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Critical Analysis of Pipe Ramming Behavior and factors that impact predictive models.

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CRITICAL ANALYSIS OF PIPE RAMMING BEHAVIOR AND FACTORS THAT IMPACT PREDICTIVE MODELS.

Dr. Kimberlie Staheli, P.E.[1], Jake Andresen, MS[1] and Dr. Armin Stuedlein, P.E.[2]

[1] Staheli Trenchless Consultants, Inc, Seattle, WA, USA
[2] Associate Professor of Geotechnical Engineering, Oregon State University, Corvallis, OR, USA

ABSTRACT: The limits of pipe ramming technology are being stretched on a regular basis. Rams are increasingly installed in longer lengths, in more aggressive/dense geotechnical conditions, below the water table, and to installation tolerances that allow the installation of gravity pipelines within the rammed casings. Advances to the technology are largely led by innovations in the field, with engineers striving to develop design parameters that model the performance of the technology. In an effort to understand the mechanisms controlling the ramming, three projects were instrumented during construction to gather information. The instrumentation, consisting of strain gages and accelerometers, commonly used for pile driving analysis, were placed on pipe rams ranging from 915mm (36-inch) to 2130mm (84- inches) in diameter. Field data from the instrumentation was collected throughout the ramming while detailed field notes were taken, documenting all aspects of construction. This allowed correlation of ramming data with construction activities, and quantification of parameters that impact overall ramming loads.

This paper details a case history of an instrumented project, presenting project details such as design parameters, geotechnical conditions, execution of the ramming, groundwater control, and contractor activities throughout construction. Important design parameters that contribute to the success of large pipe ramming projects are presented. The results of the data and ramming activities are analyzed to quantify parameters such as hammer efficiency, resistance on the casing, force on the cutting shoe, advance rates, and hammer blows per meter.

1. INTRODUCTION

The broad range of trenchless technologies leave designers with a myriad of technology choices for crossings on which open-cut installation is not desired, possible, or acceptable. The capability of each trenchless method varies widely; therefore, the choice of trenchless methodology is narrowed by feasibility. Once the feasible trenchless methods for a project are narrowed to a small group of alternatives, the method selected is often driven by risk. Trenchless risk analysis has been broadly studied and a number of tools have been developed to quantify and compare risks of multiple trenchless methods for a single crossing. (Staheli, 2017). These methods have served to inform the Owner and Engineer of the relative risks of trenchless methods as well as identifying issues that could go wrong during construction. This education has allowed Owners to understand the construction risks inherent in their projects and develop contingency planning for project budgeting. This planning effort is valuable to Owners during the execution of their projects and often saves them from depleting project budgets and having to allocate additional funds after the project is bid.

The primary factors that controls risk on most trenchless projects are the geotechnical and groundwater conditions. Generally, the more variable the conditions within a single trenchless segment, the higher the difficulty and risk. Therefore, projects that have highly variable soil conditions can be very challenging and typically carry considerable risk cost. The case history detailed herein describes a project with considerable-risk cost. The geotechnical conditions were highly variable along the alignment. Groundwater was present above the crown of the pipeline in all areas; however, in isolated areas it was perched, and other areas exhibited artesian conditions.

The project also had an established history on which microtunneling had been previously attempted under a different contract but could not successfully excavate the cobbles that were encountered along other segments of the alignment. The cobbles were tested and found to have unconfined compressive strengths between 276 MPa (40,000 psi) and 345 MPa (50,000 psi). The original microtunneling project was terminated and re-design was started with this knowledge of the geotechnical conditions and performance of the microtunneling equipment.

2. CASE HISTORY DETAILS

The original project was designed and constructed with two Owner entities under a partnership. After termination of the original project, the project was split into segments and each Owner proceeded forward with design and construction of the portion of the project for which they were responsible. The objective of the project was to convey flow for a 610mm (24- inch) water line and a 610mm (30-inch) gravity sewer line. The District obtained permission to house the two pipelines in the same casing; however, the casing needed to be large enough to allow adjustment of the sewer to maintain gravity flow should the alignment of the installed casing deviate from design grade.

The project included two crossings in close proximity that required trenchless installations– one beneath Highway 527, a very busy roadway that contained many utilities, and the other beneath Silver Creek which could not be open cut as it was a salmon-bearing stream. Figures 1 and 2 shows the profile of the crossings for the project.

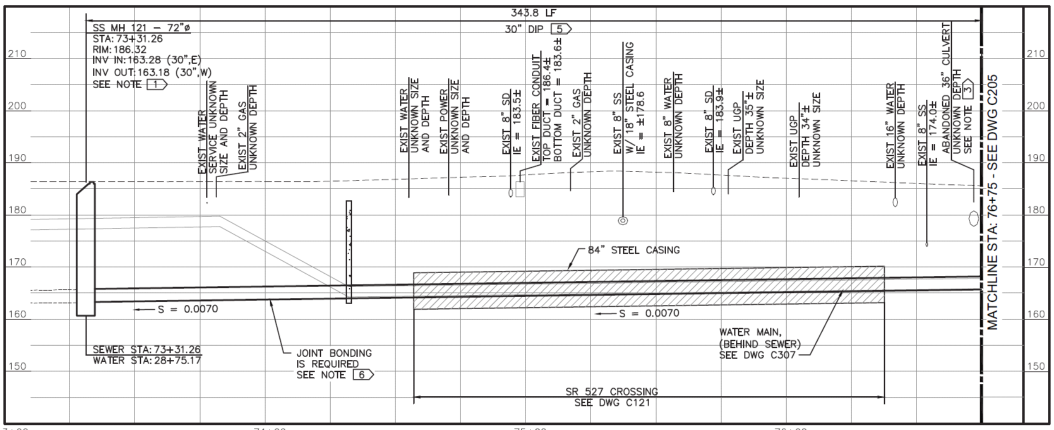

Figure 1 Crossing of SR 527

The crossing of Highway 527 was designed to be approximately 44 m (145 feet) long and the Silver Creek Crossing was designed to be approximately 10 m (35 feet) long. Choosing the trenchless method for the installation of the casings was of critical importance for the success of the project. The District would not allow the use of microtunneling because of the previous failures on the project, however, the sizes of the water and sewer pipes required that the casings be designed to provide a nominal diameter of 2,130 mm (84-inchs), leaving few trenchless methods available for selection.

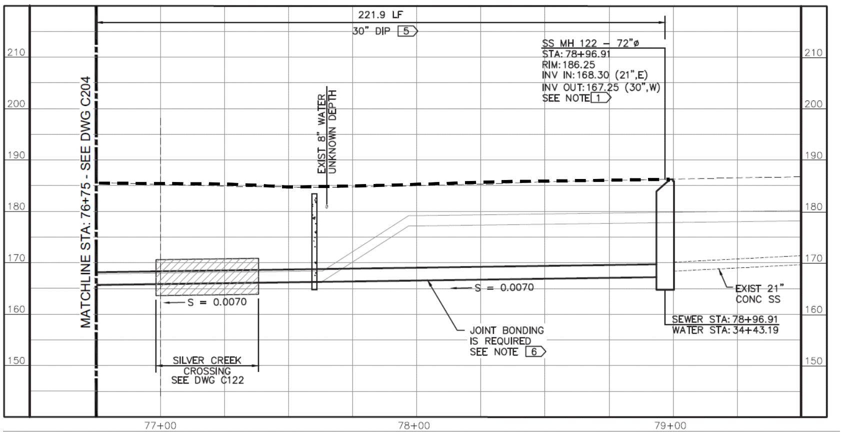

Figure 2 Crossing of Silver Creek

3. GEOTECHNICAL CONDITIONS AT THE CROSSING LOCATIONS

Geotechnical information gathered for the original project constructed in 2008-2009 was available for the design of these crossings. In addition, there were construction records and reports from the geotechnical engineer of record on the original project documenting the geotechnical conditions.

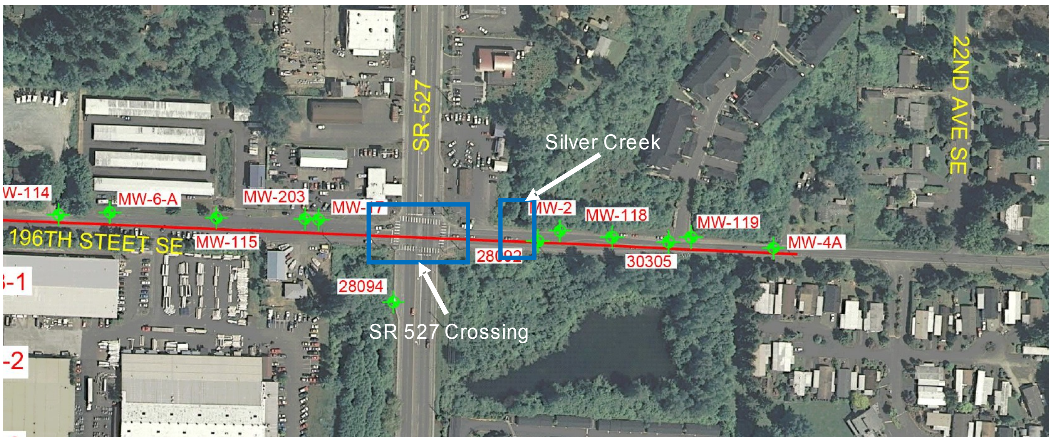

Figure 3 Locations of SR524 and Silver Creek Crossings and Geotechnical Borings along the Alignment

Figure 3 shows the geotechnical borings from the original project in relation to the trenchless crossings. It was determined that the existing geotechnical borings were sufficient for characterizing the soils along the crossings. There were three major categories of soil units used to describe the geotechnical conditions 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:

- Alluvium and recessional outwash deposits-- The post-glacial alluvium and recessional outwash soils generally consist of silt, sand and gravel and vary from medium dense to very dense, generally increasing in density with depth. These soils will exhibit moderate to high strength. The cleaner, sandy and gravelly zones of these deposits will exhibit moderate to high permeability, while the more fine-grained zones will exhibit low permeability. (GeoEngineers, Geotechnical Design Summary Report, August, 2011a).

- Dense Glacial Deposits-- "*The glacial deposits primarily consist of sand with varied amounts of silt and gravel and are generally very dense in consistency. These soils will exhibit high strength and generally low permeability. The cleaner sandy and gravelly zones may contain groundwater; however, these zones are generally not contiguous*." (GeoEngineers, Geotechnical Design Summary Report, August 2011a).

The previous project encountered cobbles on the trenchless microtunneling segments, discovered when the microtunneling machines were excavated from rescue shafts. In addition, large boulders (up to 1.5 m (5-feet) in diameter) were discovered with 100 feet of the planned trenchless crossings of this project on open-cut segments. Therefore, the District was very concerned about hitting boulders beneath Highway 527 and Silver Creek that would stop the forward progress of any trenchless method. As such, they elected to use a small diameter HDD pilot bore as an investigative geotechnical tool to try to determine if boulders were along the trenchless design path. Unfortunately, due to the frequency of gravel and cobbles along the alignment, the HDD pilot was unable to maintain a pilot bore along the design alignment and was deflected away due to hard/dense soils. It was not possible to determine if the soil that deflected the pilot was cobbles, boulders, or dense glacially overridden soil. The information obtained from the pilot bore, such as location where the bore deflected, and difficulty with drilling was provided to the bidders within the Geotechnical Data Report (GeoEngineers, 2011b).

4. SELECTION OF TRENCHLESS METHOD AND SPECIALTY DESIGN PARAMETERS

Trenchless methods for the installation of the casings beneath Highway 527 and Silver Creek had to be compatible with the following:

- Existing geotechnical conditions that included gravel, cobbles, and boulders;
- casing pipe that allowed two pipeline installations, one with a 760mm (30-inch) diameter that required gravity flow on a relatively flat grade;
- up to 3.05 m (10 feet) of groundwater above the crown of the pipe;
- restriction that SR 527 remain open at all times;
- restriction that only allows dewatering at the shaft locations;
- maximum settlement of 6.4mm (0.25 inches) on SR 527, and
- microtunneling eliminated from option analysis due to prior failures with 914mm (36-inch) machine.

Given these restrictions, pipe ramming was chosen as the preferred trenchless method. Due to the risks identified on the project, there were many specialty design parameters that were included in the pipe ramming specification. These features were items that would arguably impact the contractor's means and methods on the project. However, the Owner and Engineer decided that specifying these parameters was prudent given that the design team had studied the project and the pipe ram for several years and had identified specific risks and events that could occur and mitigation measures to address those risks. On the contrary, the Contractor was given a relatively short time period to prepare the bid price and the pipe ram was only a single element of the overall project. Therefore, the Owner and Engineer decided that alerting the Contractor to the risks by including risk mitigation items in the specification was a way to ensure that the low bidder was aware of the risks inherent to the project. The goal within the specifications was to give the contractor as much latitude with means and methods as possible while requiring certain procedures to be followed to lower the risk of the project.

Some of the specialty requirements included the design of a temporary pipe plug that would counterbalance the groundwater pressure to prevent soils from flowing into the face of the ram. The Contractor was also required to design a cutting shoe that met a diameter to thickness ratio that was developed by the pile driving industry and had been adapted to pipe ramming (Price and Staheli, 2013). The most unique requirement and one that would prove to be most valuable during construction, was providing real-time instrumentation of the pipe ram. The specifications required the contractor 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 in accordance with ASTM D4945 (ASTM 2017) which governs High Strain Dynamic Testing of Deep Foundations. The Contract required that each pipe be fitted with four pairs of strain gages and accelerometers at locations that were 90-degrees apart on the outside of the pipe, approximately one pipe diameter from the hammer-casing interface. Signals from the sensors were collected during ramming and were to be processed with Pile Driving Analyzer ® (PDA) manufactured by Pile Dynamics, Inc. Figure 4 shows the instrumentation mounted on the pipe during construction.

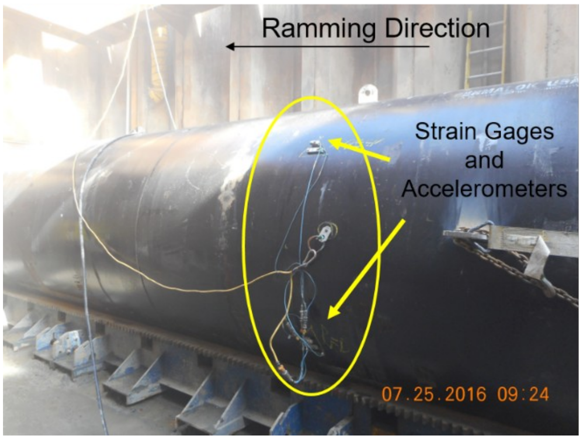

Figure 4 84-inch outer-diameter pipe showing position of strain gages and accelerometers monitored during ramming operations.

The instruments were attached to each pipe section and connected to the data logger that was located on the ground surface to allow real-time data monitoring without the need for personnel within the shaft during the ramming operations.

5. DETAILS OF THE RAM

The contractor chose to use a heavily reinforced cutting shoe that contained Kennemetal bullet bits to provide point loading to break high-strength cobbles and boulders. The bullet bits were nested within the 89mm (3.5-inch) thick shoe in pockets of hardened steel to protect them from breaking in the harsh ground conditions. The specification required the cutting shoe design to meet a nominal casing diameter to cutting shoe thickness ratio (d/t) of 30 (minimum). The pipe had an outer diameter of 2,130 mm (84-inches) and a 32mm (1.25-inch) wall thickness. The cutting shoe was four feet long and was attached to the lead casing that was 6.1 m (20 feet) in length. Figure 5 shows the cutting shoe that was used on the SR 527 Crossing.

A. B. C.

Figure 5 Cutting Shoe. A. Attached to lead pipe section.
B. Top View – welding lubrication line C. Lower section of shoe during fabrication

The contractor chose to use a sand bag pipe plug approximately 3.05 m (10 feet) in length. In addition, jet grouted columns were installed outside of the sheet-pile shaft to allow launch of the ram beneath the groundwater without loss of soil into the shaft. Once the first pipe was rammed, the sand bags were displaced as the pipe filled with soil. The hammer was removed, and the sand bags were removed from the pipeline. The natural soil replacing the sand bags served as the pipe plug and counterbalanced the groundwater head and the tendency for soil to flow within the pipe. During ramming operations, groundwater continuously flowed into the shaft, providing a relief of pressure. However, the soil plug provided a counter-balance to soil flow and the loss of stabilization at the face. Figure 6 show the sand bag pipe plug prior to launch of the ram and the sand bags when the hammer was removed from the back of the first casing.

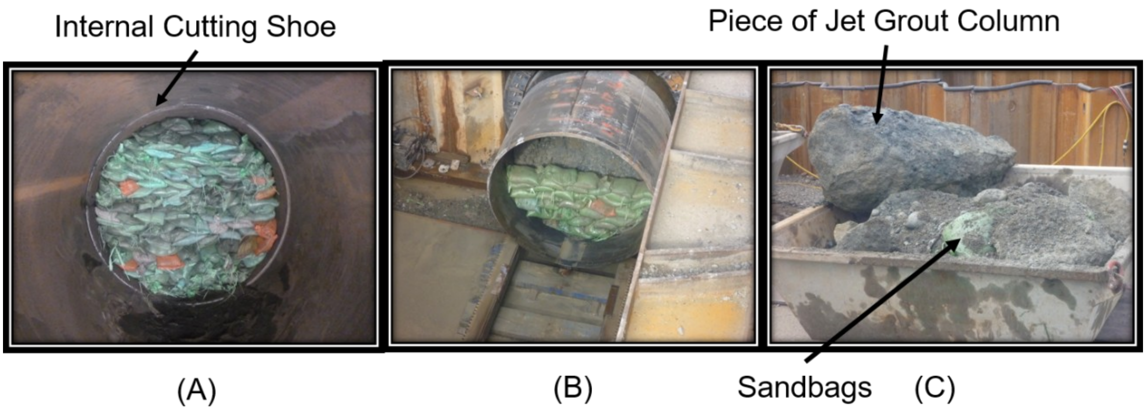

Figure 6 Sandbag pipe plug.
(A) First layer of Sandbags at face; (B) Sandbags pushed to back of pipe after pipe rammed and hammer removed; (C) Portions of the jet grout columns and sand bags found when cleaning out the casing.

The pipe ramming hammer used on the project was a Grundoram Taurus pneumatic hammer manufactured by TT Technologies. The hammer was reported to have a maximum rated energy of 39 Kn-m (29 kip-ft) and a maximum operational speed of 180 blows per minute. During pipe ramming, the hammer was supported by a sled that was attached to the pipe rails that were surveyed into position. This feature was required by specification to ensure that the hammer would be fixed at the design grade during ramming. The designers felt it was important to use a mechanism to align the hammer to minimize any angle introduced at the hammer-casing interface, which has been shown to reduce energy transfer (Meskele and Stuedlein, 2013; 2015a). It was hoped that this feature would increase the grade accuracy of the method. Figure 7 shows the hammer and the cradle/sled that would support the hammer on the pipe rails.

Figure 7 (A) Grundoram Taurus pneumatic hammer. (B) Hammer Sled to hold hammer to design grade.

6. RAMMING ANALYSIS BENEATH SR527 RAM

A total of six pipe segments were rammed for the SR 527 Crossing. As the Taurus® rammed each segment, dynamic measurements were collected from the four pairs of stain gages and accelerometers. The sensors were attached at a distance of approximately 1.5 m (5 feet) from the hammer-casing interface.   The signals from the sensors were processed using a pile Driving Analyzer® manufactured by Pile Dynamics, Inc. A host of parameters were measured while driving the pipe including:

- Penetration Resistance – blows/meter (blows/foot)
- Average Hammer Operation – blows/minute
- Average Transfer Energy – Kn-m (kip-ft)
- Average Transfer Efficiency - % (Based on the rating of the hammer)
- Average Compressive Stress – MPa (ksi)
- Ramming Duration – hh;mm;ss per-6.1 m (20 foot) pipe

The Driving Analyzer ® was used to estimate 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 Studlein 2015a, 2015b). Figure 8 shows the ultimate driving resistance for the ram beneath SR 527.

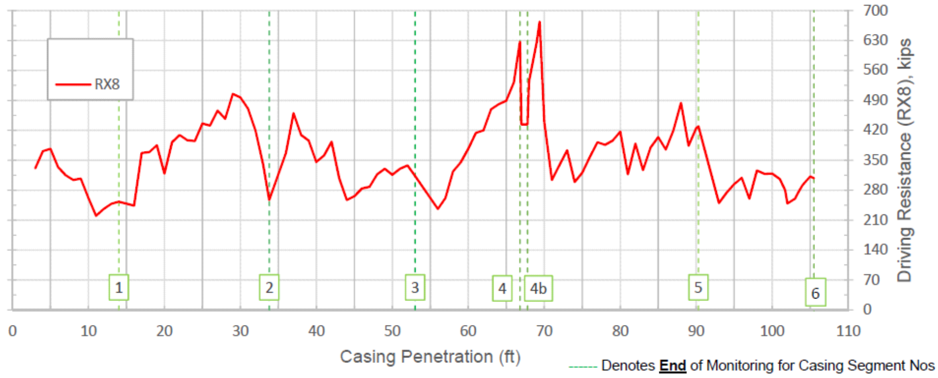

Figure 8 SR 527 – Casing Penetration v. Driving Resistance and Hammer Operation (RMDT, 2016)

At the beginning of the ram, during installation of the first pipe segment, the operator used a considerable amount of caution when ramming through the jet grout columns and did not operate the hammer at the maximum blow rate of 180 bpm (blows per minute). Upon initiation of the ram, significant rebound of the cutting shoe was observed at the interface of the jet grout columns because of the lack of reaction force that could be provided by pipe embedment through frictional resistance (Meskele and Studlein, 2015b). This caused some difficulty getting the hammer to fully seat into the hammer-pipe adapter. The force inferred by the PDA measurements at approximately 1.5 m (5 feet) of penetration was primarily due to resistance at the face. Once the pipe advanced to an embedment of 3.3 m (11 feet,) the jet grout columns and engineered pipe plug was fully engulfed within the pipe and the force reduced from 2,590 KPa (375 kips) to 1,550 MPa (225 kips), where it would then increase linearly, indicative of an increase in frictional resistance along the circumference of the casing, 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 4.3 m (14 feet), leaving 1.5 m (5 feet) of casing within the shaft.

Due to the need for space to accommodate welding and prevent damage to the instrumentation, the pipe ramming was terminated with 1.5 m (5 feet) of casing remaining in the ramming shaft. The shaft was sufficiently long to allow the second casing to be lowered and fitted to the first casing while the first casing remained on the rails. Once fitted, welding proceeded from approximately 8- to 4- o'clock around the crown of the pipe; however, the leading casing had to be rammed forward to allow welding beneath the invert. This added approximately 20 minutes of additional time to each casing installation.

When ramming began on the second casing segment, the pipe had advanced approximately 0.61 m (2 feet) upon which the ramming force increased markedly with very little additional forward penetration at a constant blow rate. Advance rates were measured equal to 9.5mm (3/8-inch) per minute of ramming at this time. Operations were stopped by the operator due to concern about over-excavation; this concern was triggered by the filling of the second casing with soil in the absence of significant penetration. In addition, a significant amount of rebound was observed that that had not occurred since the installation of the first casing. The ramming behavior was indicative of encountering something at the leading edge that was difficult to ram through, such as a large boulder. The contractor decided to attach a come-along to each side of the casing at the spring line from the front wall of the shaft to the end of the casing to apply compressive force on the casing. This was done to reduce rebound, keep the casing pipe pressed to the face of the assumed boulder, and to improve energy transfer (Meskele and Studlein 2013; 2015a). 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 480 MPa (70 kips) due to an instantaneous loss of resistance at the face (see Figure 8 at a penetration distance of 6.1 m (20 feet)). When the muck was excavated from the casing, a fractured piece of a large boulder, approximately 432mm (17-inches) in diameter was recovered from soil within the pipe. It is likely that this boulder caused the stalled casing.

During completion of Segment 2, the penetration resistance dropped markedly and ramming speeds increased. It was at the completion of ramming this segment that surface settlement was noted. It is assumed that the face of the ram lost stability during this segment, largely because the pipe was filling with soil while not ramming on the boulder without casing advance. As a result, the face pressure force was markedly reduced from previous readings, resulting in a reduced penetration resistance measured by the instrumentation. The resulting surface settlement was most significant at approximately 6.1 m (20 feet) from the shaft and a sink-hole approximately 1,016m (40 inches) deep manifested above the crown of the pipeline. The settlement trough followed the pipeline for a length of approximately 5.2 m (8 feet). Luckily, the settlement above the pipe had arrested prior to entering beneath SR 527. Figure 9 presents the variation of the penetration resistance versus penetration depth while ramming Segment 2, identifying the events that took place while ramming. 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).

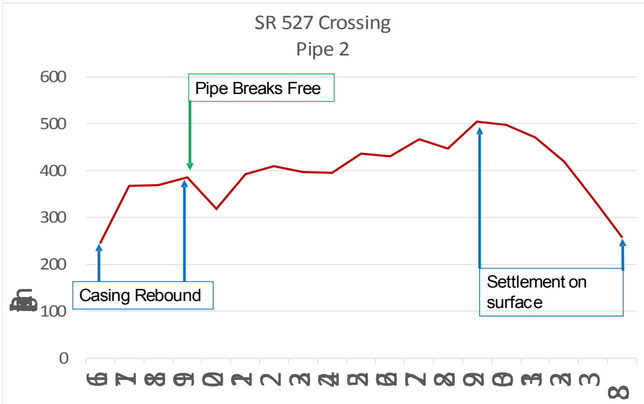

Figure 9 Casing Penetration Depth v. Ramming Resistance and events while ramming Segment 2 on the Highway 527 Crossing

Segment 3 was rammed without any significant events that might instantaneously impact the penetration resistance. The only significant difference between this and the previous segments was that lubrication was introduced on this segment from a single hard-line that was welded to the outside of the pipeline at the crown. The hammer resistance, in terms of blows per foot was uniform and significantly less than they had been for the first two pipe segments. This indicates that the soils encountered during Segment 3 provided a lower face resistance and possibly a lower friction coefficient, resulting in a penetration resistance decrease over the length of the pipeline. From a length of 10.9 to 13.1 meters (36 to 43 feet), the overall penetration resistance decreased by 861 MPa (125 kips), even though the surface area over which the friction was acting increased by one-fourth of the area previously installed. This indicates a large reduction in face resistance. From a length of 1,118 to 1350mm (44 to 53 feet), the ramming resistance increased 365 MPa (53 kips) which is indicative of a steady frictional force increase on the inner and outer surface of the pipe since there were no over-riding incidents taking place during the ramming. The normalized friction over this lubricated section was 0.31 Kg/m^2 (0.07 tons per square foot) of surface area.

There is a significant volume of studies that have been completed on analyzing microtunnel jacking force, normalizing the jacking forces for a variety of pipe materials and frictional soils in an unlubricated state (Staheli, 2006). Using the Staheli (2006) model developed to predict jacking forces on microtunneling, the predicted normalized jacking force for steel pipe in a soil similar to that found on the site with a friction angle of 38 degrees, the predictive model yields 0.41 Kg/m^2 (0.084 tons per square foot) with an estimate of a lubricated normalized frictional loading of less than 0.2 Kg/m^2 (0.04 tons per square foot). There have been comparisons between pipe jacking and pipe ramming friction values; however, the mechanisms that control the forces are markedly different. It is clear from this set and other sets of data studied within this broad analysis that the frictional jacking force model does not predict pipe ramming friction well. In addition, it should be noted that the face pressure force component of the jacking load is markedly different for microtunneling than pipe ramming; therefore, the jacking force model cannot be transferred and used directly to determine ramming resistance.

At the end of Segment 3, soils were removed from the pipe to lower penetration resistance. It was important to leave a plug in the pipe to counterbalance groundwater pressure; therefore, the Contractor elected to remove approximately 15.3 m (50 feet) of spoils – leaving a 4.3 m (14-foot) soil plug. Once the soil was removed, observations of the soil within the casing, as well as line and grade measurements were taken. Interestingly, in the middle section of Segment 3, the observed soil revealed a distinctly layered soil system. This was consistent with the geotechnical conditions identified in 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. It is believed that the face forces on the ram decreased when the ram broke into the alluvial/recessional deposits which exhibited a lower density than the dense glacial soils. Figure 10 shows the soil interface as observed within the pipe.

Figure 10 Soil inside leading pipe segment showing alluvium/recessional soils (brown) underlain by dense glacially overridden soils (gray).

The data collected on Segment 4 revealed the most interesting trend on the project. While ramming Segment 4, the ramming forces went up substantially, increasing to a maximum of 4,619 MPa (670 kips). This can be seen on Figure 8 at a penetration depth between 17.1 and 19.8 m (56 and 65 feet). In addition, ramming rates rose from 11.5 to 46 blows per meter (3.5 to 14 minutes per foot) over the duration of the ramming, and 5,250 blows per meter (1600 blows per foot) was required by the hammer to move Segment 4. The Contractor's data manager came to the shaft and explained to the operator that the data indicated a significant resistance at the front of the pipe. He also noted that the efficiency of the hammer had dropped over 50% in 1.2 m (4 feet) of ramming. These finding were very concerning, and the soil was removed once again from the pipe. Once the casing was cleaned, observations looking down the pipe from within the shaft clearly revealed that the front of the pipe was losing line to the left at a rapid rate. The curve in the pipe was noticeable with a naked eye. Upon measurement, it was discovered that the pipe was just over 610 mm (24-inches) off the design alignment, south of the line, although the grade of the pipe was within tolerance.

Segment 5 was lowered into the shaft and the crown welded; however, when ramming was attempted to move it forward to weld the invert, the segment would not move and the longitudinal weld on Segment 5 split in the shaft, which required repair and resulted in the loss of a day. The split in the weld was approximately 1.8 m (6 feet) long. During ramming of Segment 5, ramming forces continued to be very 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. Once this was completed, the high forces continued for 0.6 m (2 feet) when the pipe "broke free" of an apparent obstruction and the penetration resistance decreased from 680 kips to 2,068 MPa (300 kips).

Soil was removed from the pipe once again prior to ramming Segment 6. Upon entry into the pipeline, the layered soil was no longer observed and the soil within the pipe was sand, gravel, cobbles, and occasional boulders. One boulder found within the spoils measured 381 by 483 mm (15- by 19- inches). Survey on the pipe indicated that the pipe did show a bend during the ramming of Segment 4 but the bend did not continue through Segment 5 and the alignment had continued fairly straight (at the tangent to the angle) after the bend. The trajectory would result in the ram ending significantly off line; however, grade was the significant element of the design. Therefore, the ram beneath the busy highway was 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 base of the hammer would rise as high as 93 degrees Celsius (200 degrees Fahrenheit), at which point the ramming would stop to allow the hammer to cool. Upon final excavation, it was discovered the pipe was 1,372 mm (54-inches) off line which resulted in requiring the installation of a new manhole to allow the sewer system to function properly. Although this was not ideal, the crossing beneath the highway was completed successfully and the grade of the pipe was within tolerance to allow gravity flow of the sewer.

7. RAMMING ANALYSIS BENEATH SILVER CREEK

Upon completion of the SR527 ram, the equipment was turned 180-degrees within the shaft and the ram beneath Silver Creek was initiated. This ram was very short and only required three 6.1 m (20-foot) segments of casing. Sand bags were used as a pipe plug to counteract the groundwater pressure, similar to the SR 527 Crossing. There was much more concern about the groundwater on this drive since this fish-bearing stream could not be dewatered into the casing. Figure 11 shows the variation of driving resistance with penetration depth of the casing.

There are similarities between the driving resistance curves for the SR527 and the Silver Creek crossings that deserve attention. For example, for Segment 1 on the Silver Creek Crossing when casing penetration was low, significant rebound was noted due to the lack of casing resistance with which to react against; correspondingly, a significant amount of rebound was observed. Simultaneously, an initially high amount of face resistance would be noted, followed by a reduction and corresponding slow rate of increase as the friction along the inner and outer surface of the pipe accumulated with increasing penetration length.

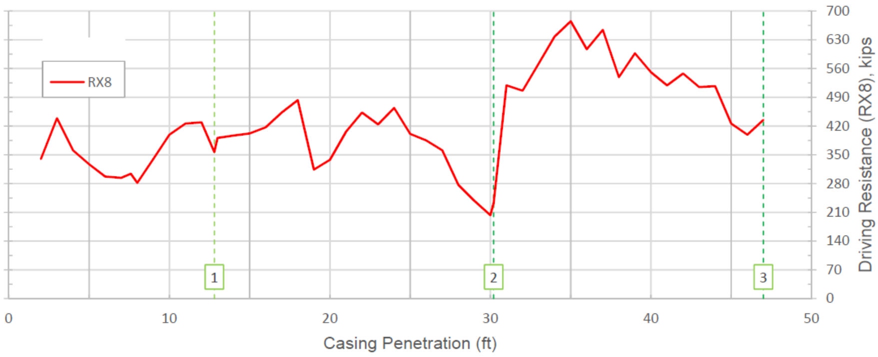

Figure 11 Casing Penetration v. Driving Resistance for the Silver Creek Crossing (RMDT, August, 2016)

The most significant correlation between the two drives occurred during installation of Segment 3 at the Silver Creek installation, when a significant line deviation occurred as observed upon removal of soil. During the grade deviation, the penetration resistance increased from 1,448 to 4,140 MPa (210 kips to 660 kips) – very similar to the increase that was realized on 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 3.1 m (10 feet) over which the casing had bent. The increase in driving resistance was then followed by an overall decrease as the pipe continued forward and the initial high load that was required to bend the pipe was reduced. Figure 12 shows an overlay of the penetration resistance of the two rams.

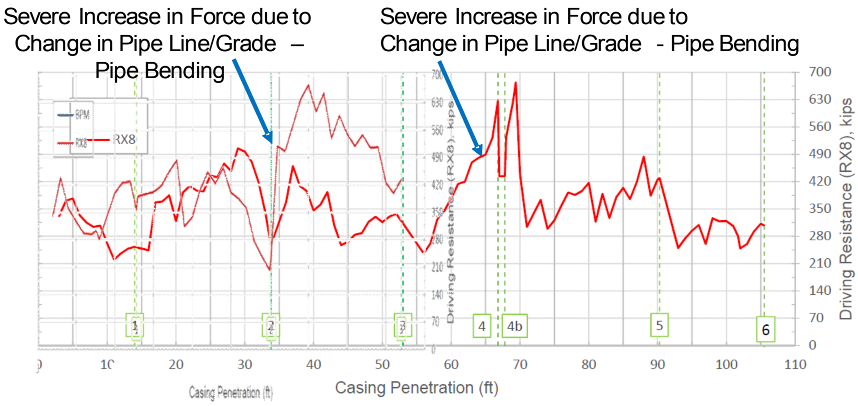

Figure 12 Comparison of Casing Penetration v. Driving Resistance for the Crossings of both SR527 and Silver Creek Crossings.

8. CONCLUSIONS

The data collected on these pipe rams is invaluable to understanding the mechanisms that govern pipe ramming behavior. This paper analyzed only one of the parameters that was computed with a single calculation model. There are a many future analyses to be completed that will assist in a full understanding of the factors that govern pipe ramming behavior. Through these instrumented tests, it was clear that the force on the cutting shoe is a very significant component of the total driving resistance and changes in soil conditions during ramming result in an increase or decrease 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 is important to note the high density of the soil through which the pipes were rammed and the incident of 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.

The most significant factor may be the amount of increase in resistance that was measured when the pipe experienced deviations in design line (and likely in grade as well). With both casings, the overall ramming resistance increased by approximately 3 times the force prior to the pipe bending. This also requires more study but was significant considering the correlation between the two rams.

One last point was the most revealing about the state of the industry and the hunger for knowledge. When the project first began, the contractor was willing to install the equipment as it was part of the contract and tied to a pay item; however, they did not appreciate the full value of the data that would be provided during ramming. By the third segment in the SR 527 crossing, the operator would stop, get out of the shaft, and talk to the instrumentation specialist about the ramming resistance, blows per meter (foot), and the hammer efficiency. This helped him understand what was occurring during the ram and informed the decisions on when to clean out the casing, when there might be an object at the face, when to apply tension on the casing, etc. These instruments were provided at a low cost to the Owner yet provided value to the Contractor to help them construct a successful project, and provided invaluable information for the CM team.

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