIRECT CONNECTOR PROJECT

The greater Seattle area is one of the fastest developing regions in the United States. To better accommodate growing populations, WSDOT strives to continue improving its freeway systems that connect residents and travelers to local communities.

In 1999, WSDOT introduced the I-405/SR 167 Direct Connector Project, a 15-year community partnership dedicated to alleviating a particularly problematic commute on I-405 between the City of Renton and the City of Bellevue at the SR 167 interchange. Enduring one of the worst commutes in the state, drivers and transit riders could expect traffic on the I-405 to last up to eight hours a day. To improve traffic flow and safety, the I-405/SR 167 Direct Connector Project unveiled plans to construct a new flyover ramp connecting the existing HOV lanes on I-405 with the HOV lanes on SR 167.

NEED FOR TRENCHLESS TECHNOLOGIES

The Project required extensive construction, first to relocate the remaining freeway travel lanes further south and then to cut a considerable slope out of Talbot Hill, namely South 14th Street, to make way for the new infrastructure. However, the partial removal of Talbot Hill would have exposed the City of Renton's gravity sewer lines in South 14th Street and under I-405 at Shattuck Avenue S. This meant that these sewer lines also needed to be shifted to follow the new cut slope.

To relocate the existing gravity sewer lines, the City of Renton had to complete two major undertakings: 1) construct a new gravity sewer through Talbot Hill at depths ranging from 10 feet to over 50 feet below ground surface, and 2) install a new gravity sewer crossing beneath I-405. However, given Talbot Hill's significant depth and the impeding presence of the I-405, both areas required trenchless alternatives to open-cut construction. After evaluating several different construction methods, the City of Renton used the following trenchless technologies in practice: horizontal directional drilling (HDD) beneath Talbot Hill's relocated South 14th Street and guided auger boring under the new I-405 crossing.

While these technologies were ideal for the project's needs and locations, existing soil conditions and the presence of localized perched groundwater posed significant challenges to construction on both sites.

SOIL AND GROUNDWATER CONDITIONS

As part of the preliminary design of the direct connector project, WSDOT conducted extensive geotechnical explorations of the project area. According to those explorations, the existing soil was largely made up of sands and silts that consisted of artificial fills, wetland deposits, and Vashon Recessional Outwash to a depth ranging from 10 feet to 40 feet below the ground surface. Beneath the sand and silt layer is the Renton Formation, a bedrock layer made of sandstone that, after compression testing of core samples, was determined to be "very weak rock."

The geotechnical evaluation also confirmed the presence of groundwater below the proposed sewer alignment at depths 10 to 35 feet below the ground surface. Localized perched groundwater could also be encountered above these levels.

The combination of the sensitive silty-sand formation and groundwater not only required contractors to make adjustments during construction, but to also take preventative measures against losing the face of the excavation and creating sinkholes under I-405.

GRAVITY SEWER HDD CONSTRUCTION

The final design for the HDD installation proposed an insertion point near the intersection of Morris Avenue S and the new South 14th Street, creating a 2.5 percent design slope that would provide the necessary fall for a gravity sewer installation.

However, during this final design, several issues arose:

1. The project location did not allow for adequate laydown area for the HDD pipe fusing and pullback at the exit point of the HDD. As a result, the contractor needed to drill uphill from the east end of the project and then relocate or install a second HDD drill at the exit point to complete the pull back.

2. As the exit point was being finalized on the west end, the design team became aware of an existing fiber optic line that would conflict with the exit point of the HDD. Rather than risk damaging the expensive fiber optic line, the exit point of the HDD was shifted to the east approximately 85 feet and excavated to the depth of the new manhole. Although this change was unexpected, the redesign actually reduced the length of the HDD installation, shortening the laydown area required for fusing and installation.

Equipment became flooded with water at one point in the Talbot Hill project.

Construction also required alterations when the contractor's HDD drill pierced a perched water layer near the intersection of Whitworth Avenue and the bore path. Water from this layer flowed continuously during the pilot tube's installation, reaming, and pipe pull back; as a result, the contractor's drill rig was inundated with water during drilling, causing the excavation around it to flood.

The water did not become a significant problem, however, until it was time to install the new gravity sewer manhole at Whitworth Avenue. The perched water layer raised the water pressure so much that the contractor had trouble installing the grout to seal the annular space around the HDD pipe. To combat this situation, the contractor built a watertight bulkhead capable of withstanding the water pressure and successfully completed construction for Talbot Hill.

GUIDED AUGER BORING

The final design for this portion of the project initially identified pilot-tube pipe ramming as the most viable option for crossing underneath I-405. The design team made this decision carefully considering the project area's geologic and hydrogeologic conditions and the associated risks:

- Losing the soil face due to the presence of groundwater and causing voids under the freeway.
- Being unable to establish an adequate soil plug.
- Facing obstructions such as cobbles and boulders.

However, during the submittal review phase, the prime contractor's crossing subcontractor submitted a request for information

(RFI), which requested to complete the sewer crossing under I-405 using McLaughlin Boring Systems' ON TARGET auger boring steering system instead of pilot-tube pipe ramming. This new system would allow the subcontractor to monitor the line and grade of the auger bore installation using twin-line projection halogen lights enclosed in the steering head and to make horizontal and vertical adjustments using hydraulic plates that expanded from the side of the steering head.

After reviewing the RFI, the City and the City's consultants accepted the newly proposed method, so long as the contractor addressed the risks. The contractor submitted a work plan and contingency plan, and then began constructing the sewer crossing in August of 2016.

Almost immediately upon exiting the shaft on I-405's north side, the contractor began pulling cobbles and small boulders from the auger. Then, at approximately 63 feet from the entry shaft, the auger encountered a large boulder that encompassed the entire face of the 36-inch diameter steel casing.

Note, that, had the contractor used pilot-tube pipe ramming, the pilot tube would not have been able to penetrate the rock. As a result, the City would've had to reevaluate and move the location of the freeway crossing or excavate the rock hoping it'd be the only one encountered. Instead, by using the guided auger bore, the contractor was able to chip and micro-blast the rock in just over two and a half weeks and advance the casing.

As the installation proceeded, the auger machine began to encounter some groundwater, which would dissipate as the casing advanced. However, once the casing advanced roughly 100 feet from the entry shaft, the soil removed from within the casing became consistently wet. Then, at approximately 137 feet from the entry shaft, the contractor began experiencing flowing sand at the face of the casing, raising concerns that the contractor was losing the face of the excavation and creating a void under I-405.

Taking preventative measures, the contractor repeatedly stopped the casing installation and pumped a cement/bentonite grout mix into any potential voids to prevent possible sinkholes from developing underneath the freeway. Over the course of the successful 230-foot long casing installation, the contractor pumped approximately 28 cubic yards of cement/bentonite grout mixture at two locations and approximately 69 cubic yards of contact grout at five locations along the casing.

After successfully installing the casing under the I-405, the City and WSDOT wanted to be confident that all potential voids created during constructions were completely filled. To accomplish this task, the City contracted GPR Data to survey the project area using ground penetrating microwave radar (GPMR), a technology that would accurately confirm the presence of any voids above the casing.

The GPMR investigation was completed overnight on January 9, 2017. The resulting data showed no voiding or highly porous subsoils in the scanned area, meaning that the contractor had successfully filled all potential voids beneath the I-405.

LESSONS LEARNED

The sewer relocation projects through Talbot Hill and beneath the new I-405 were both successful, thanks to the design team and contractor's adept use of trenchless technologies, resilient response to challenges posed by delicate soil conditions and groundwater, and effective use of GPMR to make certain that the project area remains safe long after construction.

The lessons learned through this experience will inform the design and construction of similar projects in the future.

Comparison of Inadvertent Return Methods – an approach for integration

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Horizontal directional drilling (HDD) is a widely accepted method for trenchless installation of conveyance pipelines. Inadvertent returns (IRs) of drilling fluids is an increasingly significant issue for HDD because of more stringent environmental regulations and concerns. However, a unified and commonly accepted method for predicting IRs has lagged behind other design calculations for HDD. This article is an attempt to better quantify and predict IR occurrence and to propose an integrated approach to IR evaluation by building on the already published efforts of others.

In this article, we will discuss the Cavity Expansion Theory application (the Delft Equation) and how Rpmax influences the Delft Equation. We will present a modified interpretation of the Delft Equation (referred to herein as Delta Delft) which can be applied to HDD calculations for installations through multiple soil layers. Lastly, we will discuss how an incremental analysis of each soil layer may lead to refined methods for IR prediction. Below is the standard Delft Equation (Eq.1).

$$p_{max} := \sigma_p + \left[\sigma_e \cdot (1+\sin(\varphi)) + c \cdot \cos(\varphi) + c \cdot \cot(\varphi)\right] \cdot \left[\left(\frac{R_0}{R_{pmax}}\right)^2 + \frac{\sigma_e \cdot \sin(\varphi) + c \cdot \cos(\varphi)}{G}\right]^{\frac{-\sin(\varphi)}{1+\sin(\varphi)}} - c \cdot \cot(\varphi)$$

The variables are defined as follows:

pmax = Confining Pressure, maximum allowable pressure that a given soil layer at a depth can withstand without inadvertent returns to the surface, (lb/ft2)

ϕ, ϕ = Friction Angle of the soil, determined through geotechnical testing, (°)

R0 = Bore Radius, which increases with each reaming pass, (ft)

G = Shear Modulus, determined through geotechnical testing or estimated using typical values of explorative result soils, (lb/ft2)

c = Cohesion Coefficient, determined through geotechnical testing or estimated using typical values of explorative result soils, (lb/ft2)

σ_p, u, ρ = Pore Water Pressure, initial ground water pressure which is equal to the weight of the water column (unit weight of water × the water table depth to bore hole), (lb/ft2)

σ_e, σ' = Effective Pressure, effective weight of the soil column of the bore hole minus the Pore Water Pressure, (lb/ft2)

Rpmax = Radius of the Plastic Zone, the anticipated zone of acceptable drilling mud migration through the soil layer or strata, (ft). This is usually estimated to be from 1/2 to 2/3 of the soil layer the boring is in or the entire soil strata depth depending on engineering assumptions and soil conditions.

When using the Delft Equation, it is important to understand the sensitivity of the input variables and how fluctuations within these variables affect the overall calculation's outcome. Through extensive research and application of the equations, the relationships between the variables and outcomes help us predict the outcomes. Additionally, extensive use of the equation provides practitioners an understanding of the process and allows modifications of certain variables which we have control over to produce a favorable outcome during construction. While we don't have much control over in-situ soil conditions, understanding how the confining pressure calculation uses the soil parameters will aid in understanding the Delta Delft application of the Delft Equation.

To better demonstrate the sensitivity of Rpmax, we ran three soils with different soil parameters through the Delft Equation along with a series of Rpmax values for each group as shown in Table 1. Soil 2 represents sand/cohesionless soils, while Soil 3 represents clay/cohesive type soils, and Soil 1 represents an average of the inputs used for Soils 2 and 3.

	Soil 1 Avg. of Soil 2 & 3 Parameters		Soil 2 Cohesionless Soils		Soil 3 Cohesive Soils	
R_{pmax} (ft)	**p_{max}** (psi)	**% of confining pressure at 35 ft**	**p_{max}** (psi)	**% of confining pressure at 35 ft**	**p_{max}** (psi)	**% of confining pressure at 35 ft**
350	179.21	101.0%	320.60	102.0%	96.84	100.2%
175	179.15	100.9%	320.40	101.9%	96.83	100.2%
140	178.79	100.7%	319.06	101.5%	96.78	100.2%
35	**177.51**	**100.0%**	**314.45**	**100.0%**	**96.61**	**100.0%**
17.5	172.83	97.4%	298.25	94.8%	95.96	99.3%
8.75	159.00	89.6%	255.31	81.2%	93.74	97.0%
4.38	132.67	74.7%	188.18	59.8%	88.13	91.2%
2.19	102.74	57.9%	127.32	40.5%	78.90	81.7%

Table 1. Data from R_{pmax} sensitivity analysis

For each soil, pmax was computed and the findings are shown in the "pmax" columns for each soil type. Basing the test on a Rpmax of 35 feet and treating this as the full realization of the confining pressure to be found in the soil profile, the remainder of the various radius data points in the first column were then compared to what was found for 35 feet as a percentage. These percentages are shown in the "% of confining..." column. The results indicate that, for the range of examples tested, the strength was realized very quickly as one moved away from the bore hole. After a certain point, there is a leveling-off effect that happens in the confining pressure found. Once this plateau is reached, increases in confining pressure are small as Rpmax increases. Table 1 shows that increases in confining pressure are less than 3% as Rpmax increases from 35' to 350', an increase of 1000% in Rpmax.

Looking at the data in Table 1, in the sand soil (soil 2), 80% of the strength was realized in the first 8 feet and approximately 95% was realized in the first 18 feet. In the clay (soil 3), more than 95% of the strength was realized in the first 8 feet. If the Rpmax value is pushed to 350 feet, the Delft Equation shows only an extra 2% is gained versus 35 feet.

These results suggest that the majority of the confining pressure for a particular soil is developed in close proximity to the bore. In most cases 95% of the confining pressure is developed within about 15 feet and pushing Rpmax further will have little significant effect or benefit on confining pressure.

MULTI-LAYERED SOILS

Below are methods previously proposed within the HDD industry for addressing multi-layered soils with the Delft Equation:

1. Apply the Delft Equation using soil parameters from the soil layer the HDD bore is within. This approach has sometimes been referred to as the "local confining pressure". In scenarios with soil layers thinner than 8 feet, this approach can be misleading because it lends much credence to the soil closest to the bore while ignoring the effect of other soils. While the soil closest to the bore has a major influence on the confining pressure, as we have shown, if the bore is less than 8 feet from a disparate soil layer, the influence of the disparate soil could be overlooked with this method.
2. Take a weighted average of the soil parameters above the bore hole, weighting each layer's thickness vs. the overall depth of cover or the layers located within Rpmax. Once these weighted soil parameters are determined, apply the Delft Equation using the weighted soil parameters as inputs. This approach is sometimes referred to as the "average confining pressure". This could also result in misleading answers under certain scenarios, as it artificially lends much credence to soil strata far away from the bore. Due to the nature of the Delft Equation and how the inclusion of higher Rpmax's behave in regard to confining pressure, soil far from the bore actually has less influence than this methodology would indicate. While this method has the benefit of taking multiple soil layers into account, it could obscure the realities of boring through very poor soil by incorporating more competent soils away from that bore (more than 15 feet) which could lead to a false increase in confining pressure estimates.

PROPOSED DELTA DELFT METHOD

The sketch in Figure 1 shows the general approach for the "Delta Delft" method.

The Delta Delft Formulas are described in Equations 2, 3, and 4.

$$p_{m_i} := \sigma_p + \left[\sigma_e \cdot \left(1 + \sin(\varphi_i)\right) + c_i \cdot \cos(\varphi_i) + c_i \cdot \cot(\varphi_i)\right] \cdot \left[\left(\frac{R_0}{R_{m_i}}\right)^2 + \frac{\sigma_e \cdot \sin(\varphi_i) + c_i \cdot \cos(\varphi_i)}{G_i}\right]^{\frac{-\sin(\varphi_i)}{1+\sin(\varphi_i)}} - c_i \cdot \cot(\varphi_i)$$

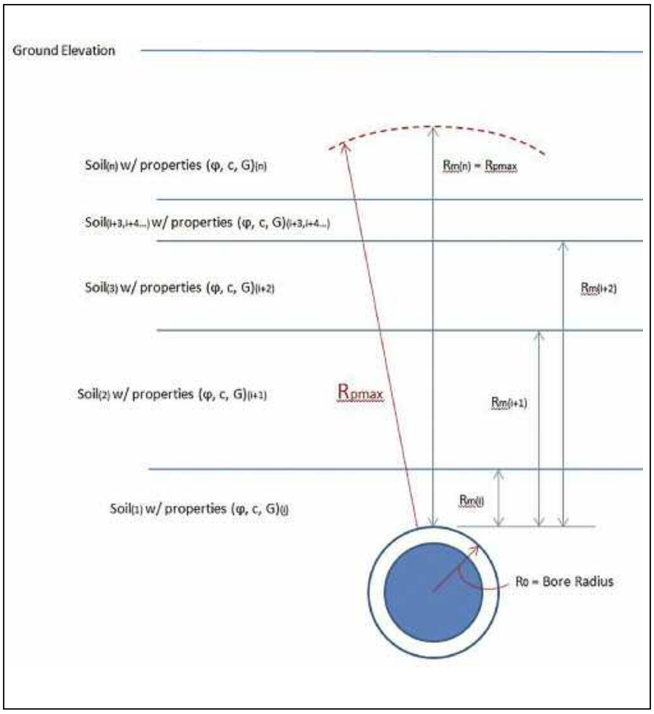

Figure 1. The Delta Delft variables

$$p_{n_i} := \sigma_p + \left[\sigma_e \cdot \left(1 + \sin(\varphi_i)\right) + c_i \cdot \cos(\varphi_i) + c_i \cdot \cot(\varphi_i)\right] \cdot \left[\left[\frac{R_0}{R_{m_{(i-1)}}}\right]^2 + \frac{\sigma_e \cdot \sin(\varphi_i) + c_i \cdot \cos(\varphi_i)}{G_i}\right]^{\frac{-\sin(\varphi_i)}{1+\sin(\varphi_i)}} - c_i \cdot \cot(\varphi_i)$$

where

$$p_{max} = \sum_{i=1}^{n} \left(p_{m_i} - p_{n_i}\right)$$

Equations 2 and 3 are direct versions of the Delft Equation. The only difference is Eq.2 calculates the confining pressure as if Rpmax was at the top of soil layer (i), noted by Rmi, and Eq.3 calculates the confining pressure as if Rpmax was at the bottom of soil layer (i), noted by Rm(i-1). pmi – pni simply calculates the difference between Eq.2 and Eq.3.

For each soil layer within the envelope of Rpmax, the soil parameters are determined by geotechnical soil testing and experienced geotechnical engineers.

$$\text{Soil}_i = \left(\varphi_i, c_i, G_i, R_{m_i}\right)$$

for each (i) where n = number of soil layers within Rpmax.

For this analysis, pore water pressure, effective pressure, and bore hole radius are constant and are given or calculated through normal means. Lastly pn1 is set equal to 0. This last equation parameter is required since Rm0 is either not developed or equal to 0 which would give an undefined result for pn1.

The variables in the above equations have the same names, descriptions, and units as those described in the Cavity Expansion Model Section above (Eq. 1). Note that for the top layer of soil (n) that is part of the analysis, Rpmax will equal Rm(n).

DELTA DELFT COMPARED TO OTHER METHODS

The following example compares the results from the Delta Delft to the previously presented methods for addressing multi-layered soil strata.

The pilot bore of an HDD is 70 feet below the ground surface. For this example, a factor of safety of 2.0 is applied to the Delft Equation resulting in a Rpmax of 35 feet. The effective pressure and the pore water pressure are calculated to be 2,300 psf and 4,200 psf, respectively. The pilot bore diameter is 10 inches, resulting in a bore radius of 5 inches (0.4167 feet). The Rpmax zone in the analysis crosses over three definable soil layers (n = 3) and each layer has unique soil properties and thicknesses in the soil strata (see Table 2).

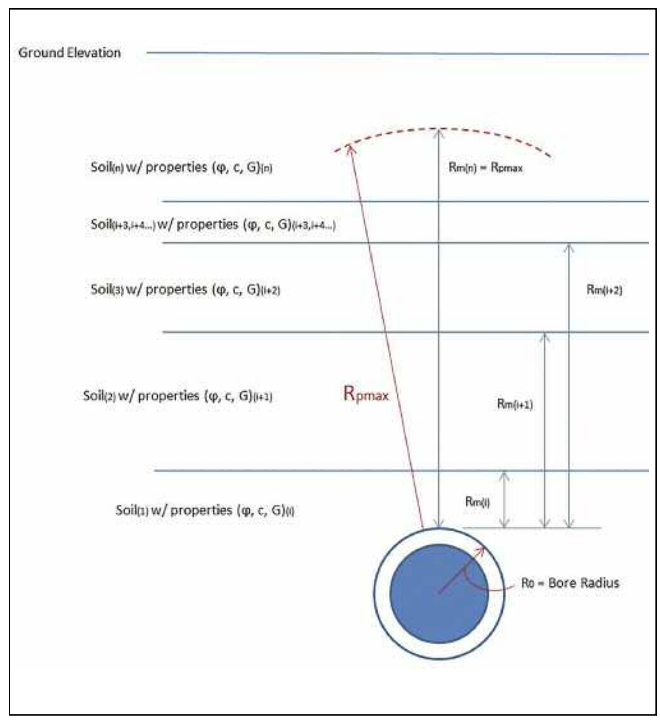

Figure 2: The Delta Delft example

$Soil_{(i)}$	ϕ(°)	c (lbs/ft^2)	G (lbs/ft^2)	$R_{m(i)}$ (ft)
$Soil_{(1)}$	42	5	800,000	3
$Soil_{(2)}$	1	1,152	216,000	11
$Soil_{(3)}$	1	1	600,000	35

Table 2. Soil parameters

Delta Delft Results:

- pm(1) = 154.36 psi, pm(2) = 96.35 psi, pm(3) = 274.14 psi;
- pn(1) = 0 psi, pn(2) = 85.01 psi, pn(3) = 238.74 psi;
- Giving pmax = (154.36 - 0) + (96.35 - 85.01) + (274.14 - 238.74) = 201.76 psi

Comparison with Local Confining Pressure Delft application:
pmax = 347.46 psi

Comparison with Average Confining Pressure Delft application:

- Weighted average soil parameters:
  - o ϕ = 29.2°,
  - o c = 264.4 lbs/ft2,
  - o G = 529,371 lbs/ft2;
- pmax = 225.20 psi

In this example, the Delta Delft equation resulted in a confining pressure that is lower than either the Local Delft or the Average Delft approach. In this scenario, the soil layer the HDD is boring through is thin. But the Local Delft equation depends entirely on the soil parameters of Soil(1). As such, this approach greatly overestimates the confining pressure.

Similarly, a weighted scenario gives the soil layers further away from the HDD bore as much influence as those near the bore. In the above example there is a poor-quality soil layer 3 feet away from the bore. With this layer close to the bore it is reasonable to expect that it should influence the confining pressure more than the highly competent and much thicker 3rd soil layer that is further away. However, the straight weighting of these parameters does not take proximity into account.

CONCLUSION

The Delta Delft version of the Delft Equation incorporates the "delta"-influence of individual soil layers in the overall confining pressure of a multilayered soil profile. As new methods are developed for evaluating sand layers, silt layers, or clay layers, these new methods can be incorporated into the Delta Delft equation and calculate the Delta Delft on a "per soil layer" basis. Ultimately, this new Delta Delft equation could use whichever individual methods that most accurately predicts for given soil layer's type and condition. It would be, in a sense, a unified theory of IR prediction.

Guided Boring Using Pilot Tubes: Variations of the Method

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National Clay Pipe Institute

Minimal community disruption, low equipment costs and pinpoint accuracy are the three most common reasons for using the Pilot Tube Method (PTM) of guided boring. But just as each project is unique, so too are the engineer's or contractor's reasons for selecting this method. PTM is practical in weak soils and at greater depths, contractors can effectively avoid existing utilities and install sanitary sewer lines below the water table. Those are all compelling reasons the method is being adopted by engineers and contractors alike.

The technique originated in Japan and Europe three decades ago as a way to install 4- and 6-inch gravity flow house connections. First introduced in the United States in 1995, it has steadily grown in popularity and expanded to widespread adoption for installation of gravity sewer mains.

Initial applications of the process in the U.S. were conservative, with a range of 4-inch- to12-inch-diameter pipes with single drive lengths limited to 250 feet. After more than two decades of experience in the U.S., the technology is now used to install pipes of up to 48-inch outside diameter with common drive lengths ranging from 350 to 400 linear feet. Single drives of up to 580 LF have been completed successfully using PTM. Accuracy to within a quarter-inch in line and grade are frequently achieved on drive lengths of 500 linear feet. Improved optical guidance systems and hydraulics in the jacking frames have made larger diameters and longer drive lengths practical.

As popular as the method has become in

Figure A. Pilot Tube Monitor Screen

Figure B. LED-Illuminated Target

some areas, there are still large parts of the country where the technique is a bit of a mystery.

THE METHOD EXPLAINED

There are effectively three approaches to pilot tube installation: the two-step method, the three-step method, and the modified three-step method utilizing a powered head. In all approaches, the first step is the same. The following steps are typically controlled by the final product pipe diameter, soil conditions and the tunnel contactor's equipment.

The Pilot Tube Method relies on the guidance system, adopting the use of an LED target, digital theodolite, monitor screen and a "real time," camera-based accurate guidance system (see Figures A and B). The video camera, mounted above the theodolite, transmits the image of the battery-powered LED-illuminated target located in the steering head to the monitor which is visible to the operator. The straight line indicated by the center of the target designates the direction and path the slant-faced steering head will follow.

Hollow steel pilot tubes which fasten to each other via a threaded hex connection are available as a double- or single-wall tube depending on the manufacturer. On some double-walled tube systems, the inner tube will rotate with the steering head during advancement for torque reduction. On other double-walled systems, a bentonite lubricant may be pumped through the annular cavity between the tubes to the steering head to assist with soil friction. These pilot tubes range in length from 30 inches to 2 meters, depending on size of jacking frame and shaft diameter.

A slant-faced steering head (similar to that of a directional drill) houses the LED-illuminated target. Steering heads of different degrees of angle are available for various types of ground conditions. During the installation process the ground is displaced by the steering head/pilot tube and directed on line and grade by rotation during advancement. Once Step 1 of the installation is complete, a survey can be performed on the pilot tube at the reception shaft to verify line and grade accuracy of the initial survey and setup. If a survey or setup error is found, the pilot tubes can be retracted and reinstalled to achieve the desired line and grade before proceeding to the second step of the installation.

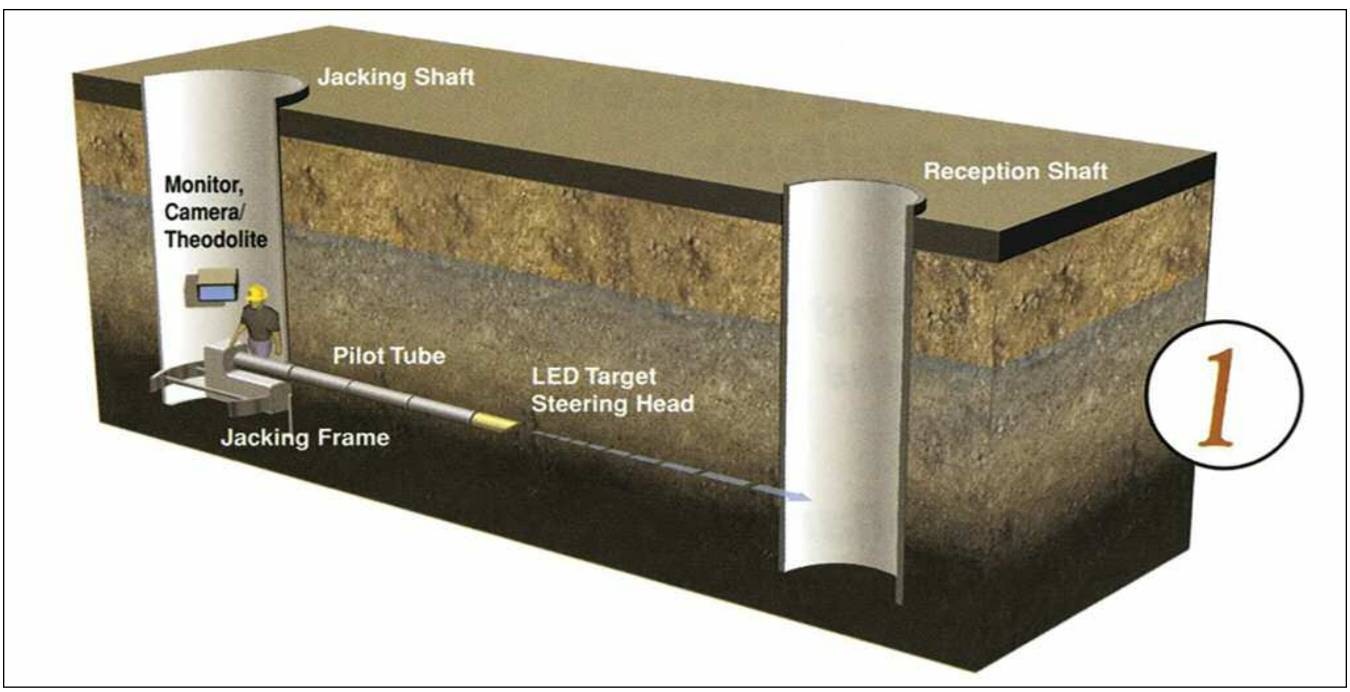
STEP ONE

The first step in all the Pilot Tube installation methods is the precise installation of the pilot tube on line and grade. The hollow stem of the pilot tube provides an optical path for the theodolite to display the head position and steering orientation. This step establishes the center line of the new installation as the remaining step(s) will follow the path of the pilot tube.

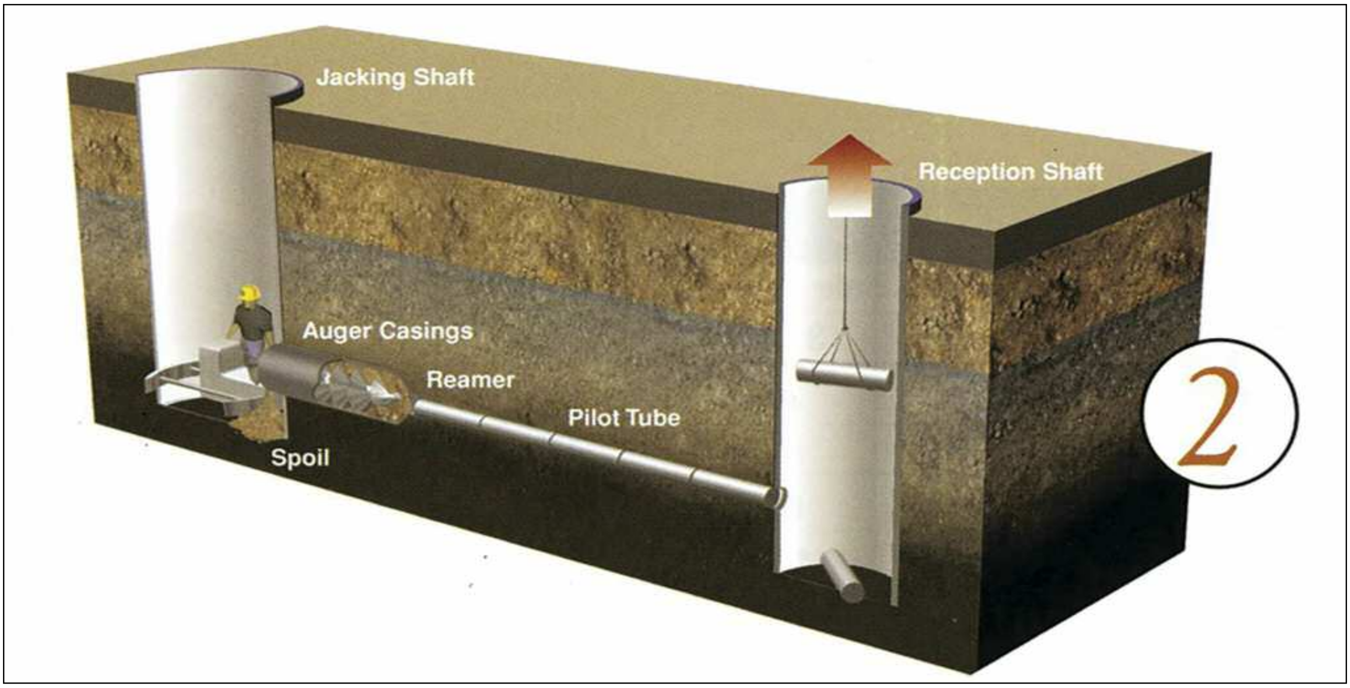
Once Step 1 is complete, the theodolite and monitor guidance system may be removed from the jacking pit as they are no longer required.

STEP TWO IN 3-STEP METHOD

The second step (in the 3-step and 3-step modified methods) is to follow the path of the pilot tube with a reaming head, which is sized to the outside diameter of the final product pipe. The front of the reaming head fastens to the last pilot tube installed in the same manner the pilot tubes fasten to each



Step One – installation of pilot rod

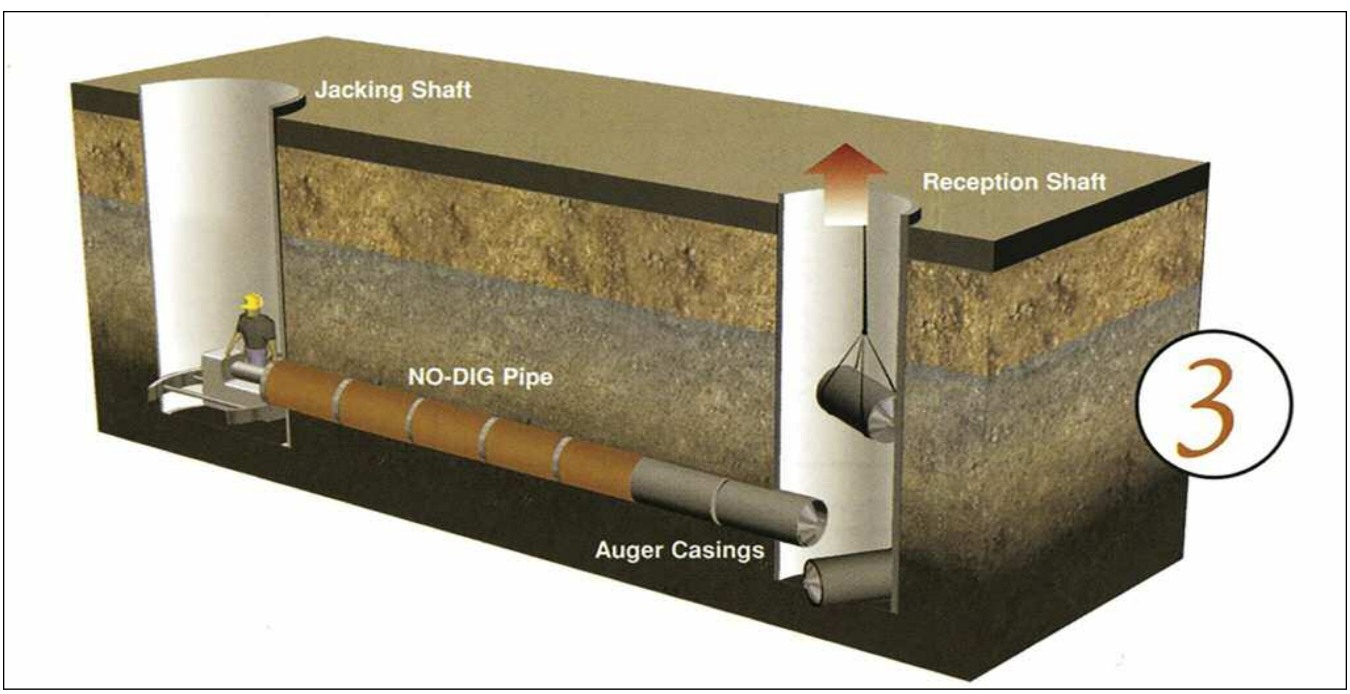


Step Two – installation of auger casings

other. Following the reaming head are auger casings of the same diameter as the head transporting the spoil to the jacking shaft for removal. The spoil can be removed by a muck bucket or vacuum truck depending on the soil type and contractors' preferences. This step is complete when the reamer and auger casings reach the reception shaft and all spoil is removed.

STEP TWO IN 2-STEP METHOD

The second step (the final step in the 2-step method) is to follow the path of the pilot tube with the reaming head advanced by the final product pipe. This reaming head funnels the excavated material into auger casings coupled together inside the product pipe and conveyed through to the jacking shaft for removal. These auger casings are then retracted from the inside of the carrier pipe via the jacking shaft. This method has an advantage to contractors as they are able to install multiple sizes of sewer lines while



Step Three – pipe installation

utilizing the same set of auger casings. The disadvantage to this 2-step system is the decreased diameter auger casings will limit the maximum diameter of excavatable cobbles and hardened material. When the 2-step method is utilized, the pipes are set into the jacking frame with the auger casings inside. The auger casings are attached to the reamer (if it is the first pipe to be installed) or previous casing for spoil transport. The product pipe carries the axial load required for advancement and is equal in diameter to the reamer.

Different types of reaming heads are available for a variety of displaceable soil conditions as well as heads capable of controlling flow when working as much as 10 to possibly 15 feet below the water table (ultimately depending on the soil type). A swivel is required connecting the pilot tube to the reaming head when a rotating cutter head is used for harder ground.

STEP THREE

The third step (final step in the 3-step method) is to replace the auger casings with the final product pipe. The reaming head and auger casings are advanced into the reception shaft and removed as the product pipes are installed. There is no spoil to be removed in this step as the product pipe has the same outside diameter as the auger casings.

The third step (final step in the 3-step modified method) is to install a powered cutter or reaming head behind the auger casings, which is advanced by the product pipe. This method is the newest innovation to the Pilot Tube Methods. These hydraulically driven heads increase the bore to match the larger product pipe diameter. The excavated spoil around the previously installed auger casings is discharged via the reception shaft by reversing the auger direction. This step is complete when the powered cutter head reaches the reception shaft.

PIPE MATERIAL

For most installations, vitrified clay jacking pipe (VCP-J) is preferred because of its unmatched axial strength and lifecycle benefits. During the service life of the installation, the natural properties of VCP make it uniquely suited to the high-sulfur, highly abrasive and highly demanding environment of a sanitary sewer. VCP also provides expanded maintenance options allowing for much more aggressive cleaning techniques to provide a better long-term value to municipalities.

Contact the National Clay Pipe Institute at 262-742-2904 for educational presentations on the Pilot Tube Method and vitrified clay jacking pipe.

RECENT PROJECTS

The cities of Omaha, Nebraska, and Portland, Oregon, have been employing the Pilot Tube Method (PTM) for over 10 years. Both cities come back to this method repeatedly, in part because of successful track records.

In Portland, Project Hemlock and Project Outfall 33 were completed in 2016. The Pilot Tube Method was chosen for Hemlock to allow for installation in a very tight right-of-way between residences. The need to relocate overhead utilities if an open trench were created was also a factor for designers to consider. Nine hundred linear feet of 12-inch-diameter VCP was installed from one 9-foot shaft in three different drives. The first two drives created a straight-line installation down the alleyways, between the homes. The jacking frame was then turned at a 90-degree angle to the other two drives to tunnel under an existing garden that included 100-year-old rose bushes. In addition to preserving the highly prized garden, using the trenchless method allowed homeowners daily access to their homes and left the area utilities undisturbed.

Project Outfall 33 was in a heavily traveled downtown area that serves both vehicle and foot traffic. New sanitary sewer mains were recently installed in the dense urban area. PTM installation's much smaller footprint meant much of the traffic could be maintained while the project progressed on roughly the same timetable that an open trench would have required. The weak soils in the area also meant an open trench would have compromised the integrity of streets and sidewalks. In weak soils, such as the soils generally found in the Portland area, PTM is one part of ensuring a safer workplace for installers.

The City of Omaha is in the midst of a sewer separation program; currently in its 12th year, the program has faced many challenges. The Pilot Tube Method has been critical to its progress. One of the consistent challenges the program faces is dealing with very weak soils.

Omaha's Nicholas Street project involved installing nearly 5,000 linear feet of 24-inch-diamater VCP at a depth of 40 feet. In this case, the combination of weak soils and great depths meant this would have been a difficult project for most other installation methods. The ability to safely install sewer lines at these depths frequently enables designers to eliminate lift stations.

A 9-foot shaft made trenchless installation in this tight Portland alleyway practical.

Curved Microtunneling

Cyrill Althuser
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Curved microtunneling is slowly being adopted in the United States as contractors, designers, and manufacturers gain a better understanding of the requirements associated with curves. In Canada and most European countries, curved microtunneling is used as a method to reduce cost and risk on projects. As Portland and Seattle continue to expand, a number of curved drives are planned to be tunneled in the near future. The cost savings behind curved micro tunnels is primarily due to the elimination of vertical shafts and labor and schedule constraints associated with them. Five shafts can be reduced to three, or three shafts reduced to one. Elimination of shafts can also benefit the public by reducing detours, blocked trails, and the closing of public areas. Environmental considerations may also play a large role, especially in areas with contaminated soils or sensitive wildlife populations.

Reinforced concrete pipe (RCP) is the standard pipe used for curved microtunneling and pipe jacking around the globe. In Europe for example, RCP covers the vast majority of microtunnels for any purpose, including direct jacked sewer systems and railway crossings.

Reinforced concrete is the preferred material for several reasons. Apart from the competitive price, if designed correctly, the pipe is robust and ductile and can be adapted to project specific requirements. Thorough design and precise manufacturing processes may provide a design service life of 100 years or more. Inliners made of FRP, HDPE, PVC or RCP with an embedded steel cylinder (AWWA C300) pipes can provide increased corrosion resistance and reduced hydraulic friction, and some have high-pressure ratings suitable for various pressure applications. Polymer concrete pipe can also be used for curved microtunnels without the need for an inliner.

When it comes to curved microtunneling, reinforced concrete jacking pipes are typically preferred because they can be designed to allow large joint deflection angles with minimum damage potential to the joint, and still maintain the required water tightness. Large joint deflection angles require some thought on pressure distribution at the joint. Typical wooden

Image Courtesy of Bradshaw Construction

Image by Thompson Pipe Group

packers are designed for straight runs, flat curves, and compensation of manufacturing tolerances.

Curved microtunneling is not new. Most tunnel boring machines (TBM) are fit to mine curved alignments. Besides, even a straight microtunnel will be executed as a curved one as various aspects, like geology, groundwater, overcut or simply the weight of the TBM require steering operations to keep the tunnel aligned.

As the pipes are pushed behind the TBM they experience similar joint deflections (with reduced amplitude) as the TBM is steered. It is important to remember the pipes are passively following the TBM as it is actively steered. Therefore there are two important considerations, both concentrated in the pipe joints: The joint needs to stay water tight, and it has to distribute the jacking force around the circumference of the pipe as the tunnel advances.

DIFFERENT REQUIREMENTS FOR PIPE JOINT

The primary purpose of a pipeline is to transfer different media between two points. The pipe itself as a bearing structure in the shape of a circular ring that withstands external loads like earth load, traffic load, and groundwater pressure. It maintains its shape and profile in the ground and is typically fabricated as a circumferentially endless, closed ring. The dimensions and reinforcement of the pipe wall can be designed according to different codes worldwide. In North America ASCE 27-17 provides a direct design approach. A combination of ASTM C76 and ASCE 27 specifications are sometimes used to simplify design. This can have some drawbacks since C76 leaves out some checks that would be performed if designing the pipe with a direct design method.

The pipe joint is a discontinuity in the pipeline and has to fulfill several requirements for a project to be completed successfully. The most obvious is the sealing effect. It has to prevent bentonite, grout, water, earth, and any other foreign materials from infiltrating the pipe during construction and prevent the transmitted fluid from leaving the pipe during pipeline operation. In a typical open-cut application, this requirement is fulfilled when no external material enters or leaves the pipeline during its service life. This can be achieved with a joint design that produces a seal anywhere in the joint.

In pipe jacking, the pipe must also serve as a bearing and load transferring structure in the longitudinal direction. Large jacking forces have to be transferred from the main jacks in the drive pit to the tunnel face through the pipe's wall and through the pipe joints. Since no external debris should enter the load bearing surface of the joint during jacking operations, the seal preventing infiltration is best suited to be on the outer circumference of the pipe.

COMMON JOINT DETAIL

A widely used joint detail that illustrates an external joint seal is the Type C joint in ASCE 27-17. This joint prevents debris entering the pipe joint bearing area and provides maximum surface area for jacking operations. The joint consists of a steel bell band that is cast into the concrete pipe around the outer circumference, forming the female end of the jacking pipe. The spigot end is shaped by a reduced outer diameter with a groove that keeps the gasket in place. Essential to this detail are the tight fabrication tolerances that keep the gasket within its design compression rate suitable to the project specific requirements. These tolerances can be achieved using steel molds and a well-controlled wet-cast manufacturing process.

The steel bell is cast into the concrete to provide resistance against lateral forces acting on it due to joint articulation. It is anchored into the concrete with shear studs or steel anchors. The water tightness between steel bell and concrete is achieved either by a cast-in steel profile, fully welded around the circumference of the steel bell band, or by applying a hydrophilic seal.

This design can be adjusted to project specific curve radii or joint deflection angles. The length of the steel bell can be increased when tighter curves have to be driven to ensure that the gasket never slips out of the steel bell. The outer diameter of the spigot may be reduced or beveled to provide enough space between the steel bell and concrete to facilitate the articulation.

The gasket groove is an essential part of the design as well. Opening and closing joints during a (curved) drive could otherwise pull the gasket off the spigot end into the pipe and lead to leakage. The gasket itself can be of circular or wedge shaped cross-section, where the wedge-shaped gasket has the advantage of easier joining of the pipes and a well-defined contact area between gasket and concrete grove.

CONVENTIONAL PRESSURE TRANSFER

Another important consideration is the pressure transfer ring. It is an import part of pipe jacking with RCP. It helps distribute eccentric and point loads by distributing the forces from the jacking operation. Eccentric loads can occur from out of square end sections and steering operations, and point loads can occur from rough or uneven joints, or when a joint deflects enough to loose full contact around the joint. Since the concrete pipe itself is very stiff compared to the joint assembly, any deflection resulting from a curve or steering operation is concentrated in the joints. Changing directions due to steering movements cause the packer to experience multiple cycles of loading and unloading during construction. These conditions define the properties that a pressure transfer medium has to fulfill: It should be rather soft compared to the pipe material and it should have reversible behavior, without inducing unfavorable tensile splitting stress into the pipe's face.

The typical timber material for conventional packers fulfills some of these conditions and provides enough admissible jacking force for straight drives and flat curves. However, the material properties of timber are highly irreversible; upon unloading a timber packer, a certain plastic deformation remains but the stiffness of the material is increased compared to the unloaded state before. The more loading and unloading cycles a wooden packer experiences, the stiffer it gets and its capacity to equalize unevenness or joint deflection is reduced. The stresses induced to the concrete pipe by the wooden packer may increase during the construction period. Consequently, the damage potential increases as well.

Along with the imposed stress on the concrete, the resulting jacking force at an articulated joint is acting eccentrically on the pipe regarding its axis: The joint opens up on the outside of the curve and the jacking force is transferred by a "point load" on the inside of the curve. This eccentric forces may also impose a turning moment onto the pipe if the resulting force on the front end of a pipe is acting at a different location than on the back end of it. This is the case as soon as the joint articulation angles differ from the front to the back of the pipe. This is commonly observed due to the variability of steering operations and manufacturing tolerances.

This moment pushes the pipe against the ground, which means that ground reactions are acting laterally on the pipe. These reactions can be of the same amplitude as the jacking forces, especially if the pipe's length is similar to its outer diameter. Such high lateral forces are normally not considered in the dimensioning of the pipe and can lead to severe pipe damage.

A straightforward solution to reduce the joint deflection angles in curves is to shorten the pipes. However, short pipes have various disadvantages. Short pipes have a direct influence on the cost gener-

Image by Jackcontrol AG

ated by the manufacturing of the pipe as well as the installation cost on site. Furthermore, a joint is always a weak spot in a pipeline, therefore the number of joints is preferred to be as small as possible.

HYDRAULIC JOINT FOR PRESSURE TRANSFER

For longer drives, and especially curved drives, the hydraulic joint can provide an easy solution to the challenges mentioned before. It replaces the wooden packer as a pressure transmission ring.

The hydraulic joint is based on two principles: First, the principle of communicating vessels stating that the pressure level inside a vessel is constant, and second, the perfectly reversible mechanical behavior of non-compressible fluids independent from the deformation/load history. The hydraulic joint consists of a hermetically sealed hydraulic conduit, filled with a specific amount of fluid, which is mounted on the pipe's face instead of the conventional wooden packer.

As the jacking force is applied, the hydraulic joint gets squeezed and because of the constant volume enclosed in the hermetically sealed conduit, the fluid is set under pressure as a reaction to the applied force. This pressurized fluid cushion transfers the jacking force to the next pipe. Due to the hydraulic communication, no stress concentrations are acting on the pipe's face.

When driving through a curved alignment, whether planned or unplanned, the hydraulic joint gets squeezed more on the inside of the curve than on the outside, but no open gap will occur as would be the case using wooden pressure transmission rings. Therefore, the jacking force is distributed more uniformly around the pipe's circumference and the unfavorable eccentricity mentioned before is reduced by dimensions.

As a result of the reduced eccentricity, the lateral forces acting on the pipe are reduced by dimensions, which generates a series of advantages compared to the conventional joint design. Reduced lateral forces mean that the maximum jacking force can be maintained in a curve even with regular pipe lengths, without risking

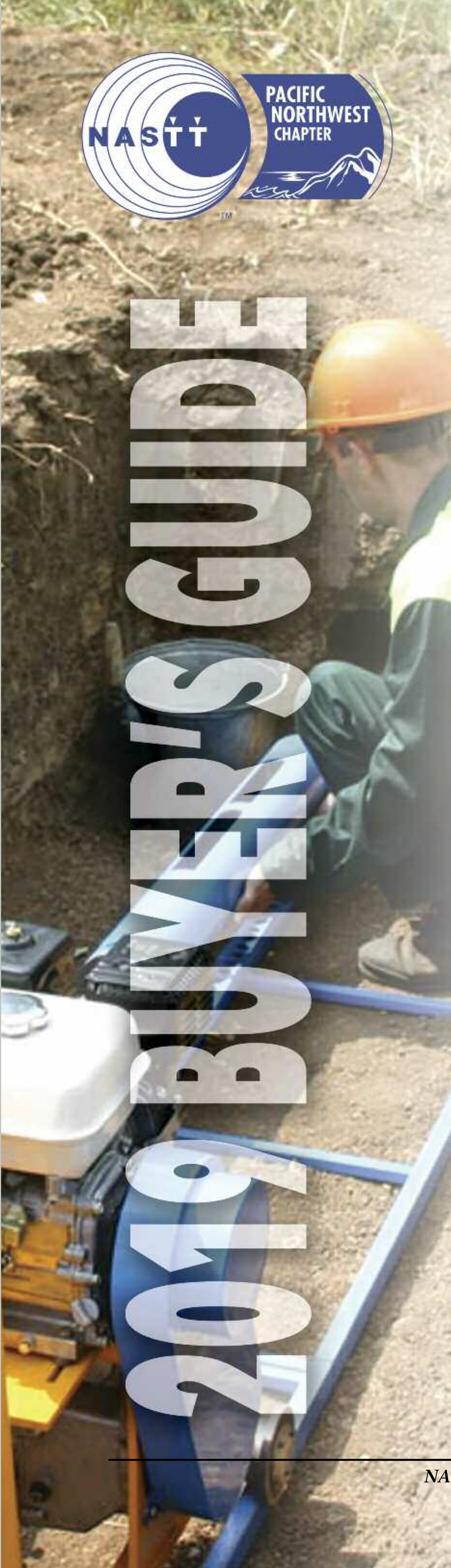
Image by Jackcontrol AG

any damage of the pipe. The possible radii of curvature for a pipe line is drastically reduced by the application of the hydraulic joint. This allows the design of new curved alignments that with wooden packers would be too risky or uneconomic, if not even impossible. Also, reduced lateral forces mean that the friction between soil and pipe is reduced by the same dimension. This reduces the possibility that an intermediate jacking station installed in the microtunnel will need to be activated, further increasing efficiency.

CHANGES IN APPLICATION FOR CONTRACTOR

The application of the hydraulic joint only requires insignificant changes for the tunneling contractor compared to the conventional joint design. The pipe manufacturer and hydraulic joint designer can work together on an acceptable design. Since the hydraulic joint's ends are tightened by steel fittings, cavities are required in the pipe's rear face for the fittings to be protected during jacking operations. Next, the hydraulic joint has to be mounted on the cured pipe using specially designed equipment, however this process is quick and easy. Besides, for these changes the jacking process can continue as it would be using conventional wooden packers.

Properly designed reinforced concrete jacking pipes are the first choice for micotunneling operations worldwide, especially for curved alignments. Various characteristics and technologies allow for easy to implement project specific designs that can cover a wide range of applications.



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