

The City of Winnipeg

High Risk River Crossings Condition Assessment Report Water Crossings

Prepared by:

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November 2, 2016

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Dear Mr. Delaurier:

**Regarding: High Risk River Crossings
Condition Assessment Report
Water Crossings**

We are pleased to submit the Final Report for the Water Crossings portion of the project.

We thank you for the opportunity to work on this very challenging assignment.

Sincerely,
AECOM Canada Ltd.



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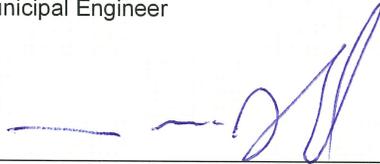
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1. Introduction

The City of Winnipeg's water distribution system and wastewater collection systems, by necessity, traverse the City's watercourses at numerous locations. The City's River Crossing Inventory includes some 70 crossings with water mains and 41 crossing locations with sewer infrastructure that include 56 separate wastewater pipelines. The inventory includes crossings of major rivers, including the Red, Assiniboine, and Seine, to the smaller creeks and drains, including Truro Creek, Sturgeon Creek, Omands Creek, and others.

In 2012 AECOM was engaged by the City's Water and Waste Department to inspect and assess the condition of a number of the assets in the river crossing inventory with very high risk ratings based on previous desktop assessments in terms of combined failure probability and consequence. The intent of this program was to assess the condition of the crossings in greater detail in field studies to provide the City greater insight into the failure probability of the assets than could be afforded in the desktop studies. This included identification of the most likely failure modes and time to failure; providing an estimate of remaining service life.

These factors are intended to be used as guidance for input into the City's overall asset management program and need to include recommended timelines for re-inspection, repairs, replacement, or other remedial works, should they be required.

The program included an assessment of pipeline condition by direct measurement methods as well as an assessment of the pipelines physical environment through geotechnical assessments (i.e. quantifying the risk that the environment around the pipe could cause it to fail as opposed to deterioration of the pipe governing failure). The assessment of physical condition was input into the context of current failure probability by assessing the reduced structural limit states of the pipe versus the applied loads on the pipe. An estimate of remaining service life was made based on measured defects and assessment of the major deterioration mechanisms acting on the individual crossing anticipating the anticipated point in the future that the deterioration would likely result in pipe failure.

The initial project included 19 pipelines at 14 crossing locations, including 13 wastewater and 6 water main crossings. At the outset of the program, little actual physical condition data was available for these pipelines, and it was expected that many of the river crossing assets could be nearing the end of their service life. As the capital cost of replacement and failure consequences of river crossing assets is very high, improving the clarity of current condition, remedial options other than complete replacement, and timing for same was deemed to be a high priority.

Reporting for this program has been divided into two reports driven by the order that the inspections were undertaken. As support to facilitate the inspections came from separate Operations Groups within the City, Wastewater Collection and Water Supply and Distribution, the staging of the work generally focused on coordinating with each Operations Group separately. This report serves to outline the water pipeline crossing inspections undertaken in 2015 and AECOM's condition assessment of these assets. A separate report has been issued addressing the inspection and assessment of the assessed wastewater crossings.

1.1 Program Background

A large driver for the river crossing assessment program was an increased regulatory emphasis to minimize the occurrence of failures on pipes whose resultant loss of fluid would discharge either untreated wastewater or chlorinated water directly to fish habitable waterways. The age of much of the river crossing inventory and the widespread use of ferrous metal pipelines in a native soil environment which is typically very corrosive to ferrous metals, all raise significant concerns over the long term reliability of this infrastructure. Further, many of the river crossings are operationally significant and their failure could compromise the City's ability to maintain desired service levels in the distribution and collection systems.

Previous studies looked into the viability of continuous monitoring of these pipelines through leak detection or other monitoring techniques to quantify failure probability, however, the most cost effective manner to mitigate future failure risk was deemed to do systematic condition assessment using risk-based guidance methods to clarify inspection method and timing^{1,2}.

Understanding how these pipelines can fail is critical to developing the most coherent condition assessment approach. Failure in the context of this program is limited to pipeline failures that will result in a loss of hydrostatic integrity of the crossing. While hydraulic capacity and water quality are reviewed in conjunction with upgrading requirements to rationalize an optimum upgrading approach; loss of hydrostatic integrity is the main criterion for failure of river crossings in this program as it will facilitate the unregulated discharge of chlorinated water directly to a habitable water course.

In this context the most common failure drivers include:

- Material degradation of the pipeline, which either results in direct release of fluid or structural failure of the pipeline due to its reduced ability to resist applied loads.
- A change in applied loads over time, either in isolation or in conjunction with material degradation which results in pipeline failure.
- Buoyancy failure, either due to loss of buoyancy protection over time or the inadvertent introduction of air into a crossing not designed to accommodate air.
- A change in the environment around the pipeline which initiates failure. For the river crossing inventory, the most common environment change around the pipe is driven by riverbank instability phenomena.
- Third party damage. While third party damage is not that uncommon in traditional shallow buried infrastructure in congested rights-of-way it is not a common failure mode for the river crossing inventory due to its relative isolation from other infrastructure.

So the primary focus of condition assessment for the inventory involved:

1. Fully understanding the applied loads on the pipelines (e.g. to understand how much deterioration they can accommodate)
2. Understanding the primary material deterioration risks (both internal and external)
3. Quantifying the means and level of protection against buoyancy or flotation
4. Quantifying the amount of material degradation that has taken place through direct assessment of the pipe
5. Understanding the stability of riverbank crossing and the potential for an active slope movements to engage the pipeline

While items 1, 2, and 3 above are primarily office assessments, Item 4, quantifying the nature and degree of material degradation that has taken place involves the use of direct measurement condition assessment techniques. As the majority of the crossings are ferrous metal pipes, the primary technologies investigated and used for assessment were continuous measurement electromagnetic technologies that had the capability to assess pipe wall loss magnitude, geometry, and spatial distribution of defects. Item 5 involves a combination of field investigation and desktop analysis of river bank slopes in the vicinity of the pipelines in questions. This program did not involve a geotechnical drilling program, required for in depth slope stability work. The intention is to complete in depth

¹ UMA/AECOM, "Trial Program to Monitor Wastewater River Crossings for Leaks in Compliance with Revised Environmental Act License No. 2669E", April 2007

² UMA/AECOM, "Water Main Criticality Study Technical Memorandum 2.1 – City of Winnipeg Watermain River Crossing Design, Construction and Management Standard", July 2011

geotechnical investigations and slope stability analysis as part of a future program if the identified pipelines have a remaining service life that warrants undertaking the additional analysis and potential slope stabilization work.

As all continuous measurement technologies involve getting an in-line inspection (ILI) tool inside the pipeline, a great deal of effort needed to be expended to assess deployment risk and logistics to ensure that the inspection itself did not initiate failure or compromise service level objectives of the system while the inspection was undertaken.

1.2 Program Development

The High Risk River Crossing (HRRC) program involved several review areas and inspection stages prior to the physical inspection programs. The program development generally involved:

- Planning and Risk Assessment
- Selection of Technology
- Geotechnical Assessment
- Inspection Preparation
- Inspection Program
- Condition Assessment

The general program flow is depicted in Figure 1.

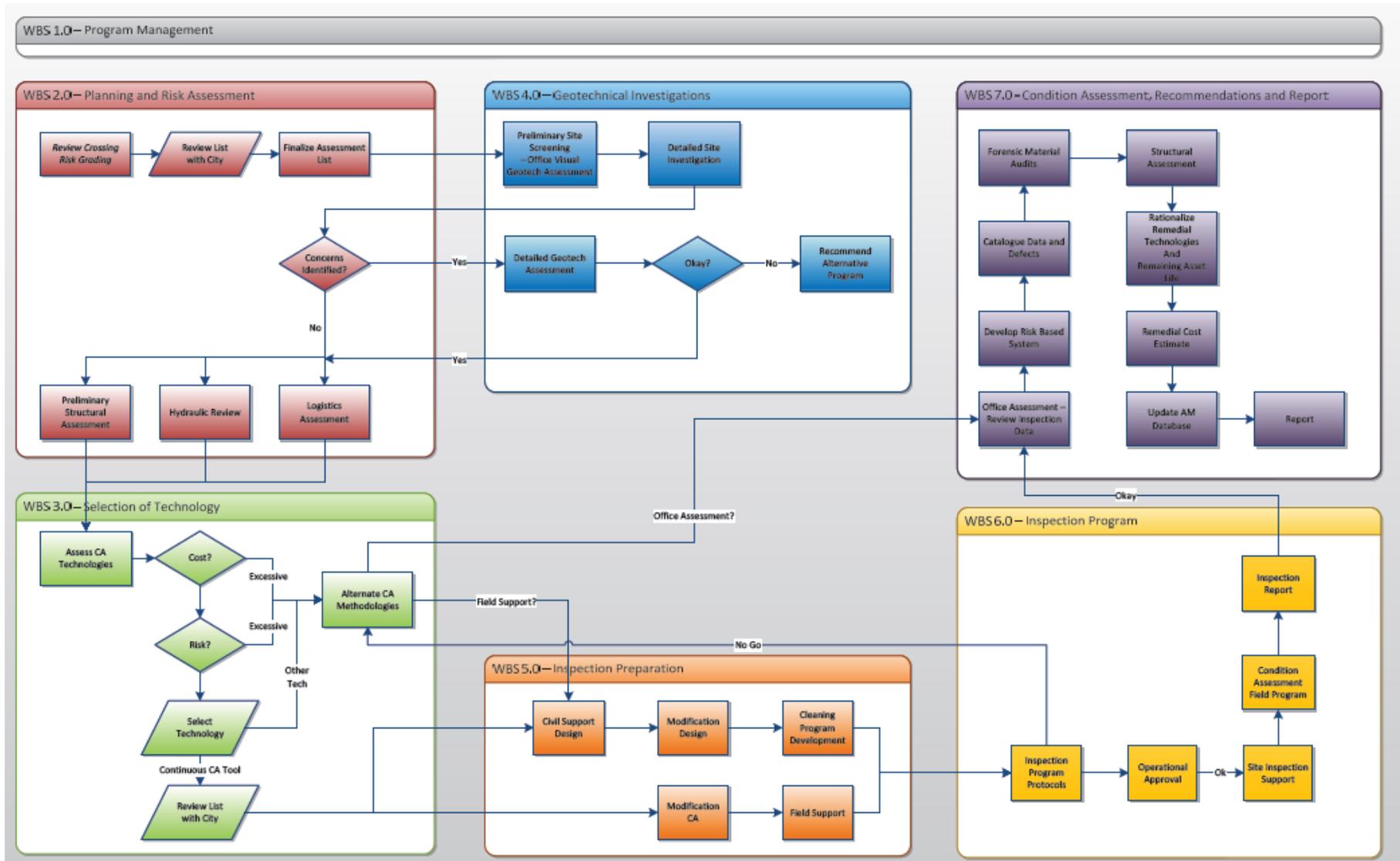


Figure 1: Program Flowchart

2. Planning and Risk Assessment

2.1 Program Finalization

The initial program included 19 pipelines at 14 crossing locations, including 13 wastewater and 6 water main assets, which represent the City's highest risk river pipeline crossings based on the desktop studies. Since award in 2012, the program has evolved for a number of reasons, including reaction to a pipeline failure, the detailed assessment of inspection risk, and the inclusion of an additional wastewater crossing to the program.

The 6 water crossing assets that have been included in this program are identified in Table 1. Of this list, three assets have not undergone physical inspections at time of writing this report, for reasons identified below. However, all assets except for the North Kildonan Feeder Main have undergone geotechnical screening, an operational risk assessment, and initial structural pipeline assessment.

Table 1: Water Crossings Included in Program

Site	Crossing Location	Nominal Diameter (mm)	Material Type	Installation Year
9	Charleswood-Assiniboia Feeder Main*	600	Steel	1965
10	Goulet-Doucet Water Main	400	Steel	1953
11	Kildonan-Redwood Feeder Main*	600	Steel	1955
12	Maryland Bridge Water Main	250	Steel	1970
13	North Kildonan Feeder Main**	600	Steel	1978
14	St. James St. Water Main	450	Cast Iron	1931

* Not inspected, assessed using assumed corrosion rates

** Not inspected or assessed

The following is a summary of changes made to the water crossing program:

- A major leak was found on the North Kildonan Feeder Main in the same time period as the 2012 trial inspection program. The leak was caused by a failed pipe joint (weld) on a section of exposed pipe within the Red River. The inspection was not completed due to structural damage on the pipe and excessive tuberculation within the pipe. The City is currently in the process of replacing the crossing.
- The Charleswood-Assiniboia Feeder Main was removed from the program during tendering of the support contract due to conflicts with the City's water main flushing program.
- The Kildonan-Redwood Feeder Main was removed from the program due to a lack of system redundancy resulting from the North Kildonan Feeder Main being out of service.

2.2 Preliminary Structural Assessment

A major input into the condition assessment program was to conduct an initial structural assessment of each pipeline. The detailed review of each assessment is presented in Section 6 in conjunction with a review of the impact each pipe's current condition on failure probability.

The objectives of the preliminary structural screening included:

- Assess the overall asset factor of safety from a structural perspective to ascertain the pipelines vulnerabilities and its ability to accommodate deterioration or changing load conditions over time.

- To better understand the most probable failure mechanisms such that the appropriate inspection technology or suite of technologies could be selected.
- Assess potential for inspection induced failures. These included the potential for:
 - Inspection induced buoyancy,
 - Inadvertent overpressures, and
 - Pipeline geometry configuration that would preclude the use of some inspection platforms.

2.3 Preliminary Logistics Assessment and Investigation

Preliminary investigations were undertaken in 2012 to assess the access requirements and site conditions. The investigation consisted of walking each site, collecting photographs, and reviewing access considerations for both the cleaning and inspection work. Further, a geotechnical investigation was undertaken to look for signs of bank instability and erosion which is reported on in Section 4 of this report.

As part of the preliminary logistics investigation and technology assessment, the requirements for pre-work was rationalized at each site to facilitate safe deployments. Details of all inspection deployments are outlined in Section 5 of this report.

2.4 Hydraulic Assessment

Hydraulic assessments for the water crossings were undertaken by the City of Winnipeg's water planning staff. Prior to removing the crossings from service, the effects on both the local and regional water systems were reviewed for pressure drops, flow reversals, and other work/outages occurring within the system. The results of these reviews resulted in the removal of two crossings from the program due to conflicts with other work occurring on the system.

2.5 Risk Assessment

An overall deployment risk assessment was conducted for each site, with the intention to highlight and quantify the risks associated with each aspect of each inspection. The risk assessment program included consideration of various risk factors that may potentially occur, as well as a site by site evaluation of risks. Appropriate risk mitigation strategies were developed, such that inspections could be conducted in a safe manner at a mutually agreeable level risk from an operational perspective. The complete technical memorandum issued to the City is attached as Appendix A. The following items were considered during the risk assessment:

2.5.1 Loss of Capacity or Service During Cleaning and Inspection Operations – Low to Medium Risk

The water main crossing shutdowns in general did not pose any significant risk with respect to loss of service, provided shutdowns are coordinated in low demand conditions and do not conflict with other system operations. AECOM does not have sufficient information on the water system hydraulics to analyse system wide effects, however, our good practical understanding of the systems operational requirements allowed us to identify potential conflicts during the preliminary planning stages. This included the compounded effects that staging multiple crossing shutdowns will have on the system and each other. The proposed shutdown schedule and requirements were passed on to the City for modelling and comment.

2.5.2 Potential to Aggravate Existing Defects in Deteriorated Pipelines – Low Risk

Aggravating existing defects was considered a low risk as long as ILI inspection technologies were selected that did not require direct contact with the pipe wall for the assessment tool. Cleaning requirements for a technology such as

Remote Field Technology (RFT) is considerably less aggressive than cleaning requirements for an ILI such as magnetic flux leakage (MFL) or broadband electromagnetic (BEM); as an RFT tool is not required to make direct contact with the pipe wall to facilitate the inspection. RFT inspection tools can generally pass through obstructions up to 25 mm or more less than pipe's internal diameter and don't have their accuracy compromised by nominal amount of tuberculation. The relaxed cleaning requirements allow for the use of traditional sewer flushing technology and pigging using soft to medium density foam pigs as opposed to scraper pigs, chain flailers, etc., which would be required with MFL or BEM tools. Thus aggravating existing defects is considered a low risk operation as long as it can be accommodated in inspection equipment selection.

2.5.3 Obstruction of Pipelines Due to Equipment Getting Stuck During the Cleaning and Inspection Process – Low Risk

Obstructing of pipelines with equipment during both the cleaning and inspection process was considered a low risk activity as long as reasonable controls were put in place during cleaning operations.

The cleaning process was undertaken using a combination of traditional high pressure flushing technology and foam pigging. New sanitized flushing equipment was utilized to remove debris from the pipeline, and foam pigs used to confirm the adequacy of the cleaning operations and swab the pipe for debris. Tethered pigging was utilized by Tri-Core Projects Manitoba Ltd. (Tri-Core), the support contractor, for all crossings, further reducing the risk associated with cleaning operations. Tri-Core encountered issues with the strength and manufacture of the tow ropes on their foam pigs used on the water crossing sites, which resulted in them losing a pig within the Goulet-Doucet water main crossing. Issues with the foam pigs were rectified with the installation of 12.5 mm steel cable and steel plates onto the existing foam pigs, see Figure 2



Figure 2: Modified Foam Cleaning Pig

To reduce the risk of getting a tool stuck within the pipeline and/or damaging the tool; a gauge pig with aluminum fins was pulled through the line prior to the inspection. As an example, Figure 3 shows a gauge pig with bent fins, caused by a partially closed gate valve on the Munroe-Polson sewer siphon.



Figure 3: Gauge Pig with Foam Swab

2.5.4 Buoyancy of Existing Pipelines – Medium Risk

Many of the pipelines to be inspected under this program are located either above the river bottom, or dredged into the channel at shallow depths. Some pipelines are weighted with concrete weights, grout bags or other devices, however, the safety factors against floatation were found to be quite low if excessive air was introduced into the lines. To mitigate this risk a procedure was developed to ensure the pipelines were full behind the cleaning and inspection equipment by backfilling with water as required.

3. Technology Selection

A key element of the HRRC program was selection of appropriate inspection technology for direct assessment pipeline condition. The pipelines initially identified in the HRRC program consisted of 18 ferrous metal pipelines [17 steel and one cast iron (CI)] and one thermoplastic high density polyethylene (HDPE) pipeline. Specifically, the Request for Proposal for the project suggested consideration of continuous electromagnetic inspection methods such as Remote Field Technology (RFT) or Magnetic Flux Leakage (MFL) and/or acoustic techniques to confirm whether river crossings were actively leaking. The objective of the program was to be proactive and focus on improving condition certainty to as great a degree as possible with due consideration to the cost effectiveness of the technology relative to the replacement cost of the river crossing asset. A review of an array of technologies was undertaken to confirm availability and applicability of the technology, and to provide specific recommendations on the most suitable technology (or suite of technologies), given data capture objectives, site specific conditions, deployment risk, and “all-in” deployment and assessment cost.

An ever-expanding array of tools and technologies are available to the municipal industry for use in inspecting and evaluating the physical and operational condition of pipes. Tools such as closed-circuit television (CCTV) and sonar technologies can be used to gather valuable information to supplement the condition assessment process or assessment of deployment risk for any type of pipe material. Their use in condition assessment, however, is limited to “visual” classification of defects on the interior face of the pipe and in the way pipe is reacting to the soil stresses around it. In gravity pipes this is often an adequate level of assessment, however, in pressure pipe flow, even when the pressures are low, more quantitative data on residual pipe structure is a desired outcome of the condition assessment process.

Material-specific inspection technologies such as electromagnetics (EM) were developed to acquire quantitative physical information on residual pipe structure and/or to detect specific types of defects and defect geometry in ferrous metal pipes. Although deployed internally they can be used to obtain quantitative data of physical condition beyond the internal surface of the pipe. Unfortunately, electromagnetic tools are typically very costly to deploy and inherently introduce some risk during the deployment process which needs to be understood, mitigated, and managed carefully. As well, there is a wide degree of variance in the ability of various tools to identify defects accurately. This balance of accuracy, cost, and deployment risk is one that needs to be considered thoroughly in order to select the correct inspection platform for each application.

3.1 Electromagnetic (EM) Inspection Technologies

Three electromagnetic (EM) technologies were reviewed based on their ability to meet defect resolution objectives, to cover the full diameter range of the inventory, and to be applied in a continuous assessment mode to inspect the entire river crossing. These included:

- Magnetic Flux Leakage (MFL)
- Remote Field Technology (RFT)
- Internal Broadband Electromagnetic (BEM)

MFL and BEM have very similar levels of accuracy and identical deployment risk profiles. For brevity, only MFL is discussed in detail herein. The rationale for exclusion of BEM is noted under alternative technologies reviewed.

3.1.1 Magnetic Flux Leakage

MFL was the first EM technology utilized on in-line inspection (ILI) tools for the detection of metal loss corrosion and MFL tools are still the most commonly used ILI tools for inspecting the physical condition of pipelines in the oil and gas industry. The technology utilizes a powerful magnet placed at the surface of the pipe to induce a static electromagnetic field, or magnetic flux, within the pipe wall between the poles of the magnet (see Figure 4). If the magnet is strong enough to saturate the wall with flux, and the wall is homogenous and contains no defects, the magnetic flux will be undisturbed and uniformly distributed within the wall. If, however, the wall contains reductions in wall thickness, surface-breaking cracks / fractures or pipe joints, the magnetic field will become distorted and the flux will “leak” beyond the pipe surface and can be detected by sensors that are located near the pipe surface between the poles of the magnet.

While very accurate at detecting both internal and external defects in ferrous metal pipelines, MFL technology has several drawbacks:

- The magnets require direct (or near direct) contact with the pipe surface in order to develop the magnetic flux field. Thus pipelines must be completely free from debris, tuberculation. While the tool can work through inner coatings such as cement, epoxies, and coal tar, the surfaces need to be clean and smooth.
- Tools require approximately 3m clear space in front of the insertion/extraction points to facilitate insertion and extraction.
- Tool insertion requires the installation of a reducing fitting that will aid in compressing the tool into the line.
- Bends on the pipes being inspected must be smooth and have a minimum axial radius of 1.5 diameters; MFL tools **cannot** pass through mitered bends.
- The mass of the magnet array and strong magnetic attraction between the tool and the pipeline make propelling tools through lines using water pressure difficult.

As the profile of almost every crossing has significant alignment variation, MFL was considered to have excessive deployment risk for this application.

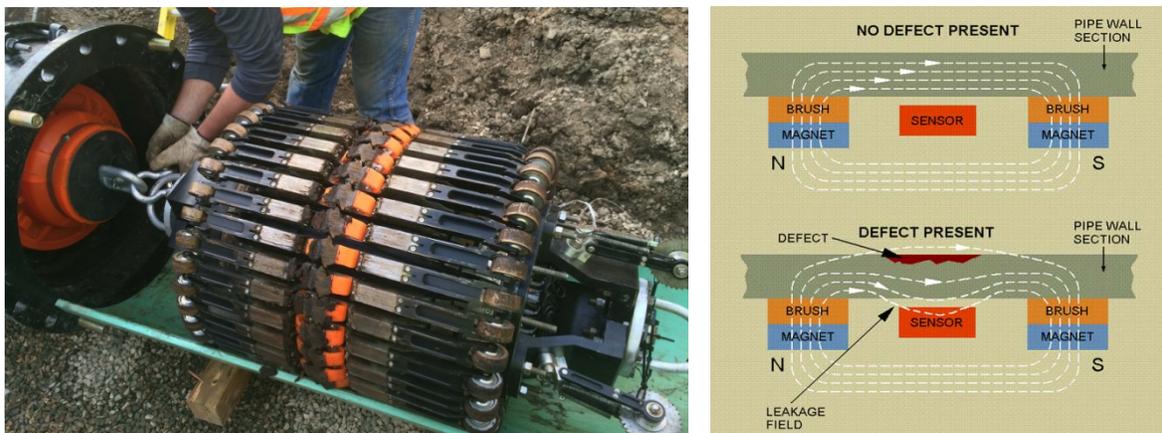


Figure 4: MFL ILI Tool

3.1.2 Remote Field Eddy Current

Similar to MFL, RFT technology utilizes an exciter coil on one end to emit an electromagnetic field into the pipe wall with detectors on the other end to pick up the signal (see Figure 5). Changes or distortions in the signal relate to

defects in the pipe wall. Interpretation of the detector data will identify joints, fittings, holes, cracks, and other defects in the pipe wall.

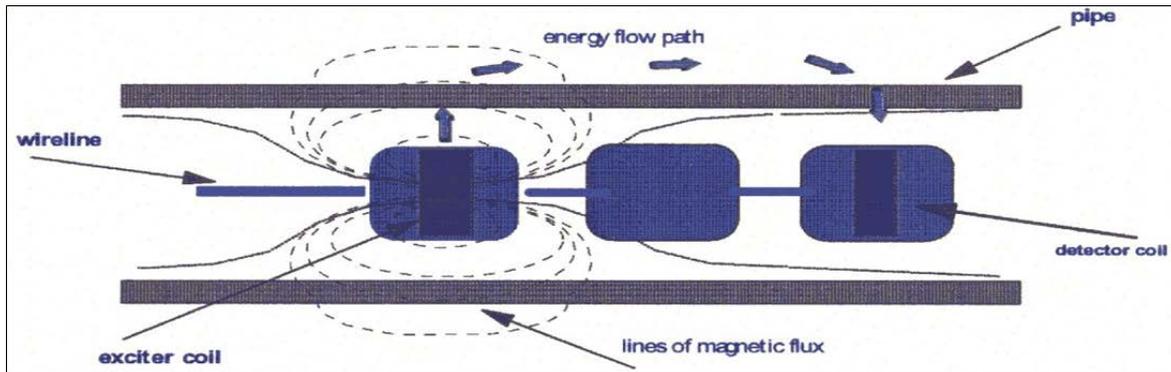


Figure 5: Remote Field Eddy Current Inspection Diagram

RFT tools have a lower level of defect resolution than MFL but usually adequate for the target defect size in low to moderate pressure applications (e.g. 100's of psi as opposed to 1000's of psi). Their deployment risk profile has some significant advantages over MFL as follows:

- RFT tools do not require direct contact with the pipe wall. RFT tools are typically designed to be 25 mm or more less than the internal diameter of the pipeline. This allows RFT tools to pass small to moderate obstructions, negotiate bends and deflections commonplace in municipal systems and operate in lined pipelines with minimal risk of damaging the coatings.
- RFT tools can be launched from relatively tight spaces (see Figure 6).
- RFT tools can navigate standard 90 degree cast iron fittings and up to 45 degree mitered joints.
- RFT tools are relatively light and can be deployed as a free swimming tool with a tow pig. Further, due to their relative open design, they can permit water to bypass the tool during inspection.

Based on this, RFT, utilizing PICA's deployment platform, was deemed to meet both the technical objectives for defect resolution and could be deployed at acceptable risk levels considering all site constraints.

3.2 Alternative Inspection Technologies Reviewed

In addition to the EM technologies discussed in 3.1 the following alternative inspection technologies were reviewed, and are summarized in Figure 7:

- Broadband Electromagnetic (BEM): BEM has a very high resolution of defects, similar to MFL. However, it needs to make physical contact with the pipe wall and cannot pass through mitered bends or sharp bends in general. It was not considered further.
- Enhanced Electromagnetic (EEM): This is a direct reading electromagnetic tool offered by Pure Technologies, with sensors mounted on flexible tines. The technology has a high level of flexibility to navigate geometrically complex pipelines. The tools as available at the time of inspection, however, had very coarse defect resolution, estimated to be approximately 100 millimetres and over 50% wall depth on a 600 millimetre pipe (i.e. it would not be able to detect defects smaller than 100 mm in area and with less than 50% wall loss). Future EEM platforms should be reviewed as they may evolve to match RFT accuracy and deployment ease.
- Ultrasonic: Inline ultrasonic tools have been developed for the oil and gas industry and work by measuring the time it takes an ultrasonic pulse to travel from one side of the pipe wall to the other. While providing sufficient

accuracy in determining wall thickness and defects, the technology has only been deployed in the municipal market in 150 mm and 200 mm diameter pipelines. Additionally, ultrasonic tools require all debris and buildup to be removed from the pipe in order to maintain a set offset from the pipe wall.

- Guided Wave UT: Also known as long-range ultrasonics, this technology was developed to inspect long lengths of above ground piping in an industrial setting. It works by measuring the time it takes a wave pulse to travel along the pipe wall and bounce back from defects, welds, etc. Defects are not captured in sufficient resolution to meet accuracy objectives for this project and the length of pipelines that can be assessed would not cover entire crossings.
- Magnetostrictive Sensor (MSS): MSS utilizes an exterior metal collar to generate a guided-wave and which it uses to detect changes in the material. Its use in buried pipes is limited to approximately 20 m from the deployment location and its defect resolution would not meet project objectives.
- Sonar: Sonar may be used to determine the dimensions of submerged objects both interior and exterior (e.g. via internal inspection or via river bottom scanning). While useful in screening pipelines for debris buildup and excessive deflection, buckling, etc.; it does not provide insight into corrosion and remaining pipe life for ferrous metal pipes.
- Leak Detection: While useful in determining if leaks are present, leak detection technology does not provide insight into corrosion or remaining pipe life. Further for the wastewater and force main crossings, leak detection tools such as SmartBall and Sahara require a minimum pressure of 90 kPa (13 psi) to detect leaks accurately.
- Closed Circuit Television (CCTV): Useful for inspection of non-submerged pipelines. CCTV however, does not provide insight into deterioration within or on the outside of buried pipelines. Again, useful as a screening technology and to assist in facilitating safe deployments but not the degree of condition assessment accuracy required for the program.

3.3 Recommended Inspection Technology for Ferrous Metal Pipes

Given the advantages outlined in Section 3.1, RFT was selected as the preferred continuous EM inspection technology as offered by PICA. PICA's 600 mm Chimera inspection tool is shown in Figure 6.



Figure 6: PICA's 600 mm Chimera RFT Tool

Crossing Inventory						Electromagnetic				Ultrasonic		CCTV		Leak Detection		
CoW #	Location	Asset Function	Crossing Channel	Pipe Diameter (mm)	Pipe Material	MFL (Pure)	RFT (PICA)	EEM (Pure)	BEM (Pure)	Guided Wave	Point Test	Crawler	Tether (Sahara)	Free (SmartBall)	Tethered (Sahara)	Fixed (LeakFinder)
9	Charleswood-Assiniboia Feeder Main	Feeder Main	Assiniboine	600	Steel	B	●	T	●	●	●	●	●	●	●	●
10	Goulet-Doucet Water Main	Water Main	Seine	400	Steel	T	●	X	●	●	●	●	●	●	●	●
11	Kildonan-Redwood Feeder Main	Feeder Main	Red	600	Steel	B	●	T	●	●	●	●	●	●	●	●
12	Maryland Bridge Water Main	Water Main	Red	250	Steel	B	●	X	●	●	●	●	●	●	●	●
13	North Kildonan Feeder Main	Feeder Main	Red	600	Steel	B	●	T	●	●	●	●	●	●	●	●
14	St. James Street Water Main	Water Main	Assiniboine	450	Cl	T	T	X	●	●	●	●	●	●	●	●

●	Tool exists and able to inspect
○	Inspection feasible if line is dewatered
X	Survey not possible or feasible
B	Tool exists but inspection may not be feasible due to bends in the pipework
T	Tool needs to be developed
T B	Tool needs to be developed; inspection may not be feasible due to bends in the pipework

Figure 7: Inspection Technology

4. Geotechnical Reviews

4.1 Overview

Geotechnical reviews were performed for all water crossings in order to assess the current and potential future risk of bank failure and pipe engagement. There were two phases of review:

- **Site inspection:** visual inspection of the riverbanks was performed in order to determine the existing condition of bank slopes at each of the water crossings, including existing slope failures. This phase also included a review of existing information, including geotechnical mapping, and relevant subsurface information and construction drawings of adjacent structures where available. Areas of concern were noted, and investigated further in phase two.
- **Slope Stability Analysis:** for crossings where potential bank instabilities were identified, a preliminary slope stability analysis was performed to assess the magnitude of slope failure, and the potential impact on underground infrastructure. This modelling approach allowed for assessment of risk to the crossing infrastructure regardless of physical condition.

For the highest risk scenarios, where pipeline failure is potentially inevitable regardless of physical condition, the nature of stability remediation needed to be considered. Serious bank instabilities requiring extensive stabilization may result in a relocated pipe asset, irrespective of its condition. Minor instabilities present would still likely require condition assessment but would need remedial works that incorporated both slope stability and consideration of any pipe rehabilitation requirements.

Results of the initial screening are presented in Appendix B and preliminary stability analysis in Appendix C. This section will provide an overview of these findings.

4.2 Site Inspection

Water crossings summarized here are numbered as Sites 9, 10, 11, 12, and 14, crossing the Assiniboine, Seine, and Red Rivers (waste water crossings have been discussed in a separate report). At each site, experienced geotechnical personnel surveyed the riverbank areas at both banks, identifying, documenting, and photographing slope features that are potentially linked to bank instability. Site inspection is a good first step toward assessing the stability and pointing toward whether or not further information gathering in the form of subsurface characterization and monitoring is necessary. For example, monitoring of slope inclinometer casings and groundwater pressure can provide definitive measurements as to whether or not displacements in the bank are presently occurring.

The precise prediction of the moment and magnitude of slope failure is beyond the current extents of geotechnical knowledge. That being said, reasonable judgement can be provided for banks that have a high probability of failing or being unusually active. Site inspection provides a succinct snapshot of the present condition that is relevant to that moment in time. It captures the present state of the terrain in relation to active displacement occurring on the bank, and whether or not there is high or low potential for instabilities developing. Displacements may have occurred in the past, and these can be more difficult to interpret, due to the masking effects of erosion and vegetation growth. However, an experienced eye can often discern past instability features.

The following is a summary of the bank inspection results in order of site number. The observations are summarized in Figure 8. A common observation across almost all of the sites was the lack of erosion protection at the river edge. Erosion protection, typically in the form of loose rip rap, offers a large degree of protection for a relatively low economical cost. Frequently, the beginnings of larger bank failures can be traced back to erosion of the bank toe;

which usually promotes retrogressive failures progressively up the bank with increasing severity. Providing erosion protection can often arrest the onset of toe erosion and the development larger bank instabilities.

9. **Assiniboia Feeder Main:** No erosion control measures in place. The north bank exhibits no evidence of instability or structural distress. The south bank exhibits possible slumping at the toe (although this may simply be an erosion feature), and does not exhibit any instability in the upper bank area.
10. **Goulet Doucet Water Main:** No erosion control measures in place. The west bank exhibits minor toe erosion, but does not exhibit erosion in the upper bank area. The east bank exhibits no evidence of instability.
11. **Kildonan Redwood Feeder Main:** The west bank is treated with grouted rip rap at upper bank. Although scarp and lower bank instabilities are present, the pipe crossing begins at the bottom of a deep shaft, and is not contained within the riverbank slope. The east bank is steep, and exhibits severe upper bank instability and erosion (2.4 m deep scarp at the river edge). The pipe is contained within the slope of the east bank, and is thus at severe risk of damage due to slope failure.
12. **Maryland Bridge Water Main:** No clear erosion control measures in place (possible toe treatment). Both banks have steep upper banks leading to bridge abutments. Drainage down both bank faces has led to deep erosion gulleys (600-800 mm deep). There is moderate risk of damage to the pipe at both banks.
14. **St. James Water Main:** No erosion control measures in place. The north bank is very steep at the upper end, flattening out downward until it reaches a negative slope at the near river's edge, ending in a 1.8 m tall scarp at the river's edge. The scarp and apparent continued erosion pose moderate threat to the pipe. The south bank is very steep at the upper edge, while the toe experiences continued erosion and exhibits multiple slump blocks which were likely formed recently. A 600 mm tall head scarp was observed, as well as a 400 mm wide by 400 mm deep tension crack. It also appears that this bank is being undermined by continued riverine erosion processes. The apparent worsening trend of bank stability poses a severe threat to pipe integrity.

CROSSING ID NUMBER	LOCATION	PIPE FUNCTION	RIVER	FIRST PIPE			SECOND PIPE (IF DUAL PIPE CROSSING)		SIDE OF RIVER	NEIGHBOURING STREET	SOIL TYPE			SCARP PRESENT ON ALIGNMENT		SCARP PRESENT ON NEIGHBOURING PROPERTIES		UPPER BANK INSTABILITIES EVIDENT		LOWER BANK INSTABILITIES EVIDENT		TOE EROSION		RIP RAP AT RIVER BANK TOE		IF RIP RAP EXISTS, RIP RAP COVERAGE EXTENDS SUFFICIENT DISTANCE AWAY FROM CROSSING		TEST HOLE (TH) OR SLOPE INCLINOMETER (SI) PIPE PROTECTIVE CASING		BRIDGE ADJACENT TO CROSSING		SIGNIFICANT ISSUES WITH THIS BANK (1-X IS LEAST AND 3-X IS MOST SEVERE)
				PIPE DIAMETER (mm)	PIPE MATERIAL	INSPECTION LENGTH (m)	PIPE DIAMETER (mm)	PIPE MATERIAL			INSPECTION LENGTH (m)	ALLUVIAL	LACUSTRINE	BOTH ALLUVIAL AND LACUSTRINE	EXIST	NOT EXIST	EXIST	NOT EXIST	EXIST	NOT EXIST	EXIST	NOT EXIST	EXIST	NOT EXIST	EXIST	NOT EXIST	YES	NO	EXIST	NOT EXIST	EXIST	
9	Charleswood-Assiniboia Feeder Main	FEEDER MAIN	ASSINIBOINE	600	Steel	185			NORTH	ASSINIBOINE AVE			X		X	X			X		X		X				X		X		X	
									SOUTH	SOUTHBOINE DR			X	X			X		X		X		X		X		X		X			
10	Goulet-Doucet Water Main	WATER MAIN	SEINE	400	Steel	120			WEST	GOULET ST			X		X	X			X		X		X				X		X		X	
									EAST	DOUCET ST			X		X		X		X		X		X		X		X		X			
11	Kildonan-Redwood Feeder Main	FEEDER MAIN	RED	600	Steel	250			WEST	MAIN ST		X			X	X			X		X		X		X		X		X		X	
									EAST	GLENWOOD CRES	X			X		X		X	X		X		X		X		X		X		X	
12	Maryland Bridge Water Main	WATER MAIN	ASSINIBOINE	250	Steel	135			NORTH	WOODROW PL		X		X		X			X	X		X		X			X	X		XX		
									SOUTH	WELLINGTON CRES	X			X		X		X	X		X		X		X		X		X			X
13	North Kildonan Feeder Main	FEEDER MAIN	RED	600	Steel	215			WEST	SCOTIA ST			X		X	X			X	X		X		X		X		X		X		
									EAST	HENDERSON HWY	X			X	X		X	X		X	X		X		X		X		X		X	
14	St. James Street Water Main	WATER MAIN	ASSINIBOINE	450	VCI	185			NORTH	WOLSELEY AVE W	X			X		X			X		X		X		X		X		XX			
									SOUTH	WELLINGTON CRES		X		X		X		X	X		X		X		X		X		X		X	

Figure 8: Geotechnical Site Investigation Summary

4.3 Stability Analysis

The results of the stability analysis presented in Appendix C represent a portion of these crossings and include both sewer and water main crossings. The water crossings will be discussed here while the sewer crossings have discussed in a separate report.

Stability analysis was performed to assess the factor of safety against slope instability. Correctly analyzed slopes with very low factors of safety have a very high probability for active movement and ultimately failure while higher factors of safety represent much lower probability of active movement. Modeling is a simplification of the real-world mechanisms, particularly given the two-dimensional modeling methods used to represent a three-dimensional world. The variability of soil and groundwater conditions, the strength of the soil and porewater pressure effects, and the variability in river levels conspire to make this a complex problem. Modeling in greater detail involves considerably greater cost, which is only worthwhile if the increased cost yields a commensurate increase in the understanding of bank behaviour.

Banks at select water main crossings (Sites 11 and 14) were chosen for further stability analysis based on the findings of the site inspection. Banks that showed evidence of active instabilities or that contained geometries that are steep in relative terms of overall bank stability were chosen to examine the relationship between potential failure geometry and the pipe infrastructure. Shallow surficial displacements, though of concern, are not as much of a concern as deep-seated displacements that could directly engage the pipe. Surficial and toe displacements can be a concern when these progress unchecked and are allowed evolve into a deep-seated failure.

A factor of safety value ranging from 1.3 to 1.5 is considered to be stable and generally acceptable for infrastructure. Critical infrastructure that has a very low risk tolerance for being out of service will generally require a factor of safety of 1.5. Within the range of 1.0 to 1.3, the bank will likely exhibit a higher degree of creep displacement. A factor of safety below 1.0 implies failure.

Table 2 summarizes the factor of safety for a failure plane that engages the pipe, illustrated as a function of river elevation in Figure 9. In the case the two water crossings analyzed, Site 11 is generally stable at the upper bank area, but prone to toe instability, and Site 14 is prone to slope failure due to the near-vertical slope face at the river's edge. Both sets of modelling results reflect observations made during the site inspections.

Table 2: Global Factor of Safety for Water Crossings where the Potential Failure Surface Intercepts the Pipe Alignment

Site	Location	Bank	River or Creek	Factor of Safety Affecting Pipe
11	Kildonan-Redwood Feeder Main	East	Red River	1.3 to 1.6
14	St. James Water Main	South	Assiniboine River	1.1 to 1.2

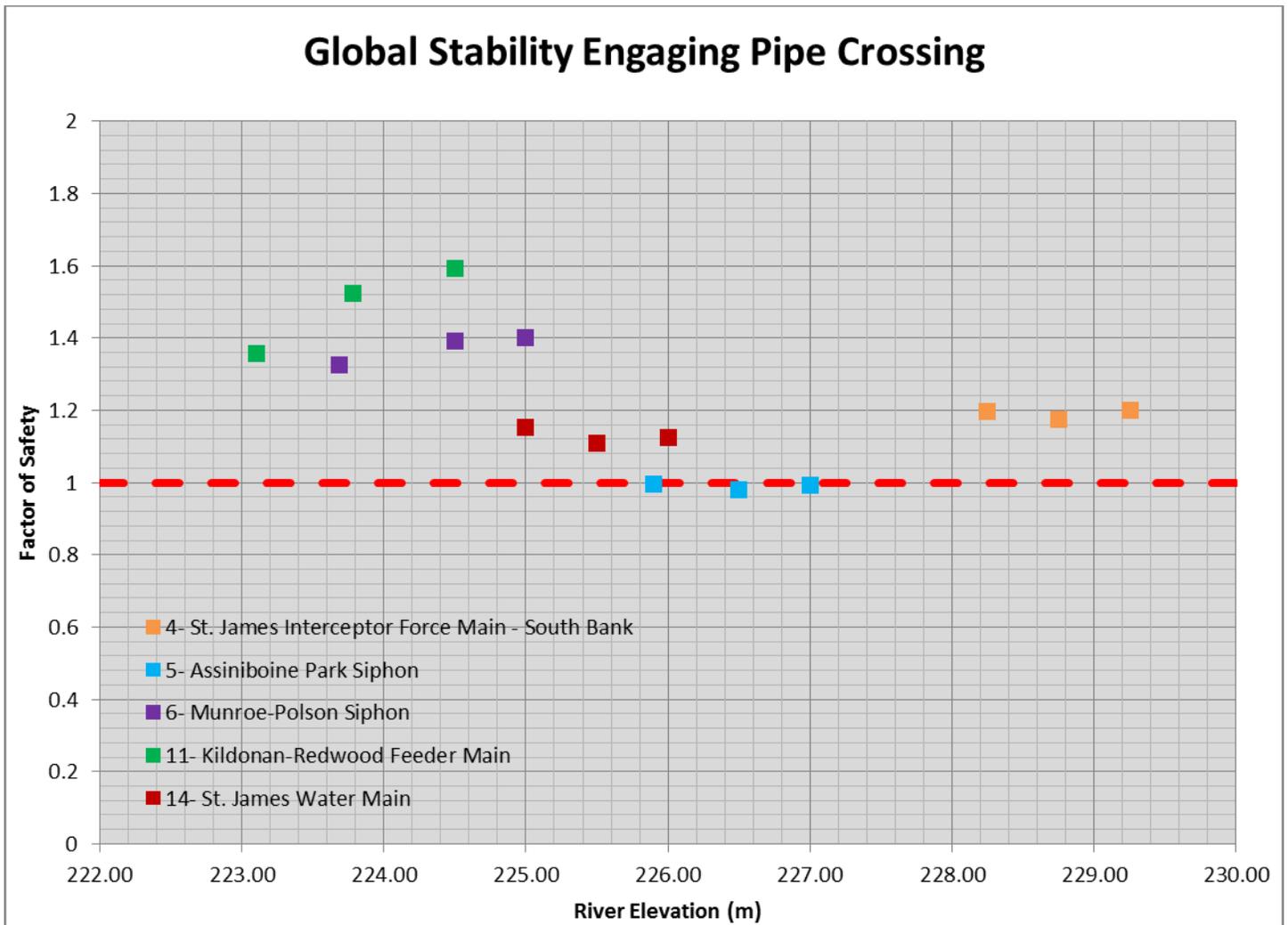


Figure 9: Global Stability Factor of Safety against Bank Failure where Failure Surface Contacts Pipe Profile

The analysis has demonstrated a potential failure surface near the riverbank toe that could engage the pipe at Site 11, Kildonan-Redwood Feeder Main. With continued erosion this can result in retrogressive failure farther upslope as support is reduced for upper bank soil blocks. Erosion measures should be implemented to reduce the potential for toe and subsequent upper bank failure. Global factor of safety ranges from about 1.1 to 1.2 based on this isolated cross-section and reinforce the potential for global instability.

At Site 14, St. James Street Water Main the pipe elevation profile consists of shallow burial within the river channel and a till surface coincident with the river channel bottom. The potential failure surface based on the analysis may engage the pipe where the pipe elevation rises into the riverbank clay further inland, see Figure 10. At present the observed failure is confined to the oversteepened face of the riverbank with failure blocks sliding along the till surface. Arresting the erosion and flattening the slope face should be implemented to prevent the failure current failure surface from progressing inland and causing a failure surface which engages the pipe.

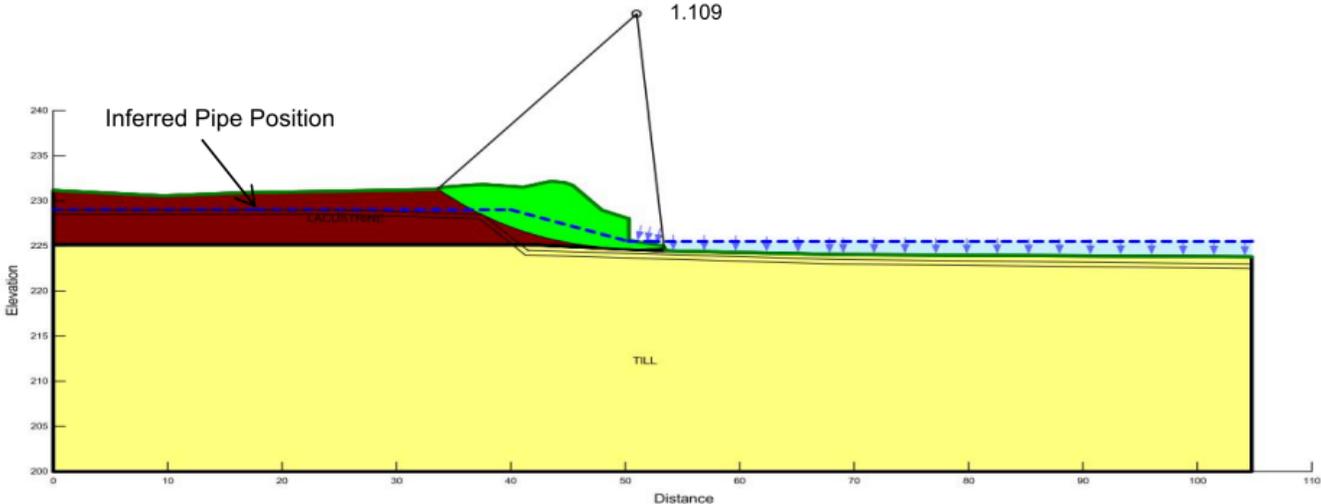


Figure 10: Slope Failure Planes – St. James Water Main

5. Field Inspection Program

5.1 Overview

Field inspection of the water crossings in the finalized work program were undertaken during one mobilization in the Fall of 2015. A total of 3 pipelines were successfully inspected.

An initial trial program in November/December 2012 was targeted to sites which PICA had tools available. The North Kildonan Feeder Main was selected for inspection as part of the trial program. Unfortunately, failure of the feeder main occurred prior to mobilization, and resulted in abandonment of the inspection.

The second mobilization occurred in October of 2015, and included successful inspection of three sites. A summary of sites inspected is included in Table 1. The original program schedule as per AECOM/ PICA proposal spaced inspections over four mobilizations, through the Fall of 2013, to allow for tool development and availability. Delays in inspection of both the sewer and water sites were caused by several factors including conflicts with inspection vendor schedule, delays in tool development, seasonal river flooding, and Operations Department approvals to support access to the pipe, and other factors.

5.2 2012 Program

The 2012 inspections sites were selected based on availability of RFT tools at the time, and also focused on site where minimal access modifications were required and deployment risk was low. Tri-Core was engaged directly by the City of Winnipeg to provide pipeline access, modification, and cleaning services. Inspection of the following pipelines was attempted:

- Site 1a – 600 mm St. James Interceptor Siphon: Inspection aborted (Sewer crossing)
- Site 5 – 200 mm Assiniboine Park Siphon: Inspection successful (Sewer crossing)
- Site 6b – 300 mm Munroe-Polson Siphon: Inspection successful (Sewer crossing)
- Site 13 – 600 mm North Kildonan Feeder Main: Inspection aborted (Water crossing)

The sewer crossing inspections are discussed in previous reports.

Inspection of the North Kildonan Feeder Main was planned to occur in December of 2012. Prior to inspection mobilization, it had been determined that the North Kildonan Feeder Main had been leaking a significant amount of water for an extended period of time. It was believed that a pipeline fracture had occurred under the river where significant length of the pipeline was suspended above the river bottom, and may have been subjected it to vertical and/or longitudinal bending. Based on Record information available, it is believed that this was an as constructed condition.

It was recommended that the inspection proceed to determine if the pipeline condition was suitable for repairs, and to help identify the leak location. During pigging attempts, a low density foam swab became lodged in the pipe. This raised concern over risk of attempting to deploy the ILI tool, as there was a higher probability of getting the gauge pig or ILI tool stuck at the pipe fracture. It was decided to abandon the inspection attempt. A partial video of the pipe was undertaken, utilizing a foam pig fitted with a portable underwater camera. This video indicated that in general the pipe and coatings appeared to be in reasonably good condition. However, a moderate number of tubercules were encountered.

5.3 2015 Program

One mobilization was undertaken in the Fall of 2015 to inspect all of the remaining water crossings in the current program.

The following sites were inspected:

- Site 10 – 400 mm Goulet-Doucet Water Main
- Site 12 – 250 mm Maryland Bridge Water Main
- Site 14 – 450 mm St. James Street Water Main

AECOM tendered a contract in the spring of 2015 (Bid Opp. 289-2015) for pipeline modification, cleaning, and support services for PICA's inspection work. The contract was awarded to Tri-Core, who completed the required cleaning with the assistance of Uni-Jet Industrial Pipe Services (their subcontractor). Cleaning was undertaken using a combination of new/sanitized sewer flushing equipment and foam pig swabbing.

Pipeline access was developed through the installation of launch assemblies on either side of the river. A typical launch assembly is depicted in Figure 11. The launch/ retrieval assemblies included features such as blind flanges or new isolation valves to provide system isolation and to facilitate flushing and cleaning. The following system modifications were undertaken to complete the inspections:

- Site 10:
 - Installation of launch wyes on Goulet St. and Doucet St.
 - The launch wye on the Goulet St. side was installed with a blind flange to permit isolation of the crossing and assist with flushing operations. The blind flange was removed after completion of the work
- Site 12:
 - Launch wyes installed off Cornish Ave (north side) and between the Maryland Bridges (south side)
 - Both launch wyes were installed with isolation valves.
 - The isolation valve off Cornish Ave was required to isolate the Misericordia Hospital water service from the river crossing.
- Site 14:
 - Launch wyes installed on St. James St. and Wellington Cres.
 - A 450 mm isolation valve was installed with the launch wye on St. James St. to provide additional isolation of the crossing from nearby water services. The existing (original) 450 mm gate valve on Wellington Crescent was found to be operational and to generally provide a water tight seal.
 - The existing cast iron piping which was believed to be Victaulic jointed was found to be a traditional lead joint pipe. Outside diameter variations in the 1930's vintage cast iron pipe resulted in difficulties in procuring proper couplings for the modification work.
 - Further there was no thrust block at the vertical bends on the south bank of the river. Modifications to the original launch wye configuration were required to complete the inspection. Additional piping was removed and the launch wye was installed to act as a 45 degree vertical bend, see Figure 11



Figure 11: Launch Assembly – St. James Water Main (South Side)

The modifications were specified complete with construction drawings which were updated to reflect changes made during construction. The following City of Winnipeg drawings are applicable to the inspection modifications:

- D-13302
- D-13303
- D-13409

Issues encountered during the program included:

- A foam pig became lodged in the pipe and broke the tow rope while cleaning the Goulet-Doucet Water Main. The pig was stuck on the east bank of the Seine River. An external excavation was required to retrieve the pig. The pig was lost due to a defect in the tow rope embedded within the foam pig and an accumulation of disbonded coating debris from the steel crossing increasing drag on the assembly. Subsequent to the incident, Tri-Core installed steel cables through the pigs for towing, see Figure 2.
- Several large pieces of lead were removed from the St. James Water Main crossing. It is believed that the lead pieces are remnants of the original construction which utilized leaded joints.
- PICA encountered issues with their inspection data during inspection of the St. James Water Main crossing. The issue was caused by a faulty module on their new inspection tool. A replacement part was air shipped to Winnipeg to complete the inspection.

5.4 Future Program

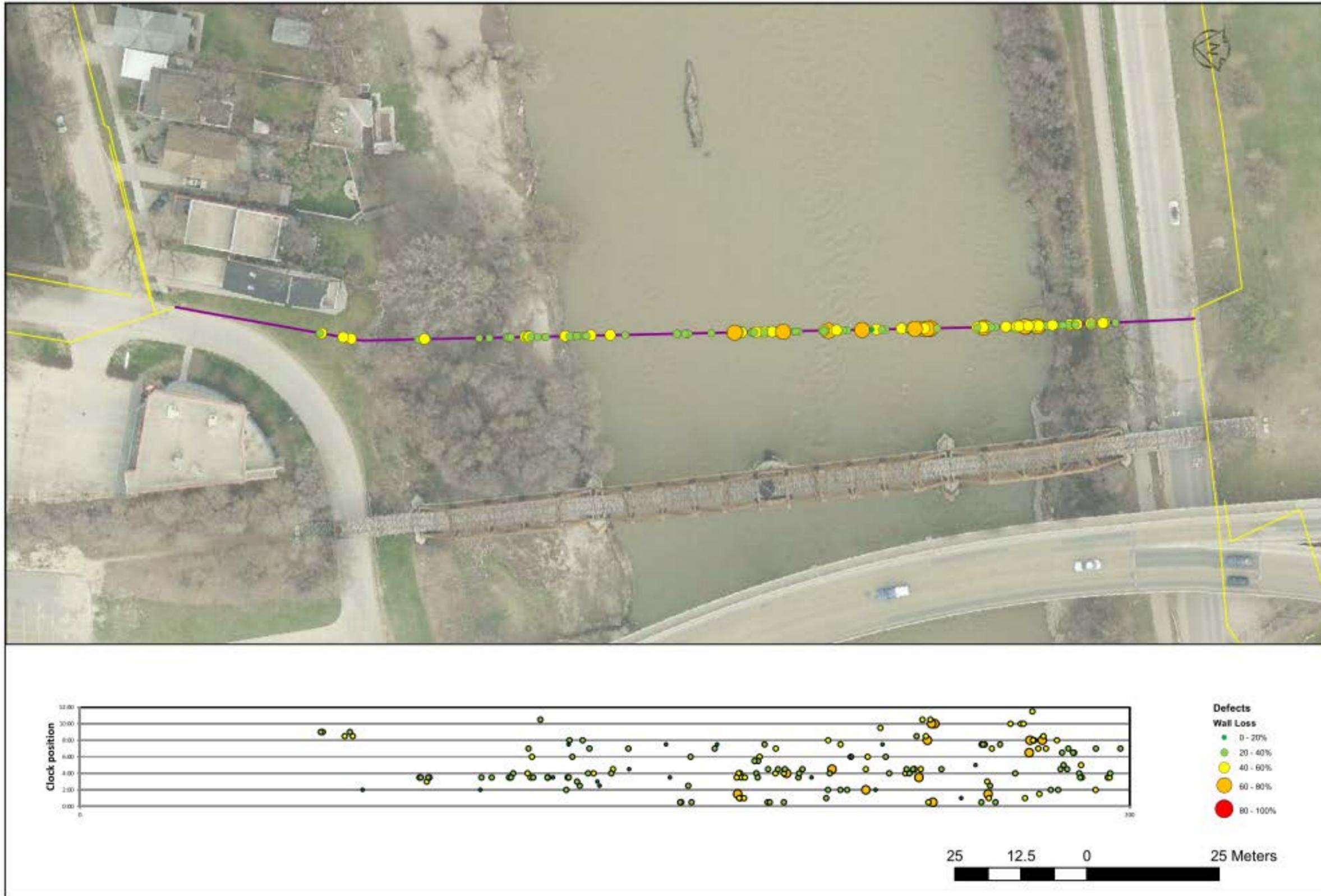
Two water main crossings were removed from the program due to operational conflicts in the City's Regional water system. These are the Charleswood-Assiniboia Feeder Main (Site 9) and the Kildonan-Redwood Feeder Main (Site 11). Inspection planning and logistics were well advanced for both sites and would be logical inclusions in future inspection programs.

5.5 Raw Inspection Data

PICA's post processing of their inspection data has been reported to AECOM via site specific Condition Assessment Reports which are attached in Appendix D. The overall spatial distribution of defects is highly variably, ranging from through wall holes, near through wall holes to pipe in very good condition. Some of the more notable defects/findings include:

- Site 10 – 400 mm Goulet-Doucet Water Main: Inspection did not detect any corrosion related defects on this crossing
- Site 12 – 250 mm Maryland Bridge Water Main: Inspection data indicates wide spread corrosion related defects of which some having reported defects with as little as 3% remaining wall thickness.

A detailed overview of the severity and spatial distribution for the St. James Water Main has been presented in Figure 12. Spatial mapping of defects for sites 10 and 12 have not been produced due to the wide spread deterioration of the Maryland Bridge Water Main and the good condition (zero defects) of the Goulet-Doucet Water Main. A detailed analysis of the reported corrosion defects has been included in Section 6.1.



Site 14 - St. James Watermain

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Figure 12: Site 14 – St. James Water Main

6. Condition Assessment

Crossings inspected as part of this program have been evaluated to determine their current structural capacity, risk of floatation as it relates to operation and future maintenance, and remaining service life. Structural checks have assumed mechanical properties consistent with the original specifications and diminished wall thicknesses and deteriorated sectional properties consistent with the results of the field program.

Table 3 lists all of the water crossings included in the original program. The North Kildonan Feeder Main (Site 13) has since been replaced and will therefore not be included in further discussions. The Charleswood-Assiniboia Feeder Main (Site 9) and the Kildonan-Redwood Feeder Main (Site 11) have been included for discussion purposes with corrosion rates based on the two steel force mains assessed as part of the 2014 inspection program, documented in the wastewater crossing report.

Table 3: Crossing Summary

Site	Crossing Location	Notes	Nominal Diameter (mm)	Material Type	Installation Year	Outside Diameter (mm)	Wall Thickness (mm)
9	Charleswood-Assiniboia Feeder Main		600	Steel	1965	609.60 (24")	9.53 (3/8")
10	Goulet-Doucet Water Main		400	Steel	1953	406.40 (16")	9.53 (3/8")
11	Kildonan-Redwood Feeder Main		600	Steel	1955	609.60 (24")	7.94 (5/16")
12	Maryland Bridge Water Main		250	Steel	1970	254.00 (10")	9.53 (3/8")
13	North Kildonan Feeder Main	Replaced in 2015/2016	600	Steel	1978		
14	St. James St. Water Main		450	Cast Iron	1931	505.97	24.38

6.1 Corrosion

The electromagnetic inspection data collected by PICA was submitted to AECOM in the form of an analysis report. The results from the inspection and subsequent analysis by PICA have been summarised in Table 4. Please note that the inspection results have not been verified through any external investigations at the time of this report.

Assumed corrosion rates and associated remaining service life have been included in Table 4 for both Site 9 and Site 11. Differences in pipe materials and varying inspection results from the 2015 inspections made inferring corrosion rates from the inspected water inventory impossible. Thus, the assumed corrosion pitting rates are based on the average pitting rates calculated for the two wastewater force mains inspected in 2014 (Site 3 and 4) both operate under pressurized conditions, have similar material/coating properties, and display fairly consistent corrosion rates.

Table 4: Corrosion Summary

Site	Crossing Location	Nominal Diameter (mm)	Inspection Year	Average Pipe Wall Thickness	Deepest Corrosion Defect (% Remaining Wall)	Equivalent Pitting Depth (mm)	Pitting Rate (mm/year)	Critical Pitting Depth (mm)	Remaining Service Life (Years from date of Inspection)
9	Charleswood-Assiniboia Feeder Main	600	N/A*	N/A	58%***	3.99***	0.0797**	9.53	69**
10	Goulet-Doucet Water Main	400	2015	100.0%	100%	0	0	9.53	N/A
11	Kildonan-Redwood Feeder Main	600	N/A*	N/A	41%***	4.70***	0.0797**	7.94	41**
12	Maryland Bridge Water Main	250	2015	98.4%	3%	9.24	0.2053	9.53	1.39
14	St. James St. Water Main	450	2015	93.7%	27%	17.80	0.2119	24.38	31

* Not inspected

** Pitting rate estimated as average of rates from wastewater force main sites

*** Calculated from assumed pitting rate

6.1.1 General Wall Loss

There are many factors which govern the corrosion of steel and cast iron pipelines. Corrosion is rarely a uniform process and typically can be found at coating flaws or damaged coating sites; galvanic effects from changes in pipe material or at welds; concentration cell corrosion from changes in bedding material and/or soil chemistry; or exposure to stray electrical currents. The form of corrosion in steel and cast iron pipe is also, typically, a very localized phenomenon classified as pitting corrosion which will ultimately compromise hydrostatic integrity of the pipe but not have a significant effect on the overall ring stiffness of the pipe and in some cases not the overall hoop strength of the pipe. Thus, corrosion in steel pipes often does not initiate large scale structural failures with massive fluid loss; however, pitting corrosion will cause a loss of hydrostatic integrity at full or near-full penetration. If the loss in hydrostatic integrity is not addressed it will lead to loss of embedment soils and the likely eventual collapse of the pipe due to loss of soil support. The critical pitting depths and the estimated remaining service life listed in Table 4 are typically based on full perforation of the pipe wall. Estimated service life is based on a linear increase in pit depth which is considered a conservative corrosion model.

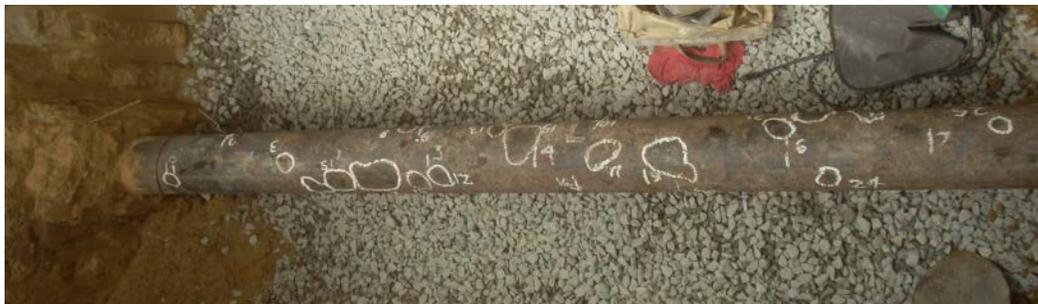


Figure 13: Example of Localized Pit Locations on a Steel Pipeline

While localized pitting is often not the governing structural failure mode for steel pipe, it must be accounted for in the structural analysis of existing infrastructure. The National Bureau of Standards compiled pitting data versus average wall loss for buried steel pipes. Their findings concluded that at the time of first perforation of the pipe wall, an average of 13% of the pipe wall has been lost due to corrosion³. Thus, where corrosion pitting is reported by PICA, the wall thickness used in our analysis has been reduced by 13% of the estimated pitting depth.

6.1.2 Remaining Pipeline Life

A critical pitting depth was determined and used to estimate the remaining service life of the pipe. The critical pitting depth was assumed to be perforation of the pipe wall resulting in leakage to the environment. The remaining service life of the pipeline was then calculated using Equation 1.

$$\text{Remaining Lifespan} = \frac{\text{Critical Pitting Depth} - \text{Current Pitting Depth}}{\text{Pitting Rate}}$$

Equation 1: Remaining Service life

6.2 Structural Assessments

6.2.1 Floatation

Floatation checks for each crossing were undertaken to determine the risk of floatation during operation and to accommodate future rehabilitation work, based on the profile for each crossing. Each crossing was conservatively assessed with 10% air entrainment over the cross sectional area. Table 5 lists the state that each siphon was assessed with and the Factor of Safety (FS) against floatation for each siphon. A generally accepted FS against floatation is recommended to be 2.0⁴, for pipes intended for permanent embedment. Many river crossing pipes actually have a much lower FS against floatation based on their installation techniques. A FS check, however, is prudent to understand any inherent vulnerability with respect to floatation.

The calculations took into account the following loads in overcoming floatation:

- Water weight
- Soil weight (prism), where applicable

³ *Handbook of Steel Drainage & Highway Construction Products, First Canadian Edition, 1984.*

⁴ *Moser, Folkman, Buried Pipe Design, 2008, McGraw-Hill.*

- Dead weight of pipe
- Concrete Anchor blocks

A majority of the siphons were buried or installed with cast-in-place concrete anchor blocks/concrete bedding. To account for fastener corrosion on pre-cast concrete anchor blocks, only half of the block weight was utilized in our floatation checks. Our analysis indicated that all of the water crossings had a factor of safety greater than 2.0 except for the Maryland Bridge Water Main (1.75). The Maryland Bridge Water Main was modeled with zero soil cover to reflect uncertainty in its construction, as record drawings do not show a soil profile overtop of the pipe beneath the river. While the factor of safety is below 2 for these two assets, the use of 10% air entrainment is quite conservative for a water distribution system. However, any planned rehabilitation work should closely review floatation prior to dewatering.

Table 5: Floatation Check Summary

Site	Crossing Location	Nominal Diameter (mm)	Buried	Anchor Blocks	Operational Condition	Factor of Safety
9	Charleswood-Assiniboia Feeder Main	600	Yes	Yes	Full, 10% Air Entrainment	3.57
10	Goulet-Doucet Water Main	400	Yes	No	Full, 10% Air Entrainment	11.05
11	Kildonan-Redwood Feeder Main	600	Yes	No	Full, 10% Air Entrainment	3.65
12	Maryland Bridge Water Main	250	Unknown	Concrete Blocks at each Bank, not used in calculation.	Full, 10% Air Entrainment	1.75
14	St. James St. Water Main	450	Yes	No	Full, 10% Air Entrainment	28.59

6.2.2 Loading

While river crossings are subject to the same combination of external and internal loading conditions as typical buried water mains, the degree to which these conditions change along their length makes their analysis unique. Factors contributing to the external loading regime of river crossings includes: soil loads, live loads, and external hydrostatic pressure. For illustrative purposes, Figure 14 shows the range of external loading conditions imparted on a typical river crossing site. This would include full Flood Protection Level (FPL) river conditions.

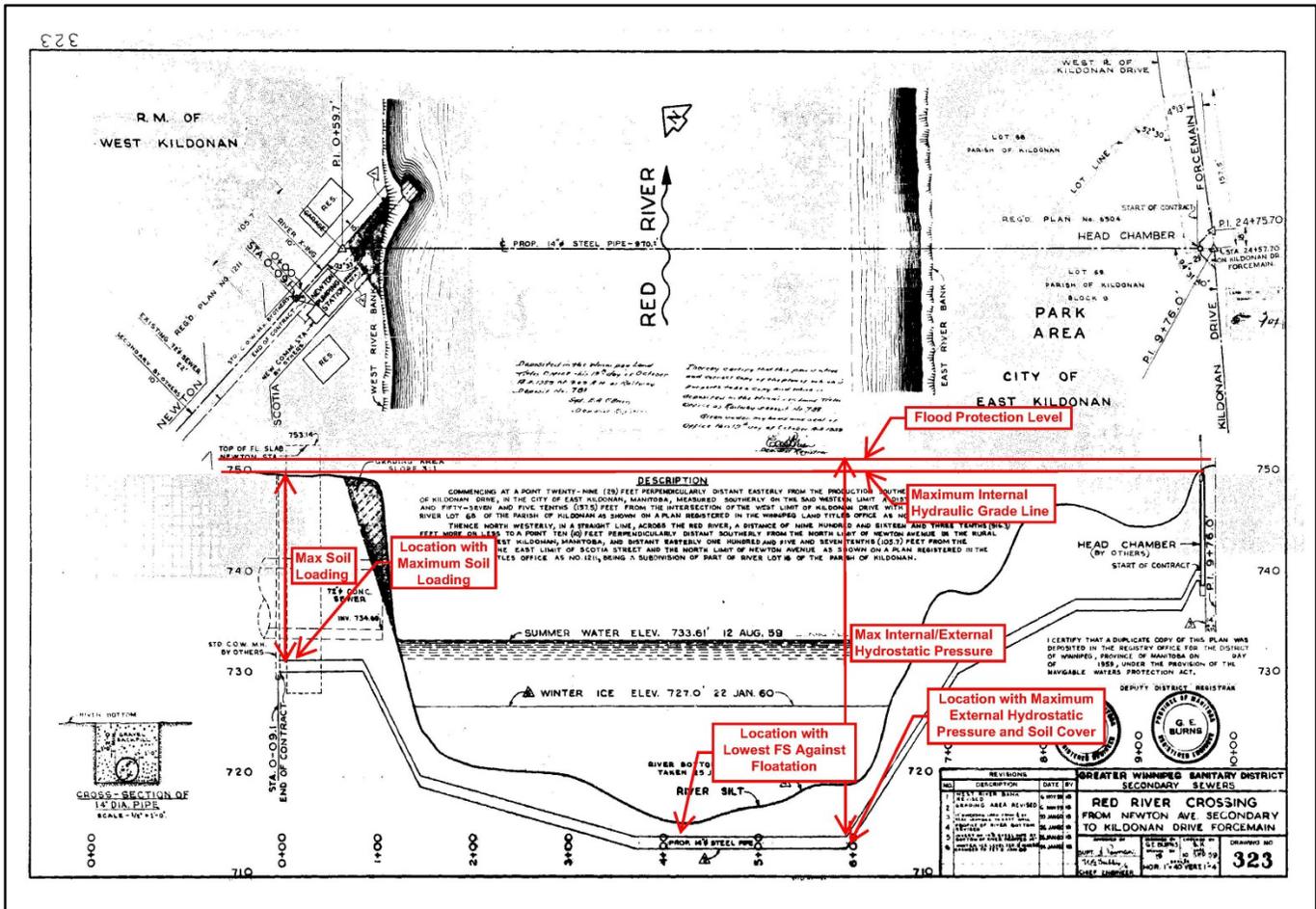


Figure 14: External and Internal Loading Conditions (Example)

The water crossings assessed are comprised of steel and cast iron pipe materials. Steel is considered to be a flexible piping as it can deflect significantly without inducing structural distress, and deflection limits are often set to prevent coating damage. Flexible pipes are assumed to be less stiff than the surrounding native and embedment soils and thus it can only see the weight of the soil located directly above the pipeline, commonly referred to as a prism loading. This is the worst long term loading case for flexible pipes. All flexible pipelines reviewed as part of this assessment have been assessed with a prism loading. Cast iron pipelines however are considered a rigid pipe and will see additional loads due to settlement of their embedment soils relative to the pipelines themselves. External loads acting on the rigid pipelines were estimated using Heger Positive Projection methods, which use a prism loading and a vertical arching factor to account for settlement loads.

Live loads have been applied to all maximum soil cover conditions where surface loads may reasonably be expected to exist. An AASHTO HS 20 design vehicle has been utilized for estimating live loads applied to the crossing pipes in accordance with AASHTO LRFD, 7th Ed.

6.2.2.1 Embedment Conditions

The existing river crossings were installed with a mixture of native, fine grained, and granular soils as embedment material. Given the typical installation procedures for river crossings (open cut or dredge and backfill) and soils present throughout the Winnipeg region a constrained soil modulus of 4.8 MPa has been assumed for all crossings. This is consistent with a soft fine grained native soil as defined by AWWA M11. While conservative for locations away from the river, this is likely consistent with conditions near or at the bottom of the river. For rigid pipe installations, a Type 3 SIDD soil pressure distribution was assumed.

6.2.2.2 Internal Pressure

Internal pressures impart a tension stress in the circumferential direction of pipelines called hoop stress and is calculated using Equation 2. If the hoop stress exceeds the yield stress the internal pressure may cause the pipe to burst. It is standard practice to evaluate the performance of a pipe under internal loading using a reduced allowable design stress. The design hoop stress may equal up to 50% and 75% of the yield strength of the wall material for normal operation and transient surge pressures respectively.

$$s = \frac{Pd}{2t}$$

Equation 2: Hoop Stress

Internal hydrostatic pressures for the water crossings were reviewed based on a worst case scenario with the intention of screening for governing failure modes. Water mains were modeled using a steady state operating pressure of 551.6 kPa (80 psi) and a transient overpressure of the greater of 275 kPa (40 psi) or 40% of the steady state pressure. This is consistent with operation of the City's regional water system. Average wall thicknesses were reduced as indicated in Section 6.1.1.

In addition to evaluating the hoop stress for reduced wall thicknesses, a Level 1 assessment was undertaken for all water crossings to assess the effects of pitting on the internal pressure capacity as per ASME B31G⁵. Using this methodology, a failure stress is estimated based on the axial length and maximum depth of pitting in a single corrosion feature. The estimated failure stress is converted to a pressure and compared to the internal operating and surge pressures experienced by the pipe.

In the reports submitted to AECOM by PICA, the depth of pitting and length of defect was provided. To account for potential future progression of defects, a review of maximum allowable defect was undertaken. The sensitivity of the estimated failure stress to corrosion length decreases with greater lengths and eventually approaches a plateau. To provide a conservative look at the impact of corrosion, estimated failure stresses reflect the value at the plateau.

6.2.2.3 Structural Checks – Steel Pipe

Structural checks on the steel river crossing pipelines were undertaken using both the ASCE Engineering Practice 119 and AWWA M11⁶. The steel crossing pipes were checked for the following failure modes:

⁵ ASME, *Manual for Determining the Remaining Strength of Corroded Pipelines, (B31G), 2012*

⁶ AWWA, *Steel Water Pipe – A Guide for Design and Installation (M11), Fourth Edition, 2013.*

- Deflection
- Wall Crushing
- Lateral Soil Pressure
- AWWA Buckling

Material properties from the original construction specifications were used where available. All specifications available required steel conforming to ASTM A283 – Low and Intermediate Tensile Strength Carbon Steel Plates of Structural Quality, Grade B or C. The following pipelines had material specified as such, whereas all others were inferred to have constructed from similar material.

- Site 9 - Charleswood-Assiniboine Feeder Main
- Site 10 – Goulet-Doucet Water Main
- Site 11 – Kildonan-Redwood Feeder Main
- Site 12 – Maryland Bridge Water Main

Deflection

Deflections have been estimated using both the Modified Iowa formula and soil strain methods outlined in ASCE 119. The Modified Iowa Formula (Equation 3) takes into account both soil and ring stiffness to estimate ring deflection. Soil strain methods, however, discount the ring stiffness and look only at the stiffness of the embedment soils and their response to the lateral loads imparted by the pipe ring as the Modified Iowa Formula over predicts ring deflection due to the methods used to calculate soil deflection in its original derivation.⁷

While considered to be more accurate for thin walled large diameter pipe with relatively low inherent ring stiffness, by ignoring the stiffness of the pipe, soil strain methods grossly over predict ring deflection for smaller diameter pipe with relatively thick walls. Where pipes exhibit higher than average ring stiffness it is recommended to use the Modified Iowa Formula. As a majority of the City's water crossings fit this description, the Modified Iowa Formula has been used to estimate deflection in a majority of the assets in this assessment.

$$\Delta x = D_1 \frac{K(W_s + W_L)r^3}{EI + 0.061E'r^3}$$

Equation 3: Modified Iowa Formula

Unlike traditional AWWA design criteria, critical deflection limits were determined using the following criteria:

- AWWA deflection limitations (5% for pipe with flexible lining or coating)
- Soil slippage causing buckling
- Outer fibre stress in the pipe wall

The small diameter crossings reviewed as part of this crossing were typically governed by AWWA deflection limits or outer fibre stress checks, partially due to the thicker than average pipe walls. Critical deflections based on outer fiber stress were determined using the tensile strength of the steel.

⁷ ASCE, *Buried Flexible Steel Pipe – Design and Structural Analysis*, 2009.

Wall Crushing

Wall crushing reviews the imparted stresses from external loading, internal vacuum, and bending to determine the maximum applied compressive and tensile stresses within the pipe wall.

Lateral Soil Pressure

Lateral soil pressure checks ensure that the lateral pressures applied to the embedment soils are within the capacity of the surrounding embedment soils.

AWWA Buckling

Buckling checks ensure that the applied pressures including soil, live loads, and internal vacuum don't result in reverse curvature and ultimately buckling of the pipe ring. We have reviewed allowable buckling pressures using both constrained buckling (Luscher) and unconstrained buckling (Timoshenko)⁸, where applicable. Equation 4 and Equation 5 were used to determine the allowable constrained and unconstrained buckling pressures respectively for the steel pipelines. Applied buckling pressures were determined by assessing live loads and internal vacuum events separately.

$$q_a = \frac{1}{FS} \sqrt{32R_w B' E' \frac{EI}{d^3}}$$

Equation 4: Constrained Buckling Pressure

$$q_a = \frac{1}{FS} \frac{2E}{1 - \nu^2} \left(\frac{t}{d}\right)^3$$

Equation 5: Unconstrained Buckling Pressure

6.2.2.4 Structural Checks – Cast Iron Pipe

Structural checks on the St. James Street cast iron water main (Site 14) were undertaken in accordance with AWWA H1-67⁹. Failure of cast iron pipelines can occur under various combinations of external loading and internal pressures. The breaking points defined by the interaction of the applied loadings can be defined reasonably well by a parabolic curve where the x and y intercepts represent the bursting pressure under no external load, and the crushing load with no internal pressure, respectively.

⁸ AWWA, *Steel Water Pipe – A Guide for Design and Installation (M11)*, Fourth Edition, 2013.

⁹ AWWA, *Thickness Design of Cast-Iron Pipe (H1)*, First Edition, 1967

$$w = W \sqrt{\frac{P - p}{P}}$$

Equation 6: Cast Iron External Load – Internal Pressure Failure Curve

AWWA H1-67 prescribes that cast iron pipes are to be evaluated for two loading conditions:

- Case 1 – Soil load, operating pressure and surge pressure
- Case 2 – Soil load, vehicle live load, and operating pressure

Each loading case was evaluated to ensure that both the external loading and internal pressure were within the region defined by Equation 6 using the reduced average wall thickness calculated as described in 6.1.1. Reduced capacity for internal pressure due to corrosion was also estimated using ASME B31G as described in 6.2.2.2. Further to estimating the axial length of the governing corrosion feature, the yield stress of cast iron was estimated to be 50% of the ultimate tensile strength for the purposes of analysis.

6.2.2.5 Loading Results

Table 6 lists the loading conditions checked for each crossing along with the governing failure mode and resulting FS. The FS's presented in Table 6 represent the FS above accepted design criteria for both steel and cast iron. The following FS's have been incorporated into the calculations:

- Deflection – Service based Criterion, FS of 1.0
- Wall Crushing
 - Steel Pipe: External Pressure & Internal Vacuum – 2
 - Steel Pipe: Wall Bending Stresses – Performance Criteria, no FS
- Lateral Soil Pressure: No additional FS
- AWWA Buckling:
 - Constrained buckling – 2
 - Unconstrained buckling – 1.3
- Internal pressure (steel)
 - Operating pressure only – 2
 - Operating pressure and surge pressure – 1.33
- Internal pressure (cast iron) – 2.5
- External loading (cast iron) – 2.5

Table 6: Loading Results

Site	Crossing Location	Pipe Material	Nominal Diameter (mm)	Reviewed Condition	Governing Failure Mode	Factor of Safety*	Notes
9	Charleswood-Assiniboia Feeder Main	Steel	600	Max Soil Cover	Wall Crushing	3.22	
9	Charleswood-Assiniboia Feeder Main	Steel	600	Max Hydraulic Head	ASME B31G Internal Pressure	3.73**	Corrosion length estimated
10	Goulet-Doucet Water Main	Steel	400	Max Soil Cover	Wall Crushing	4.74	
10	Goulet-Doucet Water Main	Steel	400	Max Hydraulic Head	Hoop Stress	8.10	
11	Kildonan-Redwood Feeder Main	Steel	600	Max Soil Cover	Deflection	2.00	
11	Kildonan-Redwood Feeder Main	Steel	600	Max Hydraulic Head	ASME B31G Internal Pressure	2.41**	Corrosion length estimated
12	Maryland Bridge Water Main	Steel	250	Max Soil Cover	ASME B31G Internal Pressure	2.45	Corrosion length estimated
12	Maryland Bridge Water Main	Steel	250	Max Hydraulic Head	ASME B31G Internal Pressure	2.40	Corrosion length estimated
14	St. James St. Water Main	Cast Iron	450	Max Soil Cover	External Loading	1.18	
14	St. James St. Water Main	Cast Iron	450	Max Hydraulic Head	ASME B31G Internal Pressure	2.52	Corrosion length and yield stress estimated

* Factor of safety above recommended design conditions

** Calculations incorporate the average corrosion rates measured and reported by PICA

7. Failure Risks and Rehabilitation

7.1 Review of Defects

Below is a discussion on the results of the inline inspections undertaken by PICA. A Special defect map for the St. James Water Main crossing is presented in Figure 12.

7.1.1 Site 10 – Goulet-Doucet Water Main

The 400 mm Goulet-Doucet Water Main was not found to have any corrosion related defects and is believed to be in good condition. There were however; some defects discovered through the course of our work, these include:

- Disbondment of the steel water main's internal coating. Pieces of coating, believed to be a coal tar, were found within the AC pipe at the west launch wye location and within the steel water main where the external excavation was undertaken to retrieve the lost foam pig. In fact, it is believed that the pig was stuck on a large deposit of disbonded lining material. Loss of the pipeline's interior coating raises some concerns over the potential for accelerated corrosion.
- A leak was discovered on the water main during the post inspection pressure tests. The leak was caused by a corroded steel plug installed on the cast iron transition from the steel crossing to the AC water main on the west bank of the Seine River. The corroded plug was replaced by the City's Water Services Division.

Pressure testing of the crossing after repair of the leak discussed above found a leak to be present somewhere between the west launch wye location and Archibald St. This is due to a bypassing mainline valve on Doucet St. immediately east of the crossing. The City's Water Services Division correlated the river crossing twice looking for leaks and concluded that no leaks were present in the vicinity of the river crossing. The water main was subsequently placed back into service.

7.1.2 Site 12 – Maryland Bridge Water Main

Over 441 corrosion related defects were found on the Maryland Bridge Water Main crossing, see Figure 15. Defects are located along the entire length and around the entire circumference of the pipe with remaining wall thicknesses as low as 3%.

An active leak was discovered on the water main prior to the post-inspection pressure test. The leak was correlated by the City's Water Services Division and repaired by Tri-Core under a separate contract. The leak was located approximately 25 m north of the south launch wye on the south bank of the Assiniboine River and used in verifying and calibrating the data collected by PICA during their inspection.

The pipe was successfully pressure tested after completion of the emergency repair.

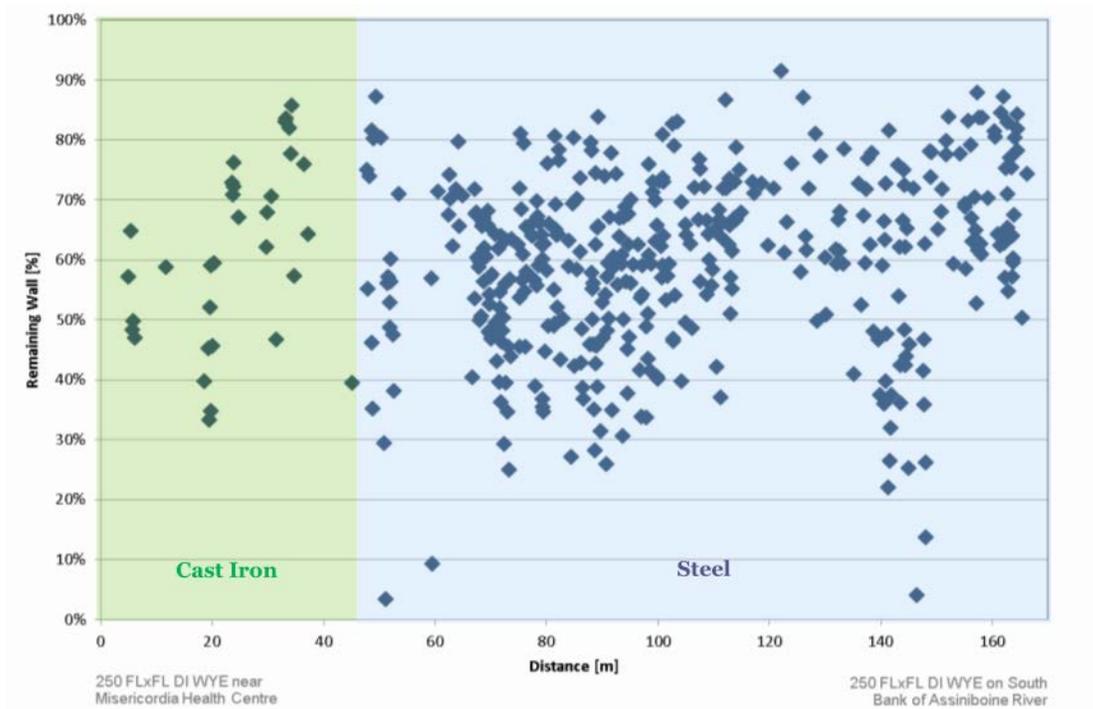


Figure 15: Maryland Bridge Water Main Corrosion Defects

7.1.3 Site 14 – St. James Water Main

165 corrosion related defects were identified on the St. James Water Main crossing, see Figure 16. Defects were generally located beneath the river and range from 91% to 27% remaining wall thickness.

The pipe was successfully pressure tested following the inspection.

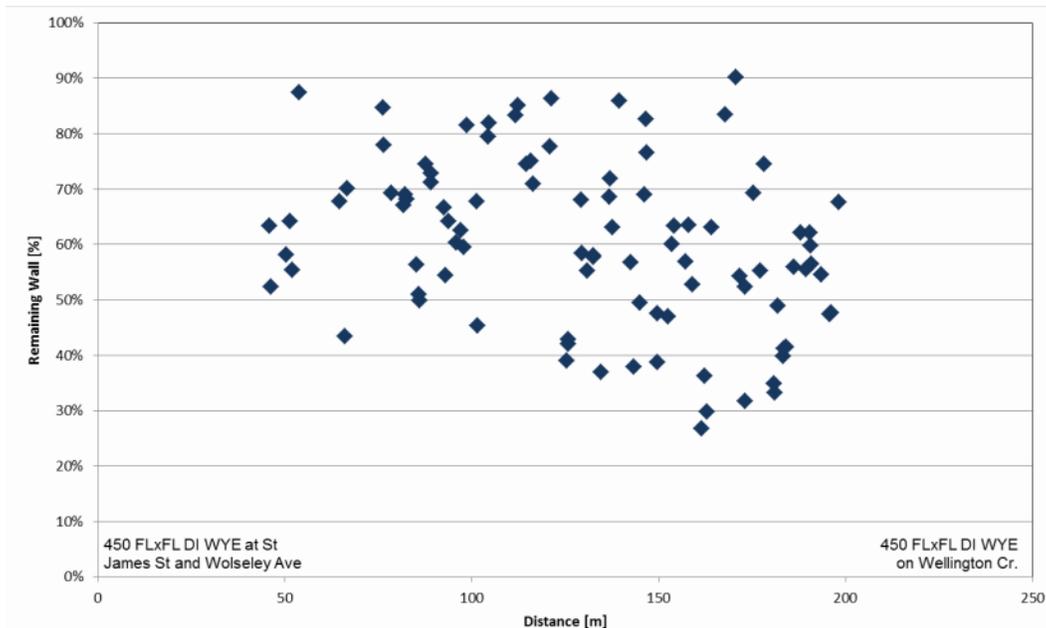


Figure 16: St. James Water Main Corrosion Defects

7.2 Risk Based Rating System

The intent of this program is to assess the failure risks associated with the water crossings included in the current program. As part of the Water Main Criticality Study completed by AECOM (UMA Engineering) each water main river crossing within the City of Winnipeg was rated for both its probability and consequence of failure. This rating was used to some extent to drive the development of the current inspection program.

The water main criticality study considered the following in determining the consequences of failure:

- System redundancy
- Pipe diameter
- Pipe material
- Crossing type
- Stream classification

At the time of the 2006 risk assessment report, no direct condition assessment had been undertaken on the river crossings, thus the probability of failure was based on record information and the characteristics of pipeline materials and expected pipe embedment environment, including:

- Pipe material
- Pipe diameter
- Crossing type
- River bank characteristics
- Expected soil conditions
- Failure history
- Availability and effectiveness of assessment technology

By plotting of the probability rating against the consequence rating for each crossing they can be placed into a standard risk management matrix (Figure 17 illustrates these sites based on the water main criticality study). The crossings inspected as part of this program were spread between the low and high consequence zones generally based on pipeline diameter/importance to overall system performance with probability of failure in the medium risk zone. Without completing condition assessment on the crossings the large diameter pipelines would have been on the list for replacement in the near to medium term.

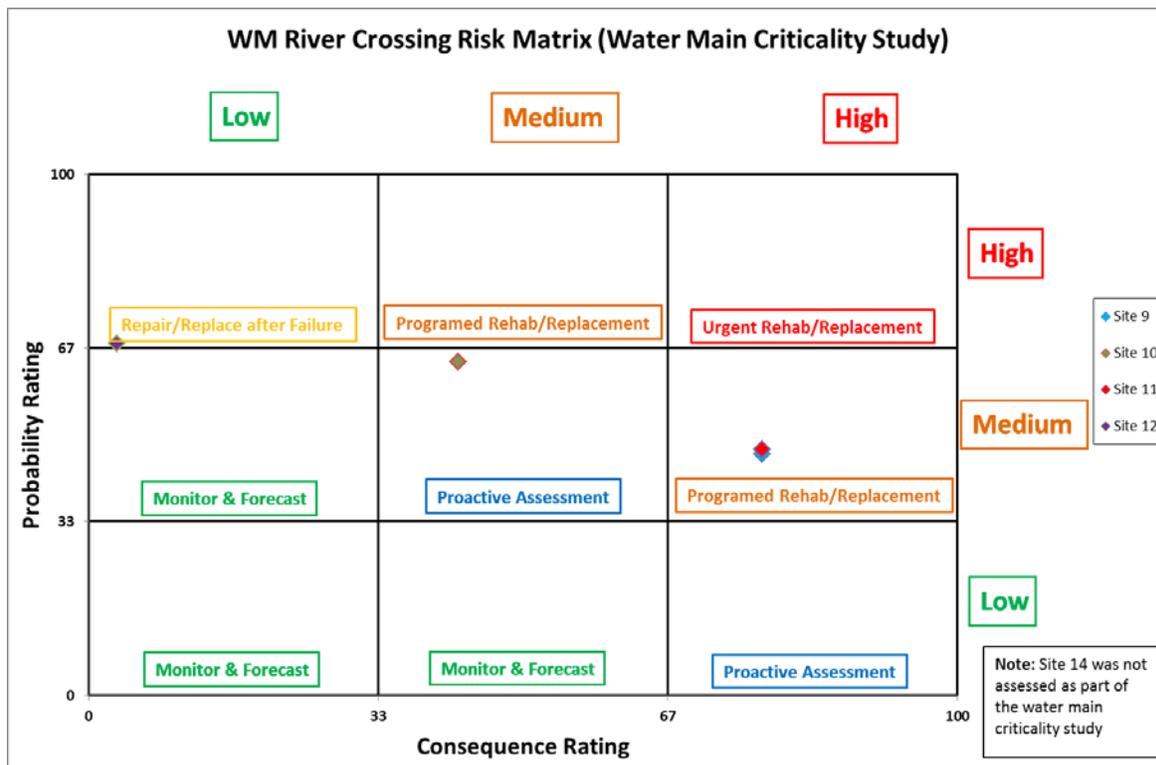


Figure 17: Risk Matrix – Water Main Criticality Study

AECOM has updated the risk profile for each crossing included in this program using condition assessment and geotechnical data collected through the current program. The updated risk matrix utilizes the existing consequence model and an updated probability model using the estimated remaining lifespan and the geotechnical assessment completed as part of this program. The probability of failure was considered to be the worst case between the remaining lifespan and geotechnical assessment as either could cause the pipeline to fail.

The remaining service life for each asset has been estimated using the corrosion related defects reported by PICA as outlined in Section 6.1. As outlined in Table 4, the remaining service life for the assessed pipelines ranges from less than 2 years (Site 12) to an indeterminate year (Site 10). The estimate of remaining service life was converted to a probability of failure with a 100% probability of failure being associated with 0 remaining years and a 0% probability of failure for assets with estimated service life of 30 years or more. This rating puts any pipeline with a projected service life 10 years or less in the upper third of the risk matrix. This is a reasonable balance of the certainty of the technology measuring condition and the resultant action that should be undertaken. A minimum failure probability of 15% was applied to all sites to reflect level of certainty of the inspection technology and criticality of river crossing infrastructure in general.

As described in Section 4, the geotechnical program was split into two components: field investigation and slope stability modeling. Each site was rated on a 1 to 3 scale in the visual assessment of the river bank condition based on signs of erosion, and general risk of a slope failure. These were converted to probability of failures as noted in Table 7 below. Where a slope stability analysis was warranted (typically where a visual rating of 3 was assigned), the resulting factor of safety was rated in accordance with the rating scale presented in Table 8.

Table 7: Probability of Failure - Visual Assessment

Visual Assessment Rating	Probability
1	15
2	50
3	75

Table 8: Probability of Failure - Slope Stability Analysis

Slope Stability Analysis - Factor of Safety	Probability
Greater than 1.5	15
Between 1.4 and 1.5	30
Between 1.3 and 1.4	50
Between 1.0 and 1.2	70
Equal to or less than 1.0	100

In generating updated risk ratings the greater of the pipe condition and geotechnical ratings were used to calculate risk. The updated risk matrix (see Figure 18) depicts the risk rating for each crossing based on an actual determination of both the condition of the pipe and the river bank stability of each crossing location. Note, river bank stability has been assessed in the context of the risk it poses to the pipe.

While Site 9 and Site 11 were not inspected as part of this program, they have been included in the risk matrix to illustrate their current risk rating and the need to undertake inspection of these assets. The probability rating for site 11 was increased from that determined in the water main criticality study due to geotechnical concerns at this site.

Site 14 was not assessed as part of the water main criticality study. Thus a consequence rating was estimated based on the crossings redundancy in the regional water system and diameter.

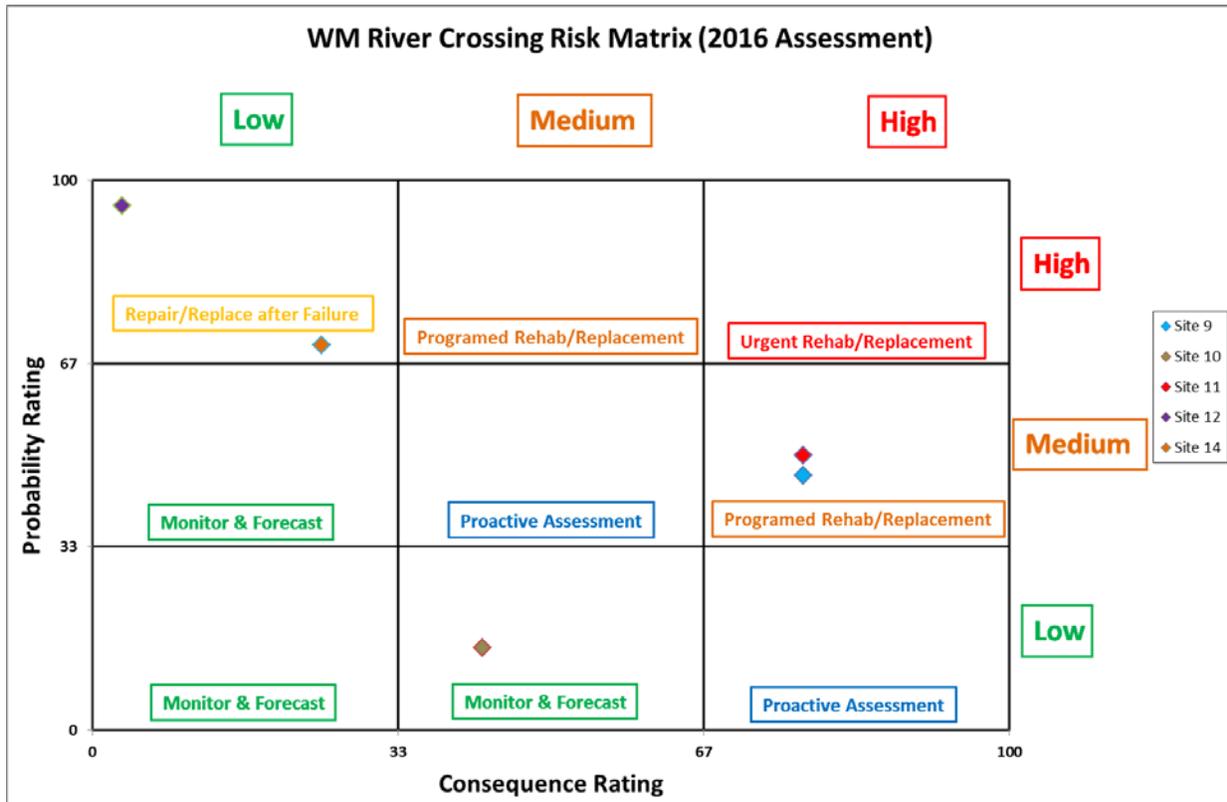


Figure 18: Risk Matrix – 2016 Assessment

7.3 River Bank Stabilization

The risk drivers for several of the pipelines are driven by geotechnical considerations as opposed to the physical condition of the pipe. This includes both the Redwood-Kildonan Main (Site 11) and the St. James Water Main (Site 14). The drivers on these sites are a low factor of safety against failure engaging the pipe.

Beyond the current slope stability issues identified in Section 4, a majority of the sites require some form of river bank stabilization. This ranges from minor regrading and armoring (placement of rip rap) to major regrading (possibly beyond the extents of the City’s right of way), armoring and revegetation. It would be prudent to address the issues identified in Section 4, including toe erosion and scarps in the short term. If left unchecked, erosion will continue to weaken the bank and will result in significantly costlier repairs.

Figure 19 was prepared to illustrate the risk profile of the water crossings with all of the geotechnical concerns addressed and mitigated.

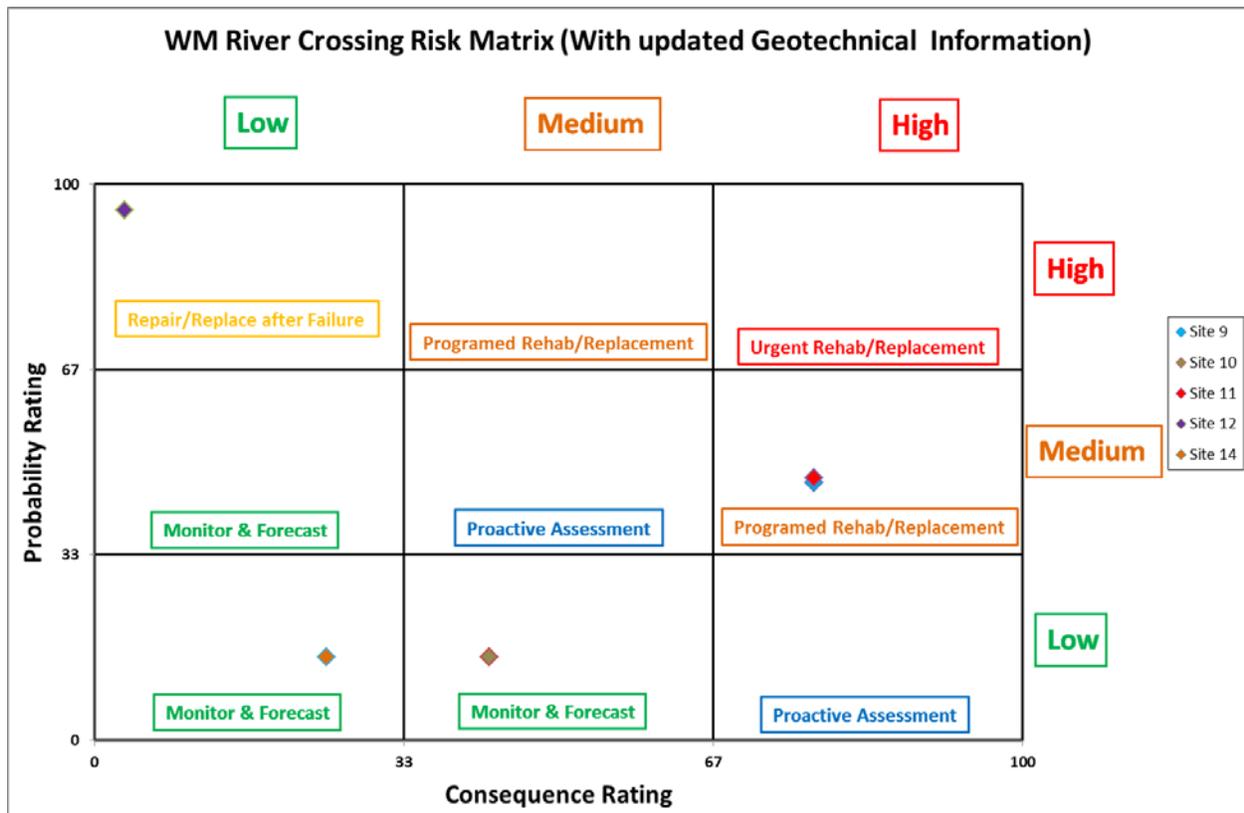


Figure 19: Risk Matrix – With Resolved Geotechnical Issues

7.4 Rehabilitation Methods

As noted in the preceding Section, the overall condition and risk profile of the assets reviewed is driven by not only the physical pipe condition; but by other potential failure modes, such as slope stability or failure due to buoyancy. Thus the rehabilitation approach to extend the asset life not only needs to consider pipe rehabilitation or replacement, but may include other treatments including slope stability improvements, bank armoring for toe erosion control, or improving factor of safety against buoyancy.

7.4.1 Pipeline Rehabilitation Methods

Replacement of river crossing assets carries a very high cost. Changes in the regulatory environment have radically changed the technical feasibility of reconstructing many of the assets in the manner that they were originally constructed. Department of Fisheries and Oceans (DFO) requirements now put a very strong emphasis on using construction techniques such as horizontal earth boring (HEB) or horizontal directional drilling (HDD) that have lower risk impact on fish habitat. These factors have elevated the replacement cost of river crossing a great deal. To provide a base reference for asset replacement, each site was reviewed to develop conceptual level costs for asset replacement based on the most feasible method that meets DFO existing work practices with habitat destruction. Table 9 includes the 2012 replacement cost budget (from the RFP) and an updated replacement cost estimate. The updated replacement costs are based on recent directional drilling contracts tendered in the City of Winnipeg. The crossing lengths were increased by a factor of 1.5 to 2.0 to reflect the additional lengths required to install pips by directional drilling vs. traditional open cut methods.

Based on the spatial distribution of defects found on the water main crossings, full rehabilitation of the affected sites is likely required.

Trenchless potable water pipeline rehabilitation technology has not developed to level of widespread use and acceptance as that currently seen in the gravity market. A majority of the technologies are non-structural and/or require dewatering for deployment. These include spray on and Cured-In-Place Pipe (CIPP) options. Research into technologies compatible with siphon crossings where dewatering is difficult or impossible has yielded limited options. One technology that in theory can be deployed without dewatering is flexible Kevlar reinforced tubes (hoses). These tubes are dragged into the water main and anchored on either end, creating a hydrostatically integral conduit. They however, have no external load carrying capacity and rely on the host pipe and internal pressure to resist overburden loads and external hydrostatic pressure.

Spray on lining methods may be advantageous for the Goulet-Doucet Water Main which is currently devoid of corrosion related defects but is known to have lost a majority of its inner lining. Given its soil cover, the pipeline can be dewatered, permitting the application of a spray on lining which would restore internal corrosion and resist future internal corrosion.

Table 9 details each crossing, diameter, length and the recommended rehabilitation type.

Estimating water main rehabilitation costs are extremely difficult given the number of variables present. For planning purposes rehabilitation costs of 2 to 3 times that of traditional gravity CIPP costs can be assumed. For the purposes of developing the work program below a spray on lining cost of \$3/mm/m was assumed for the Goulet-Doucet Water Main (Site 12). Replacement costs for the Maryland Bridge Water Main (Site 10) have been included in the 5 year capital program, however, it is expected that rehabilitation costs will be considerably lower. All costs values below are presented in 2016 dollars.

Budget rehabilitation costs have been included in Table 9. A detailed breakdown can be found in Appendix E. These costs include capital costs, and a 25% allowance for engineering and contingencies and would be considered at a Class 5 estimating level in accordance with AACE International Recommended Practice No 18R-97 for Cost Estimate Classification. The rehabilitation costs listed for the Maryland Bridge Water Main are the replacement costs as a trenchless rehabilitation method has not been identified nor a cost estimate completed. Recommended timing has been provided based on the broad criteria of:

- Short term – complete within the next 5 year capital program
- Medium term – complete in the next 5 to 10 years
- Long term – beyond the 10 year horizon for remedial works.

Table 9: Rehabilitation Methods, Costs and Timing

Site	Crossing Location	Nominal Diameter (mm)	Length (m)	Recommended Work Program	2012 Budget Replacement Cost	Estimated Replacement Cost	5 Year Capital	10 Year Capital	>10 Year Capital
Site 9	Assiniboia Feeder Main	600	240	Inspection and Toe Armoring	\$1,850,000.00	\$6,000,000.00	\$430,000.00		
10	Goulet-Doucet Water Main	400	85	Lining and Toe Armoring	\$600,000.00	\$1,500,000.00	\$162,500.00		\$210,000.00*
Site 11	Redwood-Kildonan Feeder Main	600	230	Inspection and Slope Stabilization/Toe Armoring	\$2,500,000.00	\$5,800,000.00	\$462,500.00		
12	Maryland Bridge Water Main	250	133	Toe Armoring and Rehab or Replacement	\$675,000.00	\$1,200,000.00	\$1,325,000.00**		
14	St. James Water Main	450	195	Reinspection and Slope Stabilization	\$1,387,500.00	\$3,600,000.00	\$1,087,500.00		\$250,000.00*
Totals					\$7,012,500.00	\$18,100,000.00	\$3,467,500.00	\$0.00	\$460,000.00

* Reinspection in 20 years

** Includes replacement cost. Rehabilitation options are expected to be considerably less expensive.

7.4.2 River Bank Stability Improvements

As discussed in Section 4 a majority of the sites display evidence of erosion, bank slippage, and in some cases, signs of more significant slope instabilities. As part of the HRRC program, preliminary river bank stability analysis has been undertaken on crossing sites with suspected bank instability. Preliminary stability modeling indicates factors of safety related to failure planes intercepting pipelines to be between 1.1 and 1.3. The recommended factor of safety for slopes impacting critical infrastructure is 1.5. It is noted that stability analysis to date is based on numerous assumptions of soil properties, and no in-field investigations have been completed to date. As a first stage, where stability analysis indicates inadequate factors of safety, a more thorough analysis is warranted, including detailed soil borings, confirmation of site geometry and, in some cases, slope monitoring may be warranted.

The failure risk driver on some of these pipelines is governed by the geotechnical considerations as opposed the physical condition of the pipeline. The St. James Water Main is a prime example of this, with active slope failure present at the site.

Where the pipelines are in good condition, river bank stability improvements are recommended to reduce the overall risk profile of these pipelines. Methods range from toe erosion protection measures to slope stability measures such as rock caissons or shear keys. It is noted, that as many of the pipeline crossings are located on narrow municipal corridors, that achieving appropriate safety factors may result in works outside the city owned lands.

From a budgeting perspective, costs range from approximately \$20,000 for minor armoring, to \$100,000 for regrading and toe stabilization, to upwards of \$1,000,000 for full stability upgrades such as rock caissons, all in 2016 dollars.

8. Conclusions and Recommendations

8.1 Site by Site Assessment

8.1.1 Site 10 – Goulet-Doucet Water Main

Inspection of the Goulet-Doucet Water Main did not identify any corrosion related defects. However, damage to the existing interior lining of the pipeline was discovered and it is believed to be mostly if not completely disbonded from the pipe. Thus, corrosion could become accelerated without protection from said coating. It is recommended that the pipeline be lined using trenchless methods in the near term. A 20 year reinspection frequency is recommended as outlined below.

The geotechnical site inspections identified minor toe erosion Goulet St. side of the crossing. It is recommended that erosion protection be installed as part of a near term work program.

8.1.2 Site 12 – Maryland Bridge Water Main

The Maryland Bridge Water Main is exhibiting signs of significant deterioration and has an estimated remaining useful service life of less than 2 years. Immediate rehabilitation or replacement is recommended. Regular leak detection of the crossing is recommended given the leak discovered during the inspection program and the overall poor condition of the pipeline.

The banks on both sides of the river are exhibiting signs of toe erosion and lower river bank instability. Erosion protection and regrading is recommended.

8.1.3 Site 14 – St. James Water Main

When cast iron pipelines corrode they form a graphite layer known as graphitization. Graphitization in thick walled cast iron pipe can effectively halt corrosion, and extend the remaining life of cast iron pipes. This has been seen in thick walled cast iron pipes exhumed in Winnipeg and throughout North America. Figure 20 is sample removed from the City's high pressure firefighting system in downtown Winnipeg which exhibited such corrosion behaviour.

The 2015 inspection is a snapshot of the St. James Water Main after almost 85 years of service. Assuming a linear corrosion rate and its original wall thickness we've estimated it to have a remaining useful service life of 31 years. However, if graphitization has occurred the corrosion rate for the water main may be considerably lower. Thus, it is recommended that the water main be reinspected in approximately 5 years. This will provide a baseline corrosion rate and may result in an extended prediction of service life.

It is recommended that the geotechnical concerns on the Wellington Crescent side of the crossing be investigated further and remediation steps taken as required to bring the factor of safety against failure to 1.5 or greater.



Figure 20: Cast Iron Pipe Sample Exhibiting Graphitization

8.2 Summary of Recommended Works

A summary of all recommended works is currently presented in Table 9. As noted, rehabilitation timing has been grouped into the following time horizons:

- Short term – should be carried out within the next 5 year capital (\$3,467,500.00)
- Medium term – should be carried out with the next 5-10 year horizon (\$0.00)
- Long term – implementation horizon is beyond the next 10 years (\$460,000.00)

Many of the assets noted in Table 9 are broken into multi horizons based on the value associated with carrying out geotechnical upgrading in the short term to reduce risk.

For the long term horizon, further clarity has been provided on the most probable time frame based on these inspections. For all assets with replacement or rehabilitation horizons beyond the 10 year horizon, a re-inspection frequency should be adopted. Current policy for re-inspection of sewer assets is documented in the City's Sewer Management Study¹⁰. All river crossings, by definition are Category A crossings, which carry a 10 year re-inspection frequency. The recommendations in the SMS Study, however, are based on the use of CCTV technology and visual classification methods alone. As the assessment methods utilized for the river crossings is much more definitive, we would recommend a re-inspection frequency commensurate with the lesser of the projected design life or 20 years.

¹⁰ UMA Engineering, "TECHNICAL MEMORANDA FOR SEWER CONDITION ASSESSMENT, SEWER REHABILITATION DESIGN, AND SEWER MAINTENANCE MANAGEMENT FOR THE CITY OF WINNIPEG", July 2001

Appendix A

**Risk Assessment Technical
Memorandum**

Technical Memorandum

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To	File	Page 1
CC	Cynthia Wiebe, City of Winnipeg; Armand Delaurier, City of Winnipeg; Chris Macey, AECOM	
Subject	High Risk River Crossings – Fall 2013 Inspection Program Risk Management	
From	Adam Braun/Marv McDonald	
Date	October 25, 2013	Project Number 60270487 (500)

1. Introduction

This memo is intended to address risk management for the upcoming 2013/2014 river crossing inspection program. The river crossings proposed for inspection are all high risk components of the City's regional water and sewer infrastructure as dictated by their age, function within the system, and consequences associated with failure. A large portion of the potential risk associated with these crossings is related to their condition, which at this time is largely unknown. Inspection of these pipelines to determine their current level of deterioration and assessment of their remaining useful life will clarify the actual risk associated with each crossing. This was demonstrated by the two crossings inspected in 2012, which were revealed to be in very good condition and do not require replacement in the foreseeable future to provide reliable service. One of the crossings may require an isolated point repair at an acceptable location, but the net savings from the two inspections is on the order of \$2-3 million.

The work undertaken to clean and inspection river crossings does pose numerous logistical challenges and some risks typically associated with construction and inspection programs, however, these should be weighed against the risks associated with not inspecting and clarifying the true condition of these critical assets. With the correct planning and execution, risks associated with flow bypassing, cleaning, and inspection can be mitigated and reduced to acceptable levels. Our proposed program for 2013/2014 involves cleaning and inspection of the remaining fourteen pipelines consisting of nine sewer and five water crossings. Risk assessments have been completed for the following pipelines:

- Site 1a and 1b - St James Interceptor Siphon (600mm and 500 mm)
- Site 3a - Newton Forcemain (350mm)
- Site 4a and 4b - St James Forcemain (450 mm)
- Site 6a - Munro/Polson Siphon (450mm)
- Site 7 - Main Street Interceptor Extension at Omand's Creek (600 mm)
- Site 9 - Assiniboine Feedermain (600 mm)
- Site 10 - Goulet-Doucet Watermain (400 mm)
- Site 11 - Kildonan-Redwood Feedermain (600 mm)

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- Site 12 - Maryland Watermain (250mm)
- Site 14 - St James Watermain (450mm)

Initial risk assessments for Site 2 the Northeast Interceptor siphon, and Site 8 the St Vital Bridge Forcemain Crossing have commenced but still need to be fully developed.

Site 13, North Kildonan Feedermain is being replaced and thus dropped from the program.

The scope of work for this program includes the identification of risks for gaining access to the pipelines for cleaning/inspection, and mitigation of the identified risks. Pipeline access for insertion of cleaning and inspection tools includes the installation of permanent inspection wyes, modification of existing valve chamber piping, and construction of flow diversions within existing junction chambers. Cleaning of pipelines will be completed with a combination of convention sewer flushing and pigging. Inspection of the identified pipelines will be undertaken using remote field electromagnetic tools winched through the pipelines.

2. General Cleaning and Inspection Risk Factors

There are numerous factors to be considered in cleaning and inspection programs to mitigate the possibility of pipeline damage or blockage as a result of the cleaning and inspection activities. Identification of the potential risk factors and development of mitigation and preventative strategies is key to ensuring a successful program. Risk factors vary with the pipeline crossing service condition (e.g. water or sewer, pressurized or gravity flow), and other factors such as the installation methods and details of the pipeline crossings and system redundancy.

A summary of general risks and associated mitigation factors are discussed below. Detailed risks on a site by site basis are discussed later in this document.

2.1 Reducing Capacity or Loss of Service during Modifications, Cleaning and Inspection

Risk Profile – Low-medium

Mitigation Strategies

For the sewer siphon sites to be inspected, required modifications and/or temporary works are minor, and can be removed in the event of flooding concerns. Risks will be mitigated by hydraulic modeling and planning, working in off-peak hours if required, and scheduling work where risks of major WWF events are minimal. Modeling results are discussed in more detail under site specific sections and in associated Hydraulic TM's.

For sewer forcemain sites, modifications to gain access to the pipelines are generally more extensive, and in some cases, include periods of time where systems are completely taken out of service via invasive construction techniques. Where possible, these modifications will be completed while maintaining level of service via pipeline redundancy. Where system disruptions are required, risks will be mitigated by a well-planned and conservatively scheduled work plan, such that the system can be

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returned to service in an appropriate timeframe. Where major disruptions are planned, detailed system monitoring will be employed to alert the project team of pending system concerns. Hydraulic modeling has been undertaken for all sewer siphons and forcemain crossings in order to determine shut down windows and pipeline capacities. Hydraulic technical memos have been prepared for each sewer crossing site addressing expected flow conditions, anticipated shut down windows, requirements for installing flow by-passes. Based on our analysis all crossings can be successfully cleaned and inspected without adversely affecting operation of the wastewater collection system. These are discussed further, later in this document

Most sewer pipelines will need to be completed during periods of dry water flow, to minimize possibility of CSO events. Weather conditions will be monitored closely, and inspection windows seasonally selected to minimize possibility of occurrence of wet weather events.

We do not anticipate the proposed watermain crossing shutdowns to pose any significant risk or loss of service issues to the regional distribution system, provided shutdowns are coordinated in low demand conditions. AECOM does not have sufficient information on the water system hydraulics to analyse system wide effects, however, we have a good practical understanding of the systems operational requirements. This includes the compounded effects that staging multiple crossing shutdowns will have on the system and each other. Shutdown requirements have been passed on to the City for modelling.

2.2 Potential to Aggravate Existing Conditions in Deteriorated Pipelines during Cleaning and Inspection Processes

Risk Profile – Low

Mitigation Strategies

The general purpose of the inspection program is to determine the existence and/or extent of deteriorated conditions in the pipeline. If these conditions are already present (i.e. incipient failure) then discovering these defects in a controlled and monitored manner will in itself alleviate risk of unattended, unmonitored failures. Risks during cleaning and inspection will result in removal of debris coating in pipelines. There is no risk of increasing pipe wall loss beyond what currently may exist. The technologies selected for inspection do not require aggressive cleaning to bare pipe wall, but only sufficient cleaning for inspection tools to navigate. This is generally approximately 25 mm less than the pipe ID. Aggressive cleaning via scraper PIGS and/or high velocity jetting equipment will be avoided.

2.3 Obstructing Pipelines during Cleaning Processes

Risk Profile – Low

Mitigation Strategies

Cleaning and inspection tasks will result in deployment of full diameter and near full diameter PIG's through the pipelines. The first task will be deployment of cleaning pigs. Pipeline pigging is completed in a progressive manner, starting with soft, undersized pigs and progressing to firmer, full sized

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products as required to achieve the desired level of cleaning. More aggressive pigs are not deployed until previous pigging attempts are proven successful.

Typically, cleaning pigs are deployed and advanced by differential pressure flow through the pipeline. Where possible, this method of deployment will be used (forcemains and watermains).

For the sewer siphon pipelines, where adequate pressure differential cannot be achieved, PIG's will be tethered in the event that they become lodged and require removal. It is our experience based on siphons inspected in this program, and preliminary SONAR inspection data captured under other programs, that the gravity siphons contain only small amounts of debris and experience has demonstrated it is likely to be sufficiently removed by soft pigs alone. It is also noted that for the gravity crossings, cleaning can be supplemented by conventional sewer jetting equipment. Flushing pressures would be limited to prevent potential damaged to corroded pipes.

Pipeline cleaning will be undertaken by an experienced pipeline cleaning company. Submission of Contractor Qualifications will be required as part of the tendering process to ensure the cleaning contractor has sufficient applicable experience with cleaning large-diameter sewage and potable water pipelines. Cleaning of the river crossing pipes will involve a combination of conventional sewer flushing (sewers and forcemains only) and foam/bristle pipeline cleaning pigs. A progressive cleaning program will be employed to reduce risk associated with obstructions within the pipeline by incrementally increasing the size, density and morphology/configuration of the pigs.

2.4 Obstructing Pipelines during Inspection Processes

Risk Profile – Low

Mitigation Strategies

Deployment of inspection PIG's will be undertaken by our subcontractor PICA Corp. PICA has extensive experience in deployment of their technologies. The chosen technology for these inspections do not require a tight to wall pipeline fit, and thus potential to lodge a tool in the pipe is low.

Inspection tools will be tethered on both ends in order to achieve accurate distance information during the inspection. Advancement of the tools is at a low speed, and utilizing controlled winching equipment and procedures, including pulling force monitoring. In the event tool advancement is compromised, tools can be retracted

As part of the inspection program, a gauge pig will be pulled through the pipeline after the pipe has been cleaned to assess the cleaning operation and to ensure the inspection tools are able to pass through the pipe. The gauge pigs are built by PICA specifically for their inspection equipment and we therefore do not expect any issues with obstructions during the inspection process.

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2.5 Structural Damages/Buoyancy Effects on Pipelines

Risk Profile – Medium

Mitigation Strategies

Many of the pipelines to be inspected under this program are located either above the river bottom, or dredged into the channel at shallow depths. Some pipelines are weighted with concrete weights, grout bags or other devices, however, the safety factors against floatation are low. It is imperative that pipelines are not dewatered or even partially air entrained during cleaning and inspection. It is important to ensure that sufficient flow is present behind the cleaning and inspection tools ensuring air entrapment does not occur. This will be closely monitored and specifically noted in the contract specifications.

3. Site Specific Risk Reviews

Sewer gravity siphon crossings will be inspected at Site 1 St James Interceptor Siphon, Site 2 Northeast Interceptor Siphon, Site 5 Assiniboine Park Siphon, Site 6 Munro Polson Siphon, and Site 7 Main St Interceptor Extension at Omand's Creek. Generally these sites require a low to moderate level of modification to facilitate cleaning and inspection, and are dual pipe crossing which offers a level of system redundancy.

3.1 Site 1a and 1b– St. James Interceptor Siphon

Inspection of the 500 and 600 mm pipelines of the St. James Interceptor Siphon will be completed individually, using similar flow diversion procedures. Inspection of the 600 mm pipeline requires diversion of flow to the smaller 500 mm pipeline. As this is the most critical case, risk assessment was completed on this scenario, and can be safety applied to both pipeline inspections.

For the 600 mm inspection, flow will be diverted into the 500 mm pipe as required for the cleaning and inspection operations. The 500 mm siphon is capable of handling dry weather flows for the duration of the cleaning and inspection work. This analysis is fully discussed in AECOM Technical Memorandum - St James Interceptor Crossing the Assiniboine River, dated 13-10-15. Diverting flow to the 500 mm siphon will be accomplished through the use of inflatable flow-through bypass plugs and a piping assembly. The flow-through plugs have been sized for a 375 mm flow-through capacity, which is sufficient to support daytime peak dry weather flows. The bypass may be left in place for an extended period of time given proper upstream water level monitoring. Peak daytime flows in the Westwood interceptor may cause sewage levels to overtop the weir in the outfall chamber adjacent to the upstream junction chamber. Therefore to prevent a CSO event, the sluice gate within this chamber must be closed or alternatively plugged and flow levels monitored.

The bypass plugs and diversion piping assembly will be installed during low night time flow periods. Installation of the bypass plugs potentially could be accomplished with live flow; however, the connection of the discharge piping and manifold assembly may require a complete stoppage of flow through the bypass during installation. If required, flow stoppage will be accomplished by either shutting down upstream pump stations, inflating temporary plugs in the upstream sewer, or a combination of the two.

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It is anticipated that the flow-through bypass plugs and piping to be installed for approximately 8 hr to facilitate the inspection work. Once inspection is complete the flow-through bypass plugs will be deflated and removed from the upstream piping and chamber.

In all diversion cases, flow monitoring will be in place and, if required at any time, inspections will be abandoned, flow through plugs deflated and service restored. Risk of loss of service at this site is therefore considered low.



Figure 1: Cleaning Pig Used on the St. James Siphon in 2012

The 600 mm pipe was partially cleaned during 2012 works, by deployment of a soft foam pig. During this attempt, the foam pig was largely undamaged (see Figure 1), which is indicative of minimal obstruction. This site was also inspected by SONAR in 2012. Based on preliminary SONAR data and the 2012 cleaning, it is expected that cleaning requirements for this site will be minimal, and the risk associated with the cleaning process is low.

Original Drawings of this site indicate the pipelines are buried with approximately 600mm of cover below the river bed. The pipelines are also weighted at bends. The level of cover is sufficient to prevent buoyancy of a completely empty steel pipe, however, due to potential erosion of the river bottom, it is recommended that the lower portion of the pipe be kept full during cleaning to ensure buoyancy is not a concern. This can easily be achieved by backfilling pipe as pigging progresses.

Cleaning and Inspection Procedure:

- Abandon the existing 300 mm river inlet connected to the downstream chamber, allowing full port access for removing tools in the downstream junction chamber
- Perform cleaning operation
- Perform gauging operation

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- Install bypass plugs and piping:
 - Close sluice gate to prevent CSO in adjacent outfall chamber
 - Install flow through plugs
 - Temporarily stop flow and connect by-pass piping
 - Reinstate flow
- Perform inspection
- Remove bypass plugs and piping

Key Risk Mitigation Items:

- 500 mm siphon and 375 mm bypass piping are capable handling peak dry weather flows
- Sewage levels upstream of the bypass plugs will be physically monitored during installation and use of bypass piping
- In the case of a wet weather event, flow-through bypass plugs can be quickly deflated and removed from the pipe restoring full capacity of the system
- The bypass piping assembly will be pre-assembled prior to flow shutdowns
- A minimum cleaning program is anticipated based on 2012 work
- It is anticipated the cleaning and gauging work will be completed under flow conditions thus reducing bypass requirements

3.2 Site 2a and 2b – Northeast Interceptor Siphon

Risk Assessment to be developed.

3.3 Site 3a - Newton Avenue Forcemain

Inspection of the 350 mm steel Newton Avenue forcemain crossing will involve modifying chamber piping in the upstream valve chambers. The Linden and Hawthorn forcemains will be re-connected as per the 1977 design for the addition of the second forcemain, depicted in Drawing 913. This allows for both pump station forcemains to utilize the 350 mm HDPE crossing for the duration of the work. Piping modifications in the south valve chamber will bypass the steel crossing pipe and allow for insertion of cleaning and inspection tools.

The following piping modifications are required:

- Installation of a 350 x 300 mm reducer in the U/S north chamber, reconnecting the two pump stations and forcemains
- Removal of the existing tee from the south chamber and installation of a 300 PEXPE spool piece, bypassing the steel crossing pipe
- Installation of piping for the purpose of launching cleaning and inspection tools

The piping modifications will require brief shutdowns of both pump stations prior to, and on completion of inspection work. To mitigate risks of flooding or CSO's during these operations, hydraulic modelling was completed to assess any shutdown limitations. This information is presented in a separate memorandum, Results of this modeling include:

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- Shutdowns of pump stations will only occur during dry weather flow
- Shutdowns of pump stations are limited to 12 hours, and will commence during nighttime periods to take advantage of low diurnal flow patterns
- Once modifications are completed, both pump stations can operate indefinitely on one forcemain, without adversely affecting the system operation. This eliminates time constraints for placing the 350 mm steel crossing back into service and further acts to reduce the risks associated with the cleaning and inspection work.

Upon completion of cleaning and inspection work, the chamber piping will be restored to its existing configuration.

To mitigate risk associated with pipeline modification, prior to undertaking pump station shutdowns and disassembling existing piping, all parts will be brought to site, existing piping and fittings measured, all prep work completed, and parts preassembled where possible. It is anticipated the time required to complete the piping modifications to be approximately 2 hrs per shutdown. Hydraulic modeling of the system indicates a potential pump station shutdown window of 12 hours. Once the piping modifications have been completed the system can operate on one crossing pipe indefinitely, reducing risk during the cleaning and inspection work.

As this is a pressure pipeline we anticipate the cleaning contractor will wish to clean these pipes using untethered pigs propelled by water pressure. Cleaning pigs will be launched from the upstream valve chamber and pushed through to the downstream manhole. Initial cleaning using convention sewer flushing equipment will be recommended to ascertain the volume of debris removal required.

This crossing is buried below the river bottom, with approximately 0.6 m of cover according to Record Drawings. Record drawings also indicate this pipeline has a 12.7 mm steel wall thickness which is relatively heavy for a pipeline of this diameter. With this mass, the pipeline alone is sufficient to resist buoyancy of an empty pipeline. However, as a cautionary approach, the pipeline should be kept partly full during inspection and cleaning activities. With the thickness of the pipeline depicted on the drawings, risks of structural damages and buoyancy are low.

Cleaning and Inspection Procedure:

- Complete upstream valve chamber piping modifications
- Perform initial cleaning of forcemain
- Perform pigging operation
- Perform gauging operation
- Perform inspection
- Restore upstream valve chamber piping to existing configuration

Key Risk Mitigation Items:

- Available pump station shutdown windows (12 hours estimate) are well in excess of estimated pipeline modification times.
- Limiting modifications to piping at one structure per pump station shutdown could be utilized to limit risk exposure
- All fittings and piping will be on-site and prepped for assembly prior to pump station shutdowns

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- Operating both Linden and Hawthorne pumping stations through a single forcemain has been demonstrated to not adversely affect pump station operation

3.4 Site 4a and 4b - St. James Forcemain

Inspection of the St. James forcemain requires developing access to both pipelines. Hydraulic reviews conducted for this forcemain, have indicated that available windows to complete modification work upstream of the river crossing valving is limited. The pipeline crossing is also located in a depressed section of the forcemain and river crossing, which would require dewatering of approximately 50 m³ of sewerage per shutdown. As there is no current draining capability at this site, it has been determined that the most appropriate way of modifying and accessing this site is downstream of the river crossing valve chamber.

The proposed modifications will require the installation of two wyes directly downstream of the existing upstream valve chamber for launching of cleaning and inspection tools. The single upstream 450 mm forcemain splits into two 450 mm steel crossing pipes at the upstream valve chamber on Assiniboine Crescent, resulting in full redundant capacity of the forcemain. Modification to the siphon piping will be completed one at a time to allow for cleaning and inspection. Based on this scenario, there will be minimal risk to loss of service of the asset. This scenario also does not require any upstream pump station shutdowns.

The location of the valve chamber on Assiniboine Crescent is located off right-of way, but on City owned property. The location of required excavations is, however, in close proximity to private property. The City of Winnipeg has a narrow easement for access and maintenance on the property, which will permit access as required for completion of the work (see Figure 2). Consultation with the land owner should be initiated well in advance of commencing the work however.

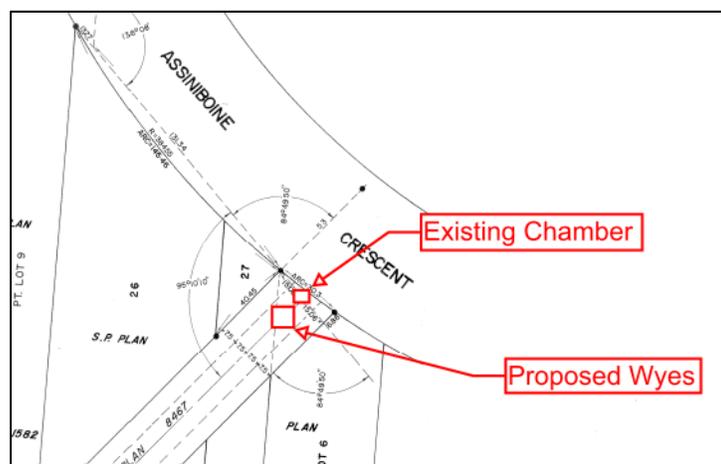


Figure 2: St. James Forcemain Easement

Conventional pipeline pigging will be required to clean this forcemain crossing. It is noted that the twinned portion of the siphon river crossing may have higher debris accumulation, due to reduced

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velocities through the crossing. The configuration of this crossing will allow for cleaning support using conventional flushing equipment if required.

The creek crossing detail shown on Record Drawing 425, indicate the crossing is encased in concrete (see Figure 3). This detail is sufficient to prevent buoyancy of the pipeline when completely empty. The encasement will also offer a high level of structural protection to the physical section of the pipeline under the creek. Other areas of the pipeline are located in areas that could be repaired by conventional excavations, in the unlikely event of a pipeline blockage or damage. Due to the redundancy of this crossing, risk of loss of service to upstream areas is low.

Cleaning and Inspection Procedure:

- Perform initial cleaning of forcemains
- Per side:
 - Install wyes on forcemains
 - Perform pigging operation
 - Perform gauging operation
 - Perform inspection
- Install blind flanges and bury wyes

Key Risk Mitigation Items:

- No pumps station shutdowns required
- No time restrictions for completing work as all flows may be diverted into the adjacent forcemain

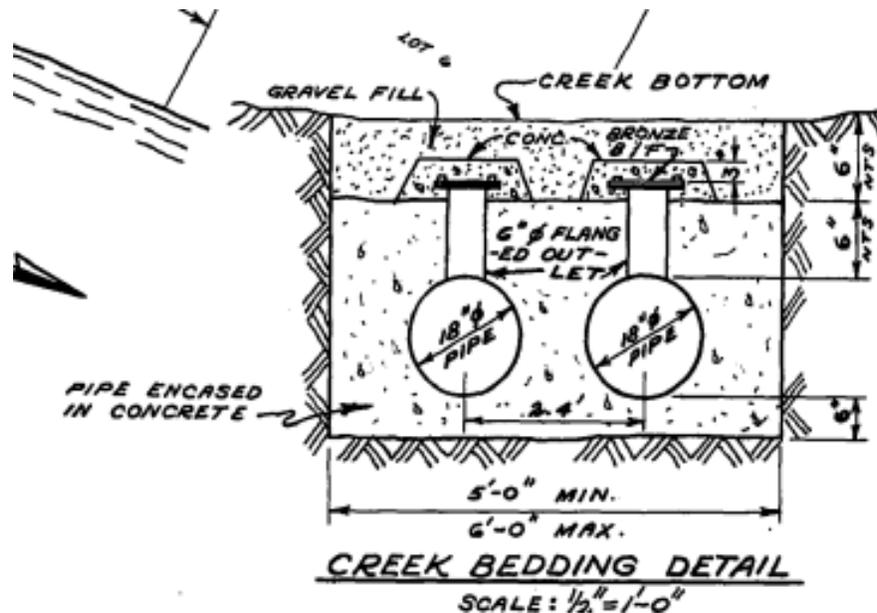


Figure 3: St. James Siphon Bedding Detail

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3.5 Site 5 - Assiniboine Park Siphon

Site 5 - Assiniboine Park Siphon is a small a single pipe siphon with a small contributing service area. This site was inspected successfully in 2012. For this site, risks of obstructing the site during inspection were mitigated by staging the work away from peak flows, and having septage hauling capacity available if required. Discussions with Assiniboine Park maintenance staff were held prior to inspection to determine low flow periods. No issues were encountered during inspection.

3.6 Site 6a and 6b - Munro-Polson Siphon

The Munro-Polson siphon is a twin gravity siphon between two existing pump/comminutor stations. To access the pipelines for cleaning and inspection, little to no system modification is required. The 300 mm pipe was successfully cleaned and inspected in 2012.

Inspection of the 450 mm pipe of the Munro-Polson Siphon is the more hydraulically critical of the two pipelines, as the 300 mm pipe will be required to convey a majority of the crossing flow during inspection activities. Hydraulic modeling of the system, presented on a separate technical memorandum, indicates the 300 mm siphon is capable of handling dry weather peak flows during the cleaning and inspection work.

Pigging and gauging of the siphon will likely be undertaken during flow conditions, as they were in 2012. However launching of the inspection tool will likely require the construction of temporary flow diversions in both the upstream and downstream chambers to aide in inspection process.

While the flow diversions are installed water levels are expected to rise but not overtop the flow channels within the upstream chamber. During the cleaning and inspection operations the positive gate at the adjacent Monroe overflow chamber should be closed as a precaution against a CSO and upstream water levels monitored.

We don't expect rigorous cleaning to be required for the siphon based on the 2012 cleaning and inspection of the 300 mm pipe. This site also has been inspected in 2013 with SONAR technology. However, it is understood this data is not yet available. Similar to the St. James Siphon we anticipate the cleaning to consist of pulling a soft foam pig and a gauging pig prior to deployment of inspection tools.

The pipe crossings of the river at this location appear to be founded on bedrock at the river bottom, according to Record Drawing 503. A portion of the pipelines that are exposed appear to have been encased in concrete, as indicated on the records. However, as placement of this concrete at time of construction would be difficult, and condition of the ballasting cannot be confirmed, it is recommended to assume the concrete may not offer substantial ballasting. It is therefore imperative that the pipelines not be subjected to air entrapment or dewatering in vicinity of the exposed river bottom.

Cleaning and Inspection Procedure:

- Remove internal components from existing gate valves to provide full port access to pipe
- Perform pigging operation
- Perform gauging operation

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- Install flow diversion within chambers
- Perform inspection
- Remove flow diversions

Key Risk Mitigation Items:

- 300 mm siphon is capable of conveying peak dry weather flows
- A minimum cleaning program is anticipated based on 2012 work. 2013 SONAR data of the siphon will be investigated prior to inspection if available.
- Cleaning and gauging will be completed under flow conditions, reducing bypass requirements
- Maintaining pipelines on a non-dewatered state during inspection.

3.7 Site 7 - Main Street Interceptor Extension at Omand's Creek

Site 7, the Main Street Interceptor is a large 600 mm single pipe siphon, crossing Omand's Creek. The siphon has a large contributing area including Ferry, Riverbend and Tylhurst Combined Sewer Districts. The crossing of Omand's creek is non-redundant, single pipe crossing.

The geometry and construction of this site are poor for cleaning and inspection tool access, due to undersized manholes and sharp mitred pipe bends. Upgrading access to the pipeline would include installing new manholes at each end of the siphon, including within the travelled roadway of Portage Avenue, and at the sharp mitred bends. Costs of developing access to this pipeline were considered excessive relative to the replacement cost of the asset.

It was also determined from previous hydraulic studies, discussion with City staff and site observation that flows through this site were extremely high. Blockages or restriction on flow resulting from cleaning and inspection operations could not be tolerated without a high degree of risk of loss of service and potential flooding or CSO's. For these reasons, it is recommended this pipeline be removed from the inspection program. It is recommended that system redundancy be upgraded at this site before attempting inspection. It is noted that the site is still considered a high risk crossing, and as such, upgrading this site should be considered a high priority.

3.8 Site 8 - St Vital Bridge

To be developed.

3.9 Site 9 - Assiniboine Feedermain

The Assiniboine Feedermain crossing is a critical distribution feed to the west end of the City, connecting the Charleswood Feedermain and the Rouge Road Feedermain. It serves as the outermost loop in the system connecting the northwest and southwest quadrants of the City.

The largest risks associated with major water crossing is scheduling of the works away from high demand water seasons, and in consideration of other regional water operations to ensure an adequate level of service is maintained. While AECOM does not have knowledge of all regional

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operations that could affect works at this site, it would be expected that Haney-Moray Feedermain and Midtown Feedermain would likely need to be in service.

It is expected that an out of service window for this crossing would be in the order of 3-4 weeks to complete system modifications, cleaning and inspection, and returning pipe to service, including:

- Approximately 3 – 5 days per side for system modifications
- Approximately 2 days for cleaning operations
- Approximately 1 day for inspections
- Approximately 2 days for system flushing and preparation to return to service
- Approximately 5 days for disinfection, sampling and health tests.
- Approximately 2 days to return site to service.

It is also noted that the pipeline at this location is a shallow cover main. This crossing however, is weighted with concrete ballast blocks as indicated on Drawing D-1111. Floatation concerns with this crossing are minimal. It is not however expected that the main will be fully dewatered in any event.

Based on a review of site drawings, the existing valve chambers at the site are not adequate for launching of inspection tools and cleaning devices due to presence of butterfly valves. To reconfigure the piping to access the pipe would require complete reconfiguration of piping and replacement of the existing butterfly valves. It has been determined that the most economical approach will be to install wyes on either end of the watermain crossings for launching and receiving cleaning and inspection tools. After completion of the work the wyes will be blind flanged and left in place for future cleaning and inspection operations.

Based on experience and the 2012 work on the North Kildonan Feedermain and the recent reconstruction of the Midtown Feedermain bridge crossing we are expecting moderate tuberculation within the watermains. The tuberculation will be removed to a level that's required for inspection through the use of form and coated foam pigs using untethered methods.

3.10 Site 10 - Goulet –Doucet Watermain

The Goulet-Doucet watermain traverses the Seine River between Goulet and Doucet Avenues. It is part of an extension of the Marion Feedermain from the MacLean reservoir complex. There are no valve chamber installations in near vicinity of this crossing location suitable for deployment of tools. Mains on each side of the site are also Asbestos Cement pipe.

The largest risks associated with major water crossing is scheduling of the works away from high demand water seasons, and in consideration of other regional water operations to ensure an adequate level of service is maintained. While AECOM does not have knowledge of all regional operations that could affect works at this site, pending valve replacement work at the Maclean Reservoir should be considered in City Hydraulic analysis of this site.

It is expected that an out of service window for this crossing would be in the order of 3-4 weeks to complete system modifications, cleaning and inspection, and returning pipe to service, similar to that noted above for the Assiniboine crossing.

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The crossing at this location is a buried crossing with a minimum of approximately 1.5m of cover, according to Drawing P 123. Buoyancy will not be an issue. It is not however expected that the main will be fully dewatered in any event.

3.11 Site 11 - Kildonan-Redwood Feedermain

The Kildonan-Redwood Feedermain is a unique configuration consisting of an approximate 12.5 m vertical drop on the west bank, a length of pipe supported in a horizontal tunnel liner, and a buried river crossing section (see Figure 4). This site will offer some unique access challenges for cleaning and inspection.

Due to main configuration, the access to the pipe would be from existing chambers on each side of the river. West side access would be a vertical approach through the crossings west tunnel shaft, while the east side will require partial disassembly of chamber piping.

For the west pipeline access, several mains will require isolation including 2 local 250 mm watermains on Redwood, a 400 mm offtake main heading south, and the 600 mm Feedermain extending west from the site. Removable concrete panels will be the primary vertical access to the vertical pipeline section. A 600 mm x 350 mm side outlet tee will require removal, and a vertical launch barrel installed.

For east side access, the Feedermain will require isolation at a valve chamber immediately east of the offtake chamber, and the 300 mm offtake valve closed. The existing 600 mm tee, spool piece and slip coupler will require removal. It is likely that an excavation to the top of the valve chamber will be required to facilitate access to removable covers.

It is expected that an out of service window for this crossing would be in the order of 3-4 weeks to complete system modifications, cleaning and inspection, and returning pipe to service, similar to that noted above for the Assiniboine crossing.

This crossing is a shallow cover main through the in channel portion of the crossing, as indicated on Drawing 12529. The shallow cover and composition of the backfill is not adequate to resist floatation. It is imperative that this crossing not be dewatered during cleaning and inspection activities. Dewatering of the pipeline is not expected.

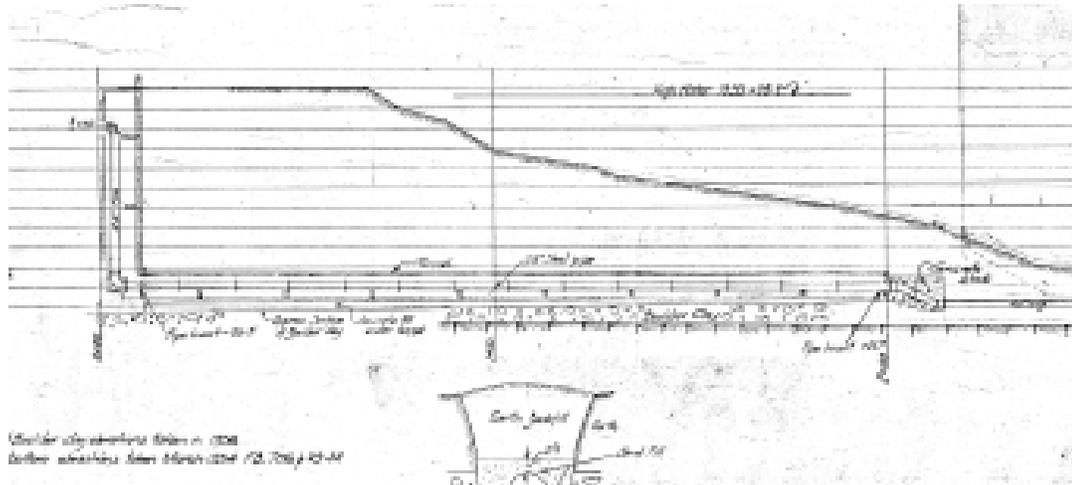
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Figure 4: Kildonan-Redwood Feedermain Profile

3.12 Site 12 - Maryland Watermain

The Maryland watermain crossing is a 250 mm steel pipe crossing, located between the two Maryland bridges. Access to the site is somewhat limited, but due to the smaller size and depth of the main, it will be accessible by moderately sized equipment.

Due to the size of the main, it is not expected to result in significant changes in service in the area. However, it is located in close proximity to a hospital, and as such, water quality concerns and redundancy of service could be a concern. It is also noted that due to system configuration and valving, one service line to the Misericordia Hospital will be required to be taken out of service during excavations. The effects and of disruption could be minimized by installation of an additional main line valve as depicted in Figure 5.

It is expected that an out of service window for this crossing would be in the order of 3-4 weeks to complete system modifications, cleaning and inspection, and returning pipe to service, similar to that noted above for the Assiniboine crossing.

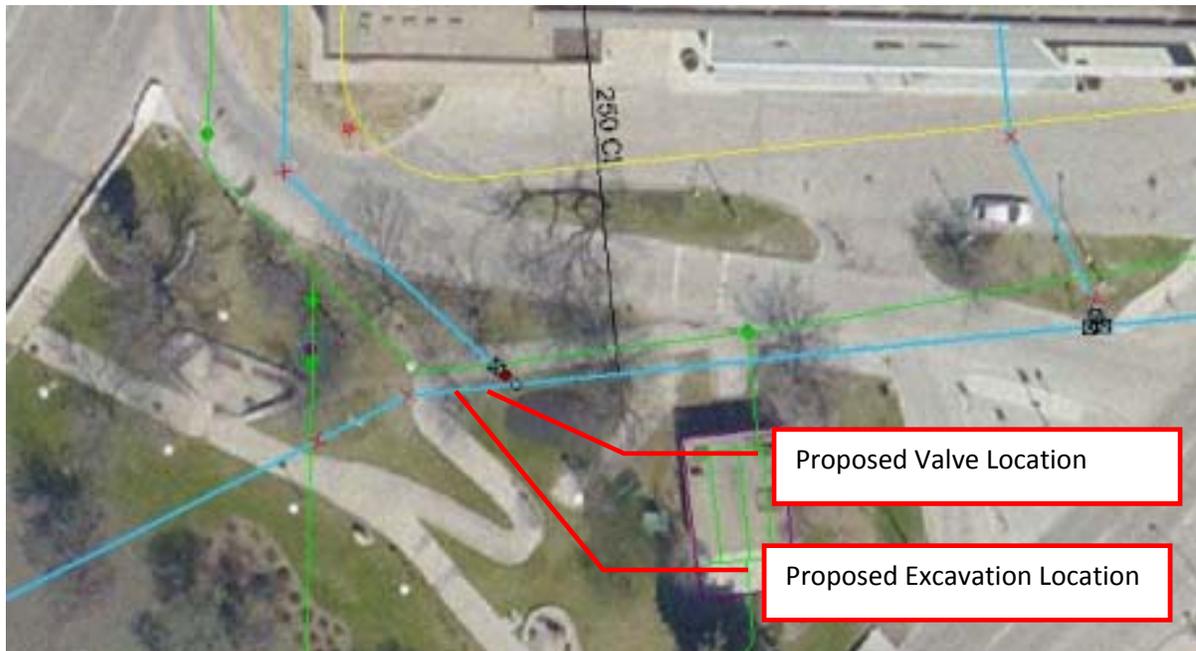
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Figure 5: Maryland Proposed Valve

The pipe crossing at this location appears to be a surface installation on exposed bedrock in the channel according to Drawing WA 19059. For this size steel pipe, the weight of the pipe is sufficient to resist buoyancy in itself. It is not expected however, that the pipeline will be dewatered.

3.13 Site 13 - Kildonan Feedermain

Prior to development of risk assessment for this crossing, it has been determined by the City of Winnipeg that this pipeline will be replaced. No further assessment or work will be completed on this site.

3.14 Site 14 - St James Watermain

The St. James watermain crossing is a 450 mm diameter crossing of the Assiniboine River. Based on available records it appears to be a Victaulic jointed cast iron pipeline.

The largest risks associated with major water crossing is scheduling of the works away from high demand water seasons, and in consideration of other regional water operations to ensure an adequate level of service is maintained. While AECOM does not have knowledge of all regional operations that could affect works at this site, consideration of this crossing being out of service while the Assiniboine crossing, Moray Feedermain and Midtown Feedermain should be made in assessing system impacts.

The crossing will require some additional investigation to determine exactly how connections to this main can be accomplished. According to available records the St. James watermain was constructed in 1930 using Victaulic jointed cast iron pipe. AWWA pipe of this vintage came with two different OD's

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depending on the pressure class and the possibility for two different joints styles. The difficulty of connecting to the pipe will be highly dependent on the OD and joint type used in the original construction. We recommending that a section of the pipe be exposed to determine the OD and joint type used. With this information we can have the correct materials on site prior to shutting down the watermain and exposing it for the hard connection.

It is expected this watermain crossing will have to be removed from service for 3 to 4 weeks, while developing access points and completing cleaning, inspection, and disinfection work. The impacts of removing these pipes from service on the City's regional water distribution system will need to be assessed by the City. Once work has been initiated on the mains they will require disinfection, thus limiting the ability to put them back into service should the need arise.

Based on experience and the 2012 work on the North Kildonan Feedermain and the recent reconstruction of the Midtown Feedermain bridge crossing we are expecting moderate tuberculation within the watermains. The tuberculation will be removed to a level that's required for inspection through the use of form and coated foam pigs using untethered methods.

Adam Braun, P. Eng.
Municipal Engineer
Community Infrastructure
/gms

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Project Manager
Community Infrastructure

Appendix B

Geotechnical Site Inspections

Technical Memorandum

To	File	Page	1
CC			
Subject	Geotechnical Site Inspections High Risk Water and Wastewater River Crossings		
From	Darren Yarechewski		
Date	August 28, 2012	Project Number	60270487

River bank stability is an important factor that can affect pipe condition. Instabilities have the potential to cause pipe damage and complete pipe failure owing to the massive weight of soil combined with shearing action that can be delivered to the pipe structure. Recognizing the signs of river bank instability can provide clues to the potential for future long term stability and in turn the potential for impact on the pipe structure. The goal of the geotechnical site inspections conducted for the high risk water and wastewater river crossings is to assess the current level of river bank instabilities.

1. Signs of Past Bank Instability

1.1 *Factors in Bank Stability*

In this memo “instability” will be used to describe evidence of river bank displacement either from the past or potentially in the future. The term “instability” when referring to a river bank can be controlled by different causal factors. If there’s no evidence of bank displacements or indicators associated with bank displacement, then it is considered that there is no evidence of bank instability present. If there is evidence of displacement, either at present or in the past, then there is evidence of bank instability. More severe evidence of instability may be termed a failure. For example, erosion of a river bank toe is not evidence of instability since other evidence of river bank displacement may not exist. If toe erosion has resulted in subtle grade changes farther upslope, then instabilities are evident. Further if a slump block is present along with a head scarp, the bank is considered to have failed. In general the term “instability” is used to point to the potential or subtle evidence of bank movements and that the bank is potentially unstable and “failure” is used when definitive evidence of bank movement exists that has resulted in large movements of the bank.

The purpose of this work is to observe whether or not there are any signs of present bank movements and whether or not potential movements may occur in the future. Not to overly generalize, but the majority of river banks in the City have undergone some type of failure in the past and often the question is how active that past failure is at present or may become in the future and to what scale that may encompass the river bank.

River bank stability is a condition of equilibrium that is dependent on the cross-sectional geometry, soil strengths, and groundwater conditions. Much like a balance scale, the combination of soil weight and strength on the upper bank is supported by the soil weight and strength at the bank toe. Remove material from the bank toe and support for the upper bank is removed. Remove enough toe material (support) and the upper bank will move down the bank toward the river to reach a new state of equilibrium.

Failures on a river bank result from erosion or loss of soil strength. The factors that affect erosion relate to the morphology of the river. Outside bends (concave curvature) of rivers will have faster currents than inside bends (convex curvature) with the result of more erosion on outside bends and greater deposition (accretion) on inside bends. If a river bank toe is armored with loose rip rap with sufficient sized rock, the tractive forces of the river will not be able to remove toe material. If the toe is not armored, the fine grained soils can be removed and result in successive reduction in river bank toe support. The process of toe support loss is progressive. Sufficient toe support may initially exist with a factor of safety significantly greater than unity (factor of safety greater than unity is stable and less than unity is unstable). However, much like the river currents gradually erode the toe of the river bank so the loss of toe support also gradually erodes the factor of safety until a point is reached where the equilibrium is not sustainable and a failure occurs.

Factors that affect the strength of the soil also play a role in river bank stability. Strength also is dependent on the soil type. In general there are two types of naturally occurring soils on the City's river banks consisting of lacustrine and alluvial. Both are clay-based soils but the lacustrine soils will have greater plasticity and lower internal friction angle. Lacustrine soils were deposited during the last retreat of the glaciers in a lake environment in post glacial Lake Agassiz. Alluvial soils are more recent depositions, occurring after glacial times and relate to the meandering of the river channels and the process of erosion and deposition produced by the lateral river movements. Alluvial soils generally have lower cohesive strength and a higher friction angle. This allows alluvial soils to stand and at greater vertical inclination but are easily prone to erosion since alluvial soils generally contain a greater sand and silt fraction than lacustrine soil and lower plasticity (cohesion).

Closely tied to soil strength is the pore pressure related to groundwater conditions. This can be the result of changes in river level or changes in groundwater pressures in the underlying glacial tills. Since alluvial soils have greater hydraulic conductivity, excess pore water pressures can be dissipated rather quickly and reduced strengths can be minimized. The more impervious lacustrine soils do not allow for rapid dissipation of excess pore water pressure and there is greater potential for loss of strength in terms of reduced effective stress and the potential for river bank failure.

These soil conditions are worthy of bearing in mind when examining the plan view of the sites. From previous work an approximate boundary between alluvial and lacustrine soil has been drawn based on aerial photograph interpretation and terrain analysis. The alluvial / lacustrine boundary is an approximation and it would require additional subsurface investigations to prove the boundary location. The alluvial / lacustrine boundary mapping provides a regional understanding of the soil conditions relative to the river morphology.

1.2 *Indicators of Bank Instability*

The following are descriptions of some of the indicators of bank instability that are expected when conducting an inspection of a river bank.

Scarps: are steep discontinuous portions of river bank. Often they are near-vertical or slightly inclined away from the river. Scarps can be present as a result of erosion or by downward movement of blocks of soil. If located near the river bank toe, a scarp may be caused by erosion or bank displacement (toe instability). If located higher on the bank where the bank is not exposed to the effects of river currents, either because of the elevation or sheltering by trees, the presence of scarps will be related to past bank displacements. It is sometimes difficult to discern between the cause of scarps at the bank toe either by erosion and shallow instabilities. A further clue that a scarp is caused by bank shallow or deep-seated bank instability is the shape of the scarp in plan view. Arcuate scarp patterns can delineate the lateral extent of a slump block. A head scarp that fully delineates a slump block will have this arcuate alignment and extends from one point that meets the river edge, extend up the bank to an apex, and then curve toward the river returning to the river edge. More often only portions of a scarp will be evident for a deep-seated failure. Recent indications of movement will have well-defined edges to the scarp and a fresh appearance to the soil surface. Slickensides may be present on the scarp face which are polished clay due to the action the two soil masses sliding against each other.

Slump Blocks: occur when a mass of soil moves downward and toward the river bank. The scarp inclination at the head of the block provides clues as to the angle of the upslope failure surface of the block. The original ground surface on top of the block may translate downward and maintain its former orientation or the ground surface may rotate back toward the upper bank. If this reverse rotation occurs the surface of the slump block will be sloped toward the head scarp. Precipitation and runoff that lands on top of the slump block will flow toward the head scarp and can feed water into the slip plane of the failure causing further loss of soil strength and further bank displacement.

Retrogressive Failure: is a condition where lower slump blocks fail and move toward the river thus removing support for upper bank soils and resulting in creation of another slump block. Multiple slump blocks can result in a stepped appearance in the bank with multiple scarps. The process continues until equilibrium is attained between the bank geometry and failed soil strength.

Erosion: is the removal of soil due to river current and wave action. Depending on the elevation of the river water surface, erosion can occur at different elevations on the bank depending on the level of erosion protection afforded by vegetation or other structures.

Tension Cracks: are cracks in the soil surface and in plan are often near-parallel to the river alignment initially. The tension crack may appear with the elevation on both sides (upslope / downslope) of the crack at the same elevation. If the ground surface on the side of the crack closest to the river is lower, the tension crack has developed into a scarp. If the the edges of the tension crack are clearly defined, the tension crack may have occurred recently.

Structures: provide indication of bank instability by the presence of cracks. Brittle components of a structure such as brick or other masonry work and concrete are particularly useful. Soil pulled away from a foundation wall or components out of vertical alignment are also potential evidence of bank

instability. Fence posts and fence alignments also provide evidence of bank displacement (discontinuity in top elevation of adjacent posts) and erosion (loss of fence portions closer to the river edge).

Pavement: can allow tension cracks to remain evident. Small tension cracks in clay soil tend to refill with soil and water runoff or can swell due to the clay swelling potential. Asphaltic concrete pavement, since it is a flexible pavement but does not self-heal, generally shows tension cracks more readily and can show subtle patterns in the tension cracks. Concrete pavements tend to bridge across tension crack locations and will not allow evidence of a tension crack to become evident until a tension crack becomes a significant size.

Leaning Trees: indicate that either shallow or deep-seated bank displacements are present. When the trees have toppled near the shoreline the root mass and attached soil may have been undermined by erosion.

Other Instrumentation: may be in place from other geotechnical work. Protective casings extending out of the ground with locks attached or flush-mounted covers indicate that previous geotechnical investigations have been conducted at that location. The casings may be protecting a piezometer installation that is used to measure groundwater levels or it may be protecting a slope inclinometer casing that is used in the measurement of long term bank displacements. The installations alone do not indicate that river bank instabilities are present but they do indicate that others have monitored the condition of the river bank for evidence of instabilities. This information can be very valuable in the assessment of present river bank instability since information gathered from these installations is quantitative as compared to the qualitative nature of a visual site inspection as conducted under this work program.

2. Field Program

Inspections were conducted from June 22 to 27, 2012 by Darren Yarechewski of AECOM with site locations presented in APPENDIX A and a summary of results in APPENDIX B. The inspection consists of observations of potential instability features which will be summarized in this memo and a photographic record of the observations presented as digital image files with a listing of the photographs in APPENDIX C. Measurements were collected of pertinent landform features for later use in slope stability analyses.

There are 14 sites that were inspected as presented in Table 1. The site locations are presented in plan in Figure 1 by crossing number. The sites on the Red River are generally north of St. Vital Bridge extending to Chief Peguis Trail. Sites on the Assiniboine River extend along its length from Maryland Bridge to Berkley Street with neighbouring sites on Omand's Creek and Sturgeon Creek. One site is located on the Seine River. The the proper names of these water courses includes "rivers" and "creeks", this memorandum will use the term "river" collectively when discussing the crossings.

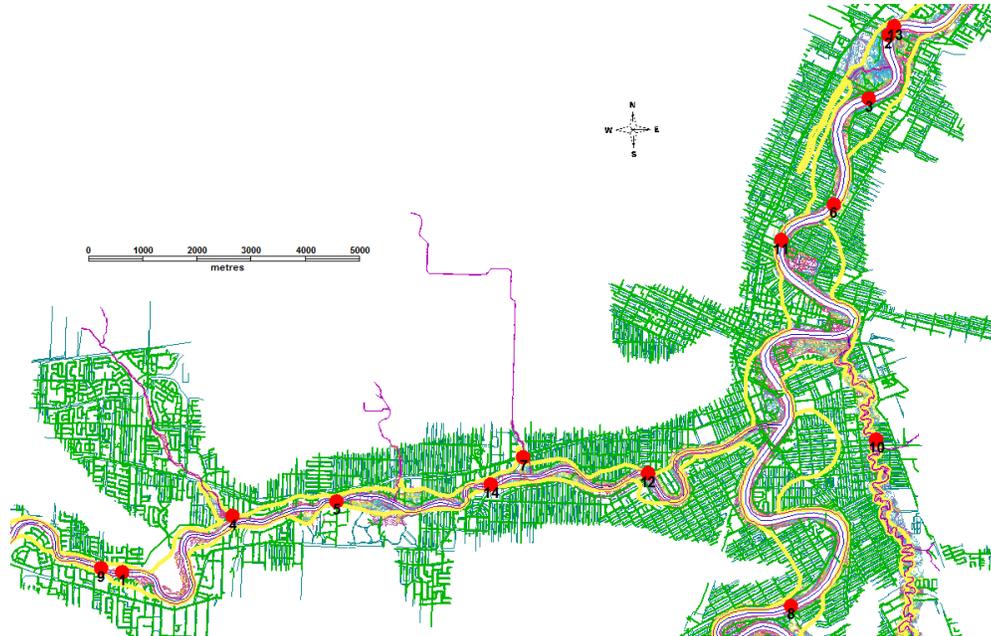


Figure 1 – River crossing locations by site number as described in Table 1.

Table 1: List of Site Inspections.

CROSSING NUMBER	LOCATION NAME	CROSSING TYPE	RIVER OR CREEK
1	St. James Interceptor	Sewer	Assiniboine River
2	Northeast Interceptor	Sewer	Red River
3	Newton Avenue Forcemain	Sewer	Red River
4	St. James Interceptor	Sewer	Sturgeon Creek
5	Assiniboine Park Siphon	Sewer	Assiniboine River
6	Munroe Polson Siphon	Sewer	Red River
7	Main Street Interceptor Extension	Sewer	Omand's Creek
8	St. Vital Bridge	Sewer	Red River
9	Assiniboia Feedermain	Water	Assiniboine River
10	Goulet Doucet Watermain	Water	Seine River
11	Kildonan Redwood Feedermain	Water	Red River
12	Maryland Bridge Watermain	Water	Assiniboine River
13	North Kildonan Feedermain	Water	Red River
14	St. James Street Watermain	Water	Assiniboine River

2.1 Site 1: St. James Interceptor (Assiniboine River)

2.1.1 North Bank

The right-of-way (ROW) terrain on the interceptor alignment consists of manicured lawn and landscaped bank. The site is located in a residential area on Assiniboine Avenue. Neighbouring properties exhibit evidence of a steepened lower ridge or former beach head. A steepened upper bank also exists in the vicinity of the chamber structure which shows no evidence of structural distress. There is no evidence of recent river bank displacement. No erosion is present at the river bank toe. The lower bank on the neighbouring properties has hummocky terrain and may have had instabilities in the past but no recent displacements are evident.

2.1.2 South Bank

Surface terrain is manicured lawn with a portion of the pipe located beneath the building footprint at 79 Elmvale Crescent. No significant trees are present on the river bank. There is no evidence of river bank instability present. A small scarp approximately 600mm high exists at the river edge due to erosion. Adjacent property at 75 Elmvale Crescent shows evidence of past fill placement with a higher lot grade than adjacent properties and a steep erosion scarp at the bank toe. The properties farther upstream and downstream (71 and 87 Elmvale Crescent, respectively) represent more natural bank conditions with steepened mid bank. No evidence of upper bank instabilities exists.

2.2 Site 2: Northeast Interceptor

2.2.1 West Bank

The interceptor alignment extends to the North End Water Pollution Control Centre through Kildonan Golf Course. The 28m expanse east of the fence bounding the golf course to the river edge is under natural conditions with large trees and underbrush. A pedestrian / bicycle path parallels the fence about 3m to the east of the fence. The crossing is located immediately south (upstream) of the Settler's Bridge (Chief Peguis Trail).

A scarp exists about 7m from the river edge with a height of 0.9m to 1.2m and extends 24m south (upstream) of the crossing alignment. Large trees located close to the top of the scarp are leaning with trees north of the crossing (downstream) having significant lean (Figure 2). Hydro poles located near the pedestrian / bicycle path and bridge (cables crossing over the river) are vertical and show no evidence of distress.

A vertical erosion face is present at the water's edge (200mm to 300mm high). No erosion protection is present and it appears that erosion of alluvial soil is occurring during high water events.



Figure 2 – Vertical erosion face and toppling trees at river edge.

2.2.2 East Bank

The ROW consists of manicured lawn and graded bank with a steeper lower bank near the river edge. Rip rap erosion protection extends from the bridge located to the north (downstream) and extends across the toe of the pipe crossing ROW, terminating immediately south of the ROW. Some sediment deposition may be occurring on the rip rap. The alignment is located immediately south of Settler's Bridge and north of 60 Whellams Lane. A chain link fence borders the south edge of the ROW.

An arcuate head scarp about 1.5m high is located south (upstream) of the ROW on the property of 60 Whellams Lane. Contours from 1998 show the scarp location in Figure 3 along with a potential extrapolation of the scarp across the ROW. Relative to the toe of the rip rap at the ROW, the head scarp is located about 20m from the river edge. The north limb of the scarp terminates at the south property line of the ROW (fence line). The south limb extends to the river edge. It appears that past grading of the ROW masks the presence of the scarp in the ROW. A potential failure surface may extend into the ROW. No tension cracks are present in the ROW. The toe of the river bank downslope of the arcuate head scarp is near-vertical and about 3m high with extensive erosion. The

toe has also receded farther in the upslope direction (conversely the rip rap protected toe at the ROW extends farther into the river channel).

There is no evidence of river bank instabilities north (downstream) of the ROW. The dual hydro poles supporting a cable crossing over the river show no evidence of distress.



Figure 3 – Scarp south (upstream) of ROW on east bank from 1998 contour data.

2.3 Site 3: Newton Avenue Forcemain

2.3.1 West Bank

This site is located at the intersection of Scotia Street and Newton Avenue (469 Scotia Street), south of Kildonan Park. The upper bank is manicured lawn and sparse trees (some of large diameter near the top of the bank) surrounding the station. The lower bank is steep and armored with concrete rubble rip rap (Figure 4). Field stone rip rap is present surrounding the outfall pipe. There is no indication of structural distress at the station nor is there evidence of river bank instability. Property north (downstream) of the crossing has concrete stairs leading down the river bank to the river edge and are in good condition suggesting no evidence of bank displacement, though retaining walls have rotated into the river. Limestone rip rap extends to properties farther upstream.



Figure 4 – Concrete rubble used for rip rap as erosion protection.

2.3.2 East Bank

Located in Fraser's Grove Park, the site consists of a shallow long slope leading from a berm that carries a pedestrian / bicycle path (that also forms part of the City's primary dike system) to the river edge. The terrain is manicured grass with large trees and no significant underbrush except near the river edge. At the river edge there is evidence of toe erosion (Figure 5) but no bank instabilities. Runoff erosion channels have also formed at some locations. Wave action has carved 150mm steps in the alluvial soil at some locations. No erosion protection is present.



Figure 5 – Erosion at bank toe.

2.4 Site 4: St. James Interceptor (Sturgeon Creek)

2.4.1 North Bank

The interceptor crosses through the residential property at 2610 Assiniboine Crescent passing immediately southeast of the dwelling and through a landscaped garden. The remainder of the surficial terrain is manicured lawn. The bank is gently sloping with no evidence of instability or erosion. The shoreline has a 600mm vertical edge (Figure 6) and appears to be a stable edge with no erosion. No erosion protection is present.



Figure 6 – Gentle slope on north bank with vertical edge of bank.

2.4.2 South Bank

The interceptor alignment passes through the northwest corner of 147 Ashcroft Point. The site is heavily treed with both large trees and underbrush. There is no evidence of river bank instability or erosion. Though the bank can be categorized as alluvial soil, erosion remains in check due to armoring at the riverbank toe consisting of large concrete rubble and large carved limestone blocks (Figure 7).

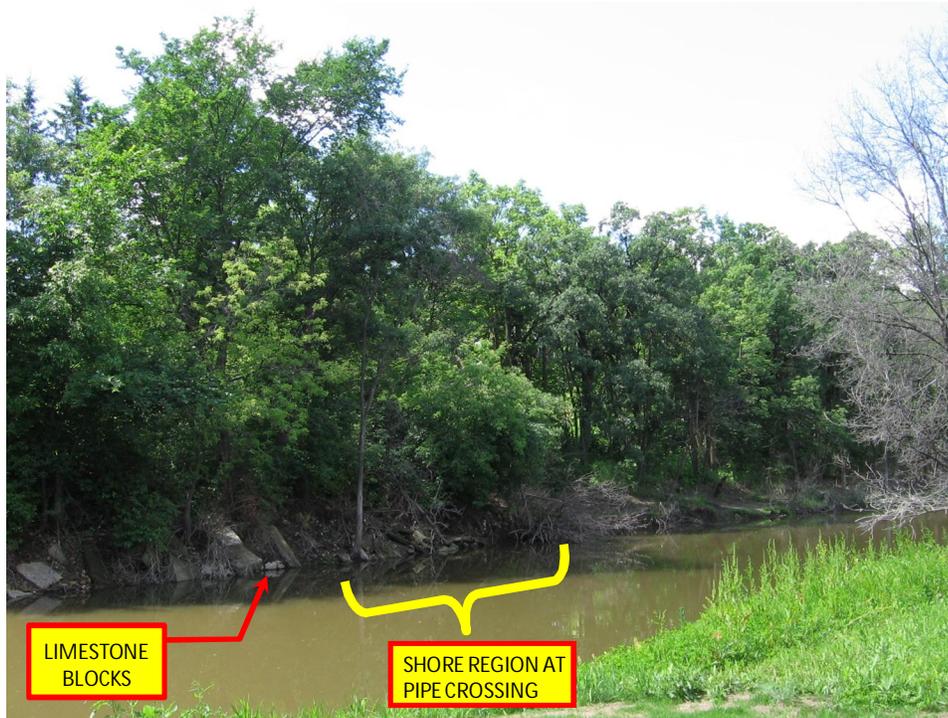


Figure 7 – Steep south bank with sparse limestone blocks and rubble at river edge.

2.5 Site 5: Assiniboine Park Siphon

2.5.1 North Bank

The north bank ROW is located west of 2194 Portage Avenue and consists of a graded bank, manicured lawn, and no trees. The ROW toe is protected with loose rip rap. Minor erosion is evident where the top of the rip rap meets the lawn. There is no evidence of bank instability within the ROW or distress in the structures.

Outside of the graded bank, steep and high scarps exist above a lower bench in the bank. East of the ROW (2194 Portage Avenue), 2.1m-high scarps are present while scarps to the west are 3.7m to 4.3m high (2220 Portage Avenue). The river bank toe upstream (west) of the ROW exhibits deposition with marsh grass present along the shoreline. This flatter toe environment transitions to the steep high scarp west of the ROW. To the east there is deposition of fine sand on the lower bench. The high scarps on the neighbouring properties are about 23m from the river edge.

2.5.2 South Bank

The south bank is located on Assiniboine Park Drive within Assiniboine Park and immediately north of the zoo. The bank is heavily treed with predominantly smaller trees and underbrush. A slump block (bank failure) extends across the pipe crossing alignment and is about 95m-long, parallel to the river, and about 6m wide at the crossing location (Figure 8 and Figure 9). The head scarp ranges from

1.2m to 2.1m in height. The soil at the scarp appeared fresh and may suggest relatively recent displacements. Erosion is not significant at the toe with the bank height at the river edge of 900mm. The river edge is near-vertical and well vegetated. Dead willows were present on the slump block.

West of the crossing (7.3m) a rock-lined channel extends through the river bank. Erosion due to surface runoff has incised a channel into the resulting in the boulders placed to armor the channel being sunk into the bank. There is no evidence of filter fabric beneath the armor to prevent migration of fines from beneath the armor. Water was running through the channel at the time of inspection.

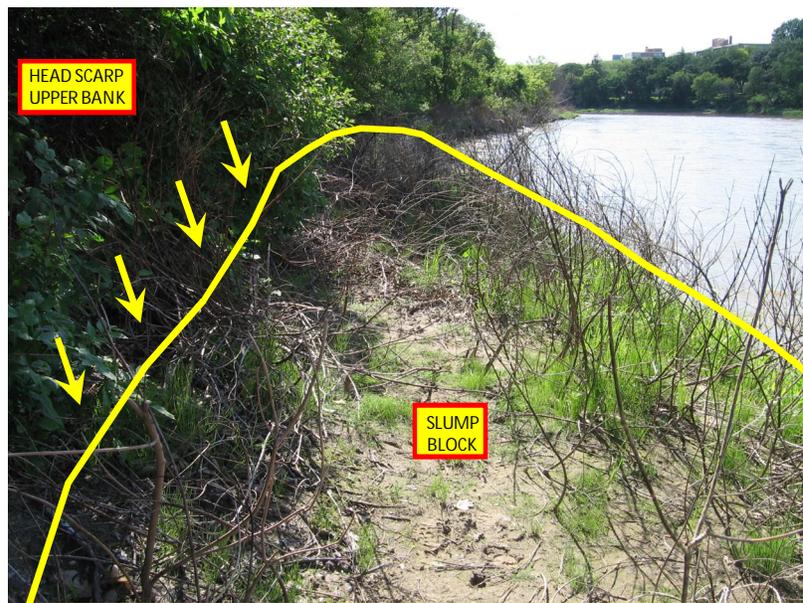


Figure 8 – Top of slump block facing upstream (west) from location of pipe crossing.

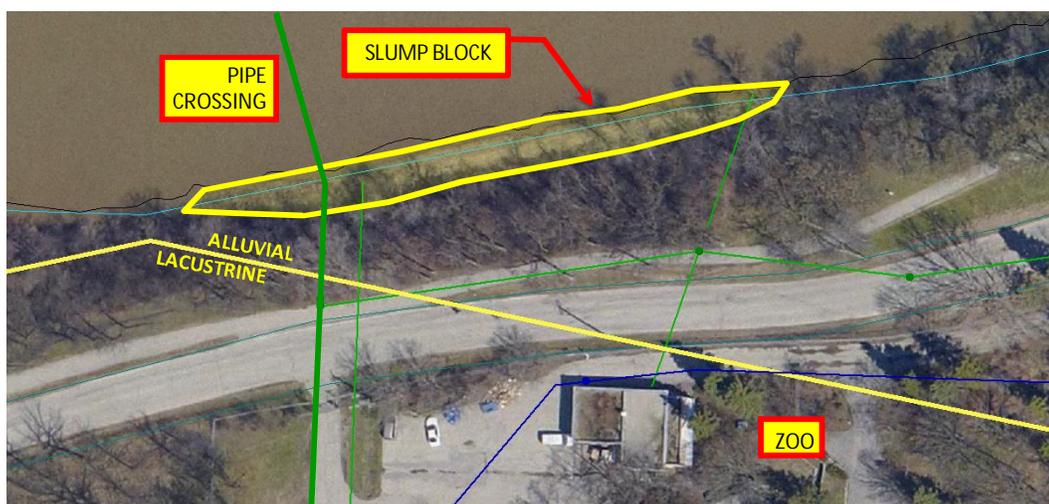


Figure 9 – Extent of slump block relative to pipe crossing.

2.6 Site 6: Munroe Polson Siphon

2.6.1 West Bank

This site is located at 75 Scotia Street at the intersection of Scotia Street at Polson Avenue. The ROW consists of manicured lawn and no significant tree vegetation. There is no evidence of river bank instability. The outfall pipe at the river toe is encased as a monolithic structure with concrete headwall and wing walls. Grouted rip rap is located immediately upstream and downstream of the wing walls with no adjoining loose rip rap. Erosion has occurred on the bank adjoining the grouted rip rap and resulted in the bank toe receding upland. The vertical difference between the top edge of the grouted rip rap and the existing shoreline surface is 400mm which translates to about 1.2m of horizontal bank recession (Figure 10).

Sparse rocks are located at the toe but in insufficient quantity to be considered as a concerted effort at rip rap armoring. Erosion at the shoreline has produced a 300mm to 600mm erosion scarp. North (downstream) of the ROW there is a 2.1m high scarp (79 Scotia Street). Limestone rip rap is present at the first property south (upstream) of the station (71 Scotia Street) and at the second property north (89 Scotia Street).



Figure 10 – River bank erosion relative to grouted rip rap location at outfall pipe.

2.6.2 East Bank

The ROW for this side of the crossing is a narrow strip immediately north of 526 Henderson Highway at the intersection of Henderson Highway at Munroe Avenue. There is no physical separation between the ROW and 526 Henderson Highway property (such as a fence, for example). At the time of our site visit in June and subsequently in August fills were located near the top of the bank and

adjacent to the chamber building along with construction of a concrete pad near the south edge of the property and rock landscape feature with concrete grade beam over the top of the bank (Figure 11). Placement of additional fill material at the top of a river bank has a destabilizing effect and is to be avoided.

The Waterways Engineer was contacted to alert of the possibility that this work may be undertaken without proper analysis by a geotechnical engineer and particularly that granular fill material is being stockpiled at the top of the bank. The Waterways Engineer indicated no Waterways Permit has been filed for this property and further had a meeting with the homeowner (August 21, 2012)). The homeowner was instructed to remove the fill or immediately retain a geotechnical engineer to provide recommendations and construction supervision for appropriate mitigations to bring it to a stability condition equivalent to, or better than prior to construction.

The mid bank and lower bank is treed with tall grass within the ROW. Similar to the west bank, the outfall structure is monolithic and in good condition. Grouted rip rap is present upstream and downstream of the wing walls with minor cracking. There are no cracks in the monolithic structure. Loose limestone rip rap extends across the remainder of the property toe (Figure 12). There is no evidence of river bank instability in upper bank.

Properties located south (upstream) of the ROW at 518, 502, and 500 Henderson Highway present current river bank failures with multiple scarps and evidence of retrogressive bank failure (Figure 13). These properties no longer have large trees on the property and had been landscaped to form manicured lawns. Aerial photographs from 2007 show large trees present at 500 Henderson Highway that were removed in the 2009 aerial photographs. In 2009, a head scarp and rolling terrain, indicative of bank failure, is present at 500 Henderson Highway. The large building at 488 Henderson Highway is not present in the 2007 and 2009 photos and large trees are present at the river bank toe. This toe vegetation was since removed to become the present condition.

The potential for these instabilities to progress northward (upstream) toward the ROW of the pipe crossing can be limited if site conditions are left unaltered at the properties 522 and 526 Henderson Highway located between the nearest failure (518 Henderson Highway) and the ROW. These properties presently retain large trees and vegetation at the river bank toe. For this reason, the construction activity immediately adjacent to the ROW coupled with evidence of bank failures to the south poses a potential hazard to the pipe crossing.



Figure 11 – Fill placed at and over top of bank on ROW by neighbouring property owner.



Figure 12 – Rip rap erosion protection at river bank toe.



Figure 13 – Neighbouring river bank failures upstream of ROW.

2.7 Site 7: Main Street Interceptor Extension

2.7.1 West and East Bank

Site 7 is located on Omand's Creek immediately south of the Portage Avenue bridge crossing. It's also located west of 1420 Portage Avenue. Since the site is contained within a small area it is treated here in its entirety. The slopes on both banks have been graded and protected with concrete articulating blocks (Tri-Lock) and grouted rip rap (Figure 14). The erosion protection is located at discrete locations on the bank surface and appears to coincide with specific water elevation events. The articulating blocks are about 3m wide and located on a higher portion of the bank. The grouted rip rap contains about 300mm diameter limestone and is about 600mm wide and is closest to the creek edge. Both types of erosion protection extend parallel along the creek for a distance of 30m south of the bridge abutment.

The banks are vegetated with tall grass and low shrubs. Presumably, the grading of the channel, erosion protection, and vegetation are a result of construction related to the piping buried at this location and the adjacent bridge. In this sense the channel is manmade as opposed to natural and the original design continues to function adequately with no evidence of bank instability. Further there is no evidence of structural distress in the bridge facade.

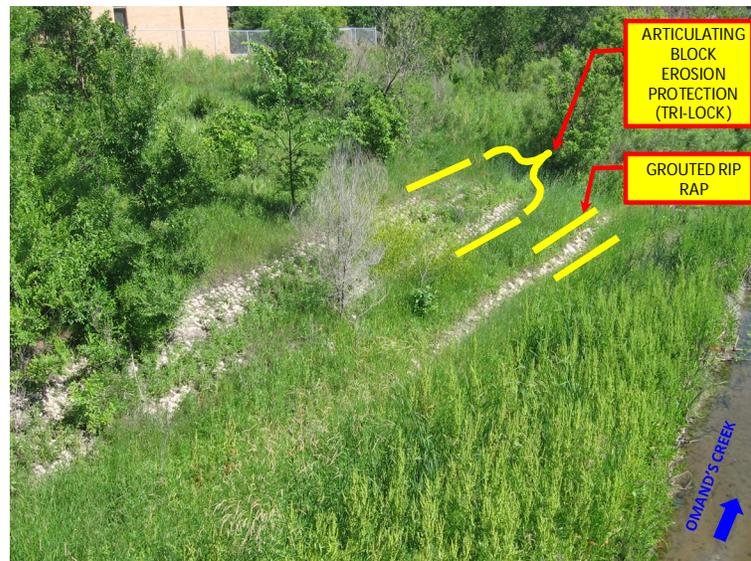


Figure 14 – Articulating block erosion protection (Tri-Lock) and grouted rip rap on east bank. Photograph taken from bridge at Portage Avenue.

2.8 Site 8: St. Vital Bridge

2.8.1 Pipe Located on Bridge

For both sites the risk to pipe is minimal since the pipe is mounted on the underside of the St. Vital Bridge and cannot be engaged by river bank failure as compared to a buried crossing. The first buried pipe sections (that consist of vertical piping) leading from the bridge to ground are further protected by the bridge piers since on both sides of the river the pipe is on the upslope side of the bridge pier. In effect, if a bank failure should occur, the piping at the point where it enters the ground is protected in the shadow of the bridge pier. In another sense, if bank instabilities occurred mitigative measures would likely first be addressed on behalf of the bridge structure since this is an asset of greater value. In following sections, the banks have been described solely based on the stability conditions though the risk to the pipe is minimal.

2.8.2 North Bank

The north bank is located on Osborne Street at Churchill Drive. The pipe enters the ground upslope of the second pier from the river edge (43m from river edge). There is no evidence of river bank instability at the site. Riverbank toe is armored with loose rip rap with no erosion present. Rip rap appears to have deposition in void spaces. The rip rap extends 45m upstream and 30m downstream of the nearest edge of the bridge. Outside of this protected area the river banks are steep with evidence of bank failure due to undercutting erosion on alluvial banks. Concrete drains show no sign of structural distress. Separation of grouted rip rap at drain outlet does not appear to be related to underlying bank stability issue and may be caused by post construction settlement, freeze-thaw action, or ice floe action (Figure 15).



Figure 15 – Cracking of grouted rip rap and separation from bridge drain outlet.

2.8.3 South Bank

The south bank is located on Dunkirk Drive at Kingston Row. The pipe enters the ground upslope of the first pier from the river edge (30m from river edge). There are no strong indicators of bank stability issues but subtle indicators do exist, potentially related to shallow creep displacements. No tension cracks or scarps exist on the upper bank. A slope inclinometer casing is located about 32m west of the bridge edge suggesting that past geotechnical monitoring has been conducted toward the potential for bank instabilities. Other subtle evidence of displacements are present in the concrete bridge drains where cracks are present with both separation and vertical offset. Some patching has been conducted on the west bridge drain which is farthest from the pipe structure.

Rip rap coverage is limited to 16m west (upstream) and 10m east (downstream) of the bridge. To the west, between a distance of 16m to 28m upstream, the rip rap becomes sparsely distributed and there is evidence of surficial displacements in the upper bank. In the downstream direction (east), beyond the 10m rip rap limit the upper bank is steep (2.4m high) and overhanging which is indicative of surficial displacements (Figure 16). This oversteepened section is concerning since there are several pieces of sewer infrastructure upslope of this bank that may be engaged by these displacements in the future. This oversteepened bank also lacks consistent armoring at the bank toe that could lead to further erosive effects.



Figure 16 – Erosion and potential surficial bank displacements downstream (east) of bridge downslope of other pipe infrastructure.

2.9 Site 9: Assiniboia Feedermain

2.9.1 North Bank

This site is located on Assiniboine Avenue east of 3276 Assiniboine Avenue at the intersection of Assiniboine Avenue at Rouge Road. The bank is graded with manicured lawn and no erosion protection at the river bank toe. There is no evidence of bank instability within the ROW at the mid and upper bank. A shallow scarp about 300mm high exists at the bank toe and extends to the west and potentially is a shallow slump block.

Banks representative of natural conditions are on the properties to the west and east of the ROW. Mid bank in these areas is steep with a lower bench leading to the bank toe. Slight reverse slope inclination of the lower bench east of the ROW is possibly the result of rotation of a past slump block. The lower bank west of the ROW is steeper than than at the east.

2.9.2 South Bank

The south bank is located at Southboine Drive at the end of Berkley Street with a ROW between 6161 Southboine Drive and 385 Berkley Street. The ROW is a graded bank. Manicured lawns exist on the ROW and adjacent properties. Trees are present on the adjacent properties. The lower bank on the adjacent properties consists of a small berm about 1.5m to 2.4m high and a gulley upslope of the berm which is part of the river morphology. There is no evidence of bank instabilities on the upper bank. The bank toe shows possible evidence of a small slump block but this feature was partially submerged at the time of our inspection and may only be an erosion feature. No erosion protection is present at the bank toe.

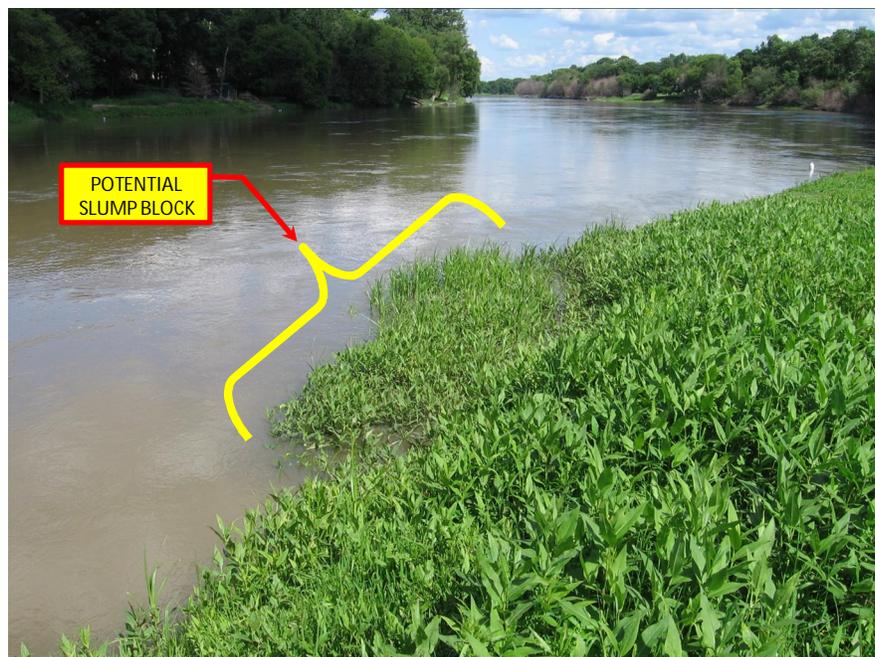


Figure 17 – Partially submerged potential slump block at river bank toe at crossing location.

2.10 Site 10: Goulet Doucet Watermain

2.10.1 West Bank

The east bank is located at the end of Goulet Street off Youville Street. The bank has a steeper upper bank at the end of the street that leads down to a broad floodplain that extends to the edge of the present river channel proper (Figure 18). Some fill has been placed at the upper bank. Based on measurements from the current geographic information system (GIS) the location of the bend in the pipe alignment is located on the flood plain. The location of the elbow and potentially the pipe alignment is marked by a low berm (less than 300mm high) that is likely the mounded trench backfill. The lower bank is vegetated with mature trees and low underbrush (less than 300mm tall). There is no evidence of river bank instability. Minor erosion is present at the river edge with a 900mm high erosion scarp. Grasses offer the only form of erosion protection with no other hard armoring.



Figure 18 – Flood plain at crossing location.

2.10.2 East Bank

This site is located at the west end of Doucet Street off Dufresne Avenue. Similar to the west bank, the east bank has a steeper upper bank at the end of the street that leads down to a broad floodplain that extends to the edge of the present river channel proper. The lower bank is vegetated with mature trees and low underbrush (less than 300mm tall). There is no evidence of river bank instability.

2.11 Site 11: Kildonan Redwood Feedermain

2.11.1 West Bank

The pipe crossing at the west bank is constructed as a vertical shaft and horizontal tunnel beneath the river bottom with the purpose of the pipe location in the bank being away from potential slip surfaces in the bank. The bank is located at Redwood Avenue off Main Street with the crossing alignment immediately north (downstream) of the Redwood Bridge. Adjacent to the abutments the upper bank is covered with grouted rip rap. Cracks are present in the grouted rip rap with separations as much as 50mm to 75mm on the north side and also vertical offset. The vertical offset at cracks is smaller on the south side. North of the bridge abutment, the grassed slopes present no indication of bank stability. The north side of the bridge is known to have past bank stability issues but are not currently evident in proximity to the bridge. Cracks in the grouted rip rap may be caused by post construction settlement and freeze-thaw cycles. Erosion protection is present south (upstream) of the bridge for a distance of 18m and beneath the bridge but no rip rap is present north (downstream) of the bridge. Rows of large limestone blocks are present along a pedestrian path beneath the bridge and have not shifted due to bank instabilities though the lower blocks may have shifted due to undermining and ice action. No cracks are present in the pavement at the top of bank. The south grouted rip rap contains a surface-mounted opening to a slope inclinometer installation.

2.11.2 East Bank

This site is located on Hespeler Avenue at Glenwood Crescent. Steep unstable banks are located on the pipe crossing alignment on the north (downstream) side of the Redwood Bridge (Figure 19). Unlike the west bank that was constructed in a shaft and tunnel, the east bank pipe was buried and therefore is located within the river bank. With no erosion protection present, the bank has eroded (2.4m height) and receded upslope. Large trees are presently anchoring the uplands. There is no evidence of bank instabilities beneath and south (upstream) of the bridge but erosion and bank recession has also occurred south of the bridge. Two large marine cables (75mm diameter protruding 700mm above the shore) are exposed and can be presumed to have been buried at the time of initial installation and provide an indication to the degree of erosion.

Properties farther north show more natural bank geometry with a steepened upper bank and flat bench leading to the river edge. These properties to the north also have loose limestone rip rap erosion protection at the shoreline from 166 Glenwood Crescent northward. The immediate property (164 Glenwood Crescent) north of the bridge, that's also affected by the 2.4m scarp at the river edge, does not have rip rap erosion protection.



Figure 19 – River bank erosion on pipe crossing alignment immediately downstream of Redwood Bridge.

2.12 Site 12: Maryland Bridge Watermain

2.12.1 North and South Bank

The north bank is located south of Misericordia Hospital at Cornish Avenue and the south bank is located at the intersection of Academy Road at Wellington Crescent. Both banks have similar characteristics with a steepened upper bank leading downslope to a flat bench followed by a steep toe at the river edge. The riverbank toes are about 1.5m high. Drainage near the bridges and leading to the river flows without erosion protection and as a result is producing deep erosion gulleys (600mm to 800mm deep) as shown in Figure 20. The exposed soils are silts and fine sands (alluvial deposits). The north bank is treed with low standing willows at the pipe crossing and the south bank consists of young trees and grass vegetation. There is no evidence of rip rap at the toe of either bank though occasional boulders are present. Dessication cracks were present in the soil on the lower bench but there is no evidence of tension cracks related to bank instability.



Figure 20 – Gully erosion near bridge pier without erosion protection on lower bank.

2.13 Site 13: North Kildonan Feedermain

2.13.1 West Bank

The west bank is located north of the Settler's Bridge (Chief Peguis Trail) at the end of John Black Avenue off Main Street. The site is immediately south of 2641 Scotia Street. The terrain is predominantly manicured lawn with pockets of large trees on the upper bank and large trees along the river edge. On the ROW, there is no evidence of bank instability. A corrugated metal pipe of approximately 600mm diameter extends from the river edge at the ROW. At the bank toe, grouted rip rap is immediately north of the ROW. South (upstream) of the ROW, toe erosion is occurring with large trees leaning into the river along with some fallen trees. A potential slope inclinometer casing is also located in the grove of trees south of the ROW 15m southwest of the crossing alignment. From the potential slope inclinometer location extending for an additional 18m upstream (15m to 33m southwest of the pipe crossing alignment), the river edge has toe erosion with lower bank instabilities, tree leaning, fallen trees, and a slump block with the toe of the failure mass moved into the river channel (toe bulge). No erosion protection exists south of the ROW.

The mid to upper bank shows no evidence of bank instability south (upstream) of the ROW. This area is on an outside bend of the river and bounded by the ROW to the north and Settler's Bridge to the south. This area may have been a former homestead as evidenced by the low (600mm tall) stone wall with concrete mortar and inset concrete steps. The stone wall is in good condition with minimal cracks and no evidence of influence by bank instabilities. There is some undulation to surface terrain

of the bank but no distinct pattern is evident to suggest river bank instabilities exist and these undulations may be a result of uneven site grading.

North (downstream) of the ROW there is the previously mentioned grouted rip rap and a private dock (2641 Scotia Street). Farther downstream there is evidence of sparse loose rip rap at 2647 and 2653 Scotia Street. Downstream of these locations there is no erosion protection present and similar erosion features as exist south of the ROW are prevalent.

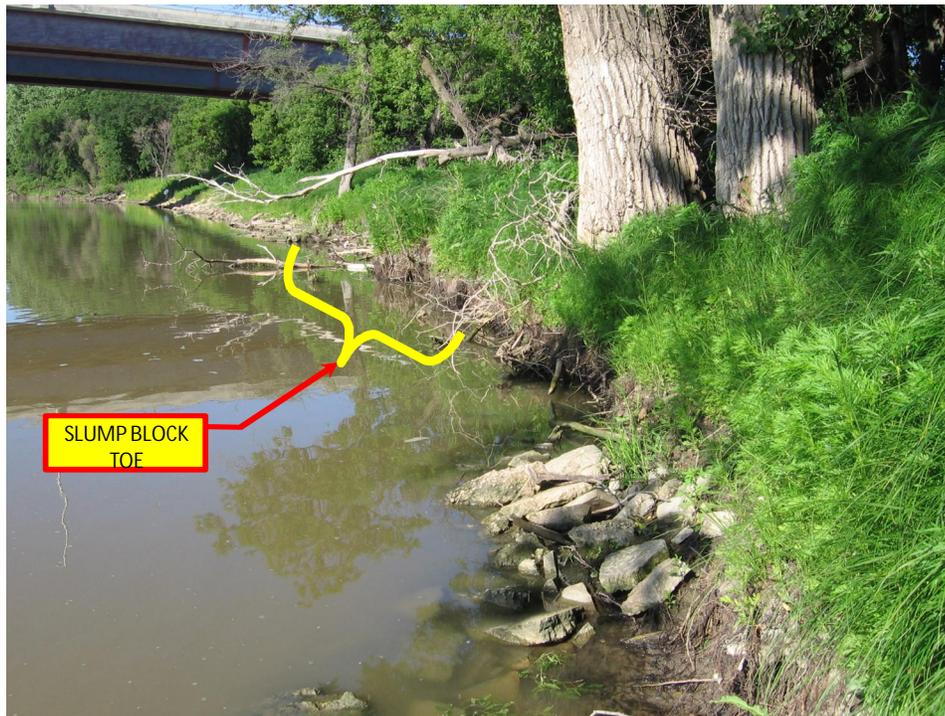


Figure 21 – Toe erosion and toe of slump block upstream (south) of pipe crossing alignment. Photograph taken from pipe crossing.

2.13.2 East Bank

Located immediately north of Settler's Bridge, this site is located east of Henderson Highway and consists of a forested bank on the inside bend of the river. The upper bank is a flat tree-covered plain with no evidence of river bank instability. The active area affected by river currents at the bank toe is eroding with a 900mm scarp at the high water level and a 600mm scarp at the shoreline at the river edge. There is no evidence of further instability issues. Exposed soils are alluvial silt. Rip rap is present near the bridge location but no erosion protection is at the crossing location.

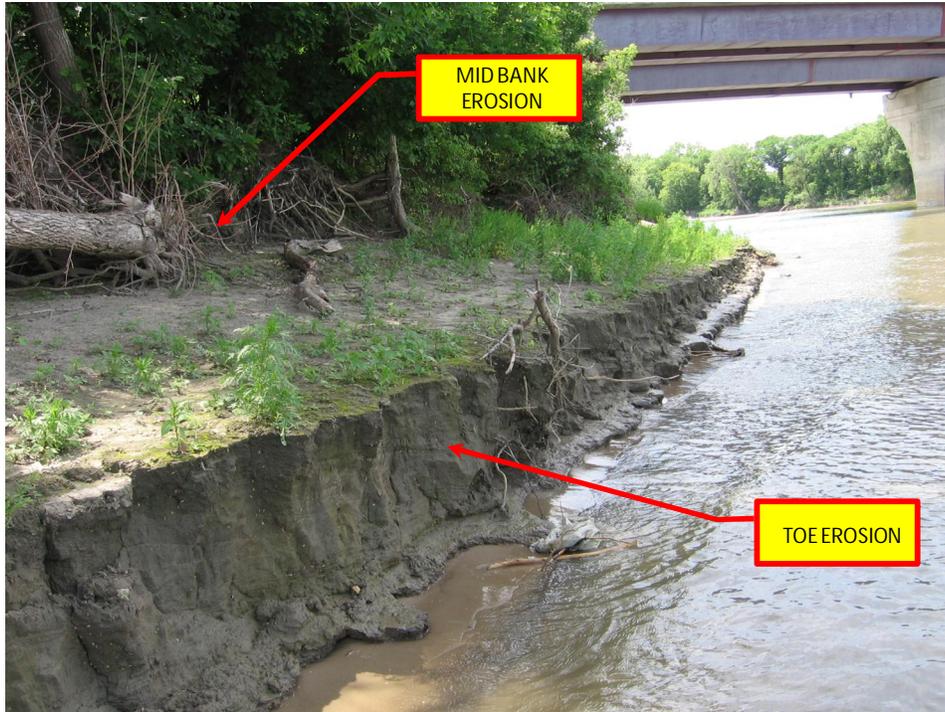


Figure 22 – Toe and mid bank erosion on east bank facing (upstream) south.

2.14 Site 14: St. James Street Watermain

2.14.1 North Bank

This site located south of the intersection of St. James Street at Wolseley Avenue West. The pipe crossing is immediately west of 1610 Wolseley Avenue West and enters this property near the river bank toe. This site is located immediately east of the St. James Bridge. The bank from the upper bank to the river edge consists of 1) steep upper bank about (3.7m tall), 2) wide flat bench with a berm and negative slope near river edge resulting in a shallow swale near the base of the upper bank face, and 3) vertical scarp about (1.8m tall) at the river edge. The upper bank contains large trees and some large trees near the bank toe with no significant underbrush.

Active erosion occurs at the bank face at the river edge. No erosion protection is present east of the railway bridge. There is no evidence instability on the upper bank or the bench leading to the river edge.



Figure 23 – Toe erosion with vertical bank at river edge at pipe crossing alignment.

2.14.2 South Bank

The south bank is located on Wellington Crescent immediately east of St. James Bridge and the abandoned railway bridge crossing. This site was visited on June 26 and August 15, 2012. The river water elevation decreased during this period and allowed more of the lower bank to be visible at the river edge. By way of comparison, at the James Avenue Pumping Station the water level decreased by 110mm on the Red River during this same period. The upper bank is steep and heavily treed along with thick underbrush. The edge of the river bank is actively being eroded with multiple slump blocks and toppled trees present. The toppled trees continued to have live (green) vegetation on the branches which suggest that the slump blocks on which these trees are rooted may have toppled in the recent past. Upon visiting the site a second time on August 15, 2012, a new slump block had toppled. There is no erosion protection downstream (east) of the railway bridge. At the railway bridge erosion protection consists of concrete armor in the form of burlap sandbags previously filled with concrete. This armor also protects the slope downslope of an outfall pipe (400mm corrugated metal pipe) located immediately east of the railway bridge. A second clay tile outfall (470mm outside diameter and 390mm inside diameter) does not have erosion protection downslope of the pipe outlet. Both pipes are located near the top of bank. Locally, a head scarp to a height of 600m was present during the June site visit with a tension crack 400mm wide and 400mm deep. The exposed face of this localized scarp had a shiny clay surface that was becoming a slickenside. The freshness of this face and lack of weathering or dessication also suggests this failure has occurred recently. The overall height of the erosion scarp is 2m.

The river level in June obscured the lower bank and did not afford a view of the mechanism underlying the toppling failures. In August, and upon viewing a new section of bank toppling, it is apparent that the bank is being undermined by the river current through erosion processes. Once the

overlying root mat and soil is sufficiently undercut the mass topples as a unit. As a result these failures are shallow-seated.

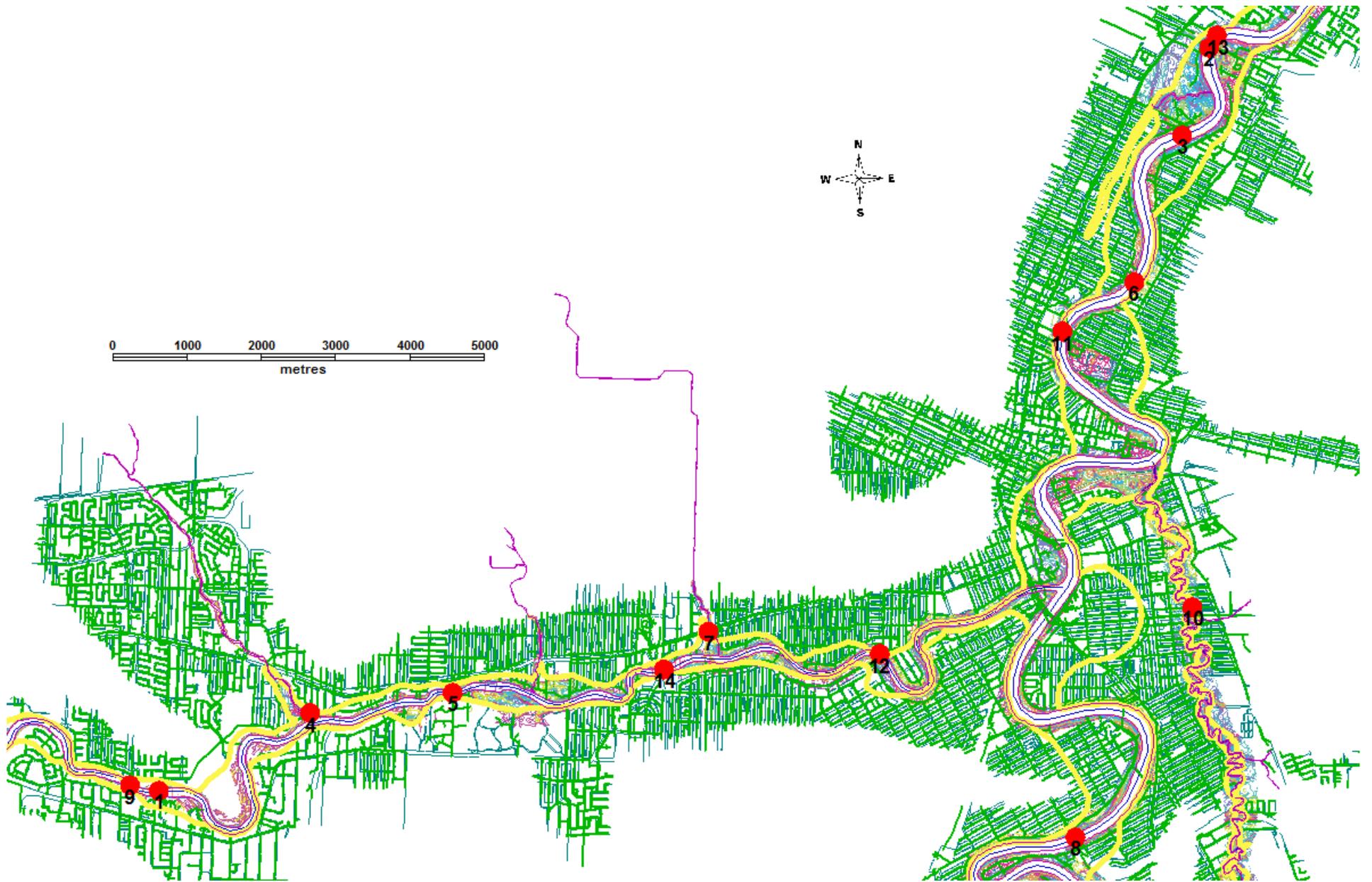


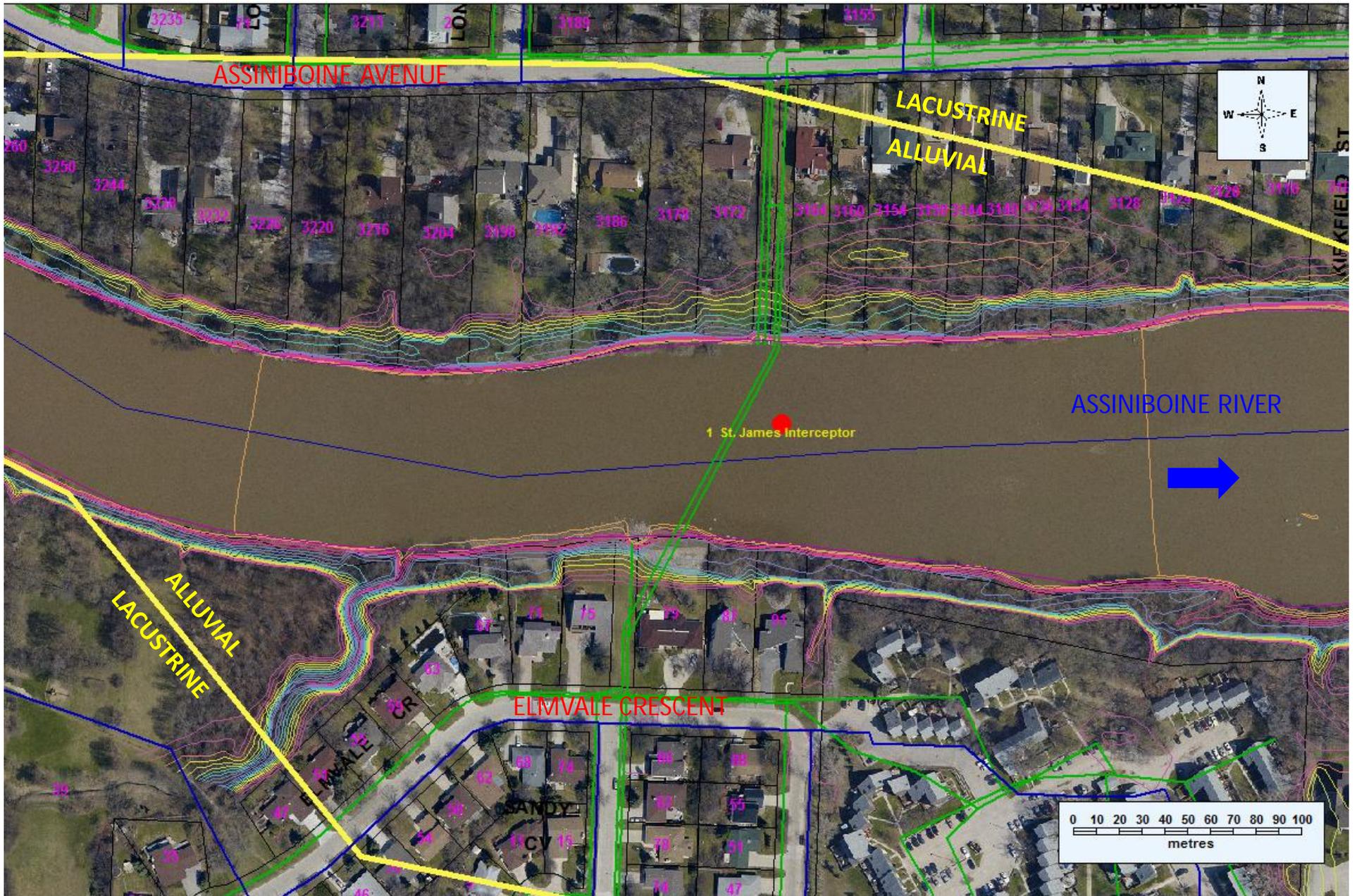
Figure 24 – Eroded bank with toppling blocks of overhanging soil with root mat and trees.



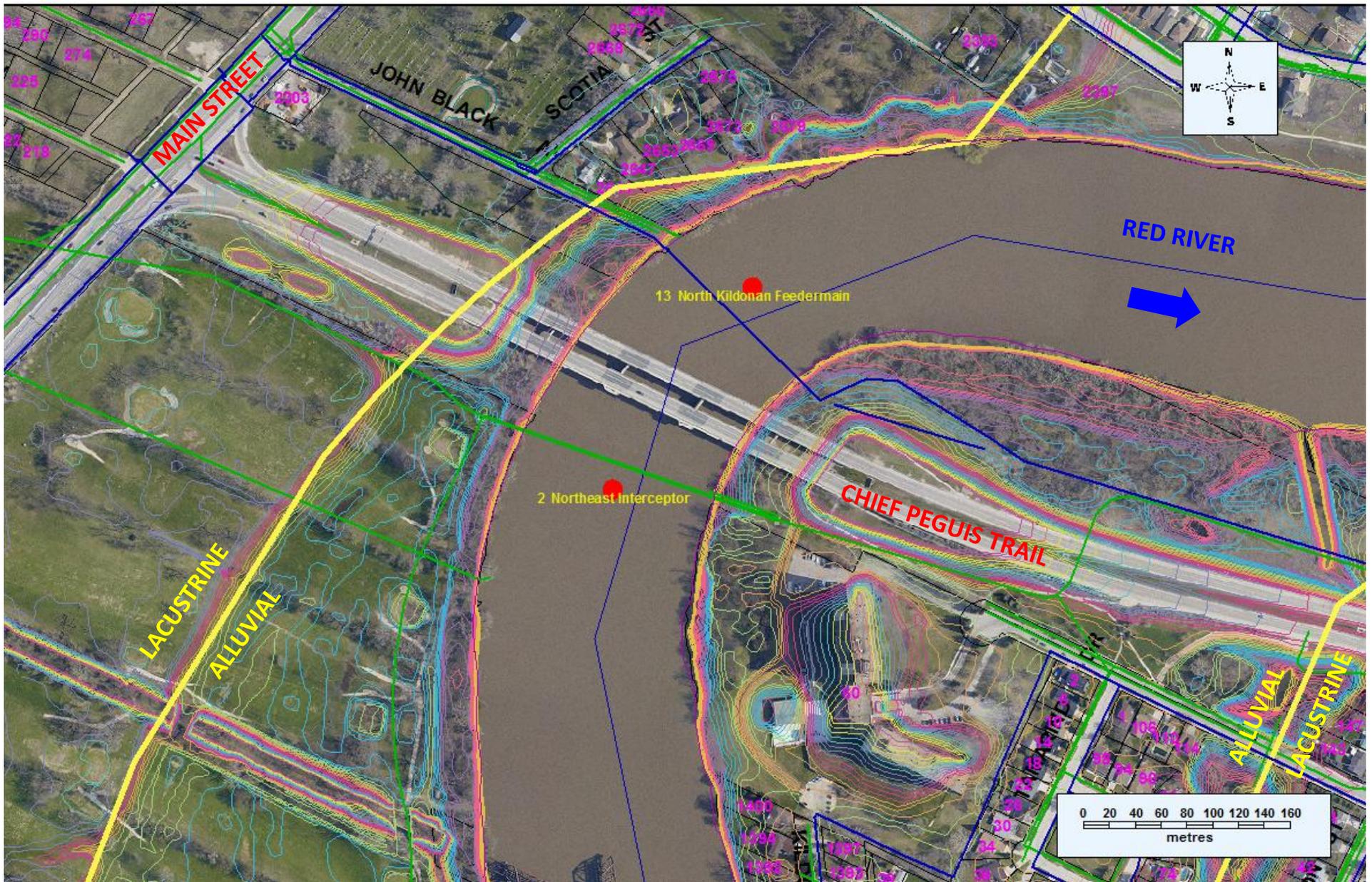
Figure 25 – Fresh slump block toppled between June 26 and August 15, 2012 indicative of active failure pattern.

**APPENDIX A
SITE LOCATIONS**

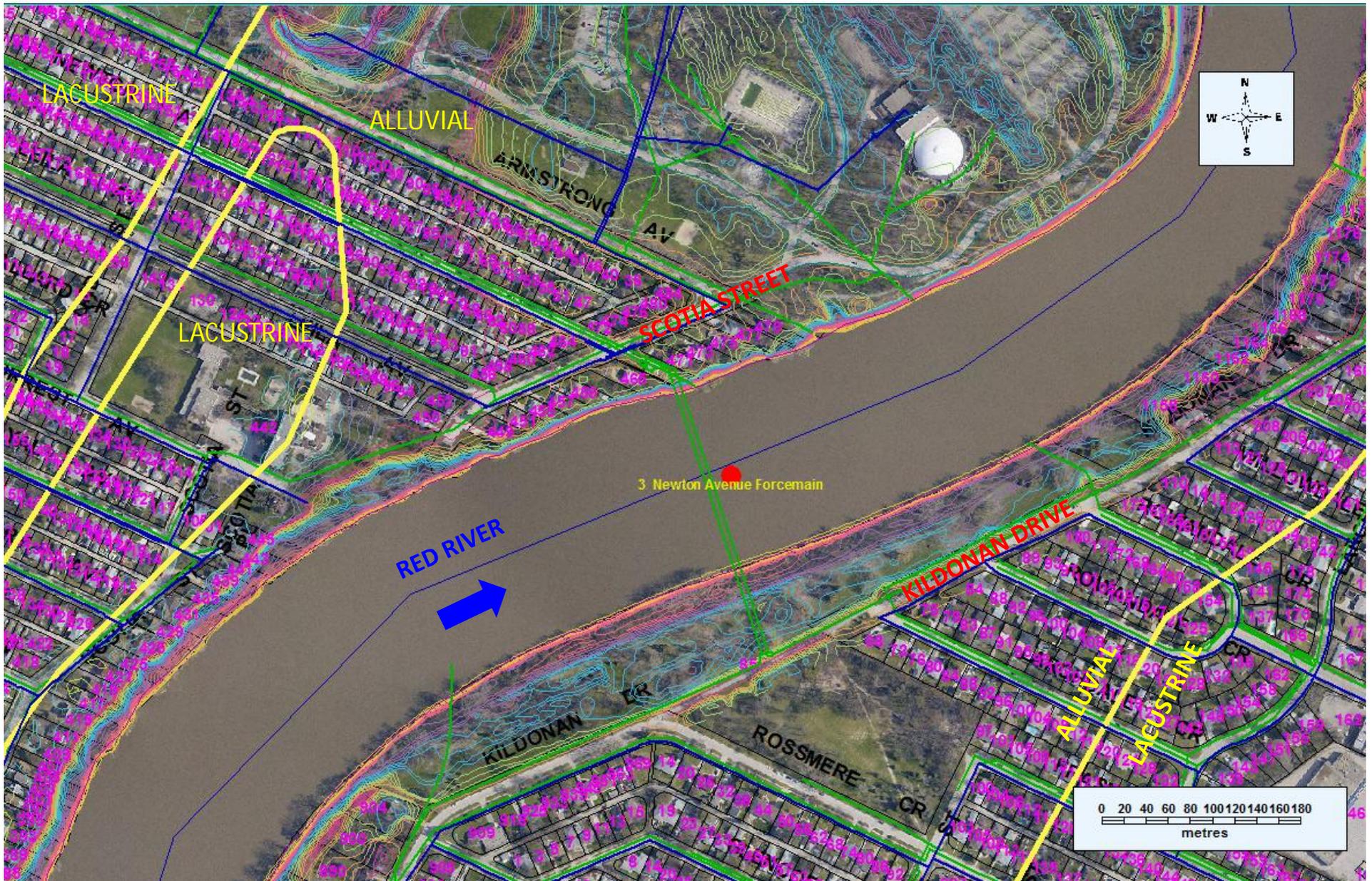




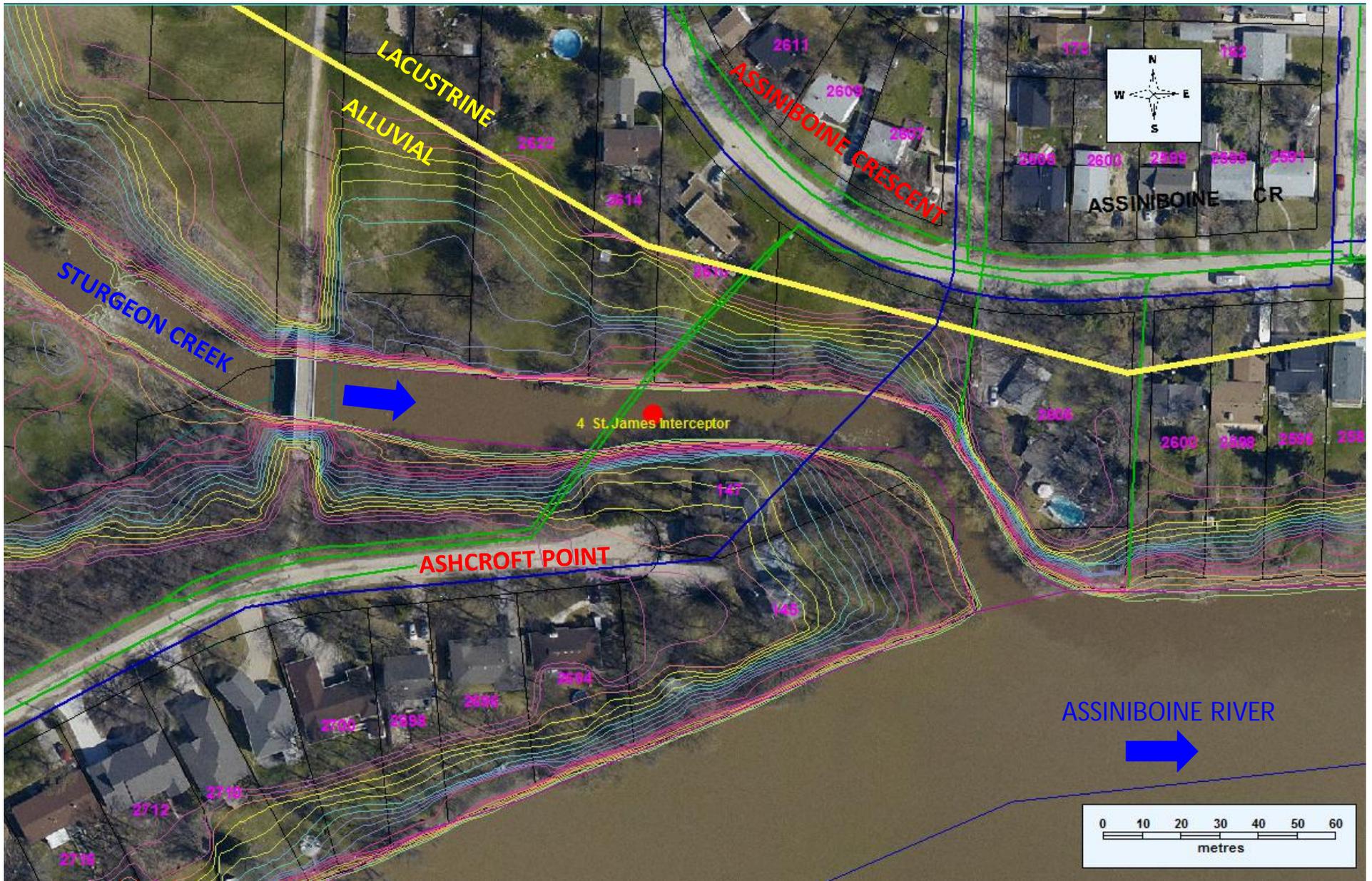
SITE 1: ST. JAMES INTERCEPTOR



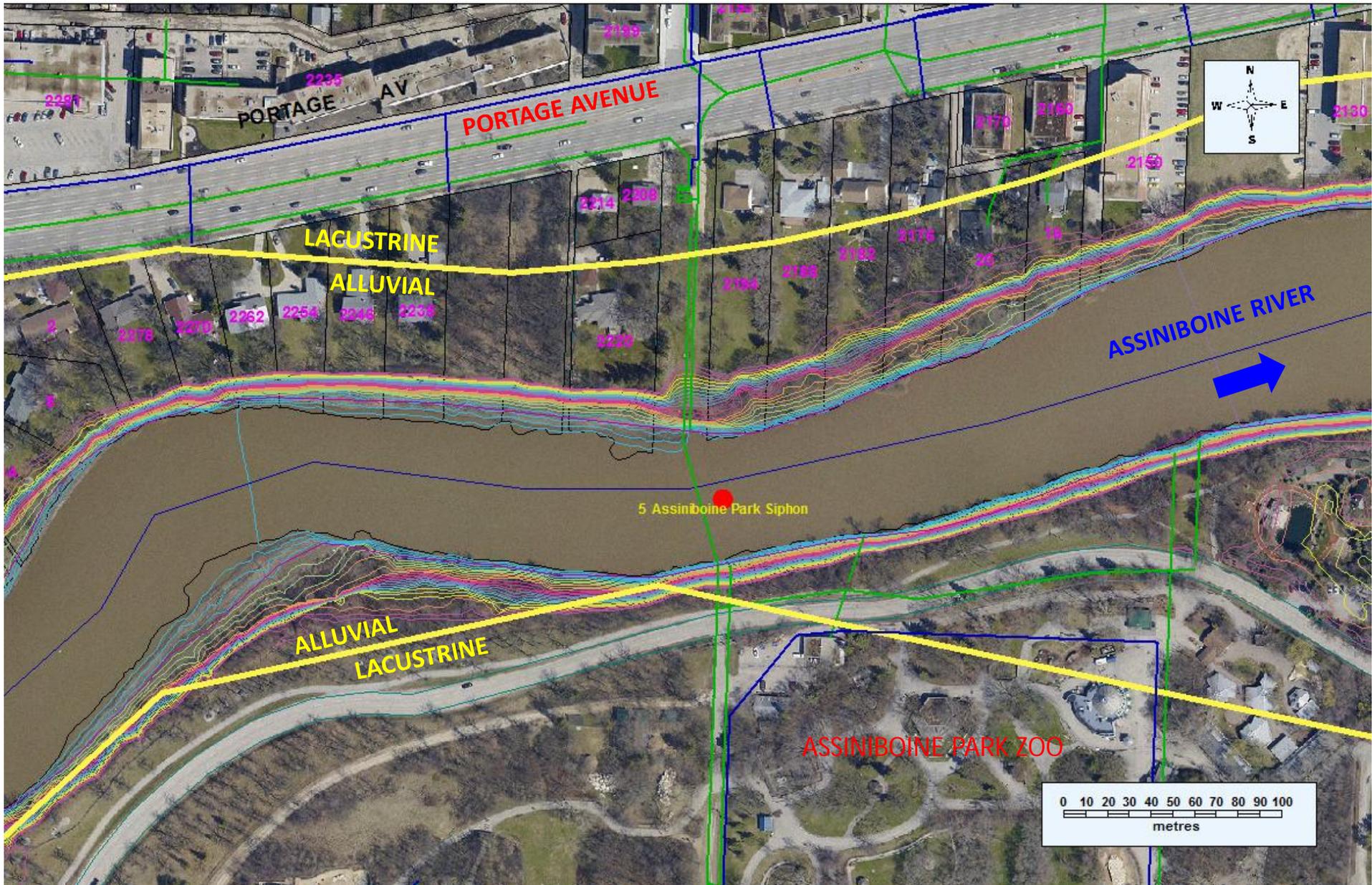
SITE 2: NORTHEAST INTERCEPTOR
SITE 13: NORTH KILDONAN FEEDERMAIN



SITE 3: NEWTON AVENUE FORCEMAIN



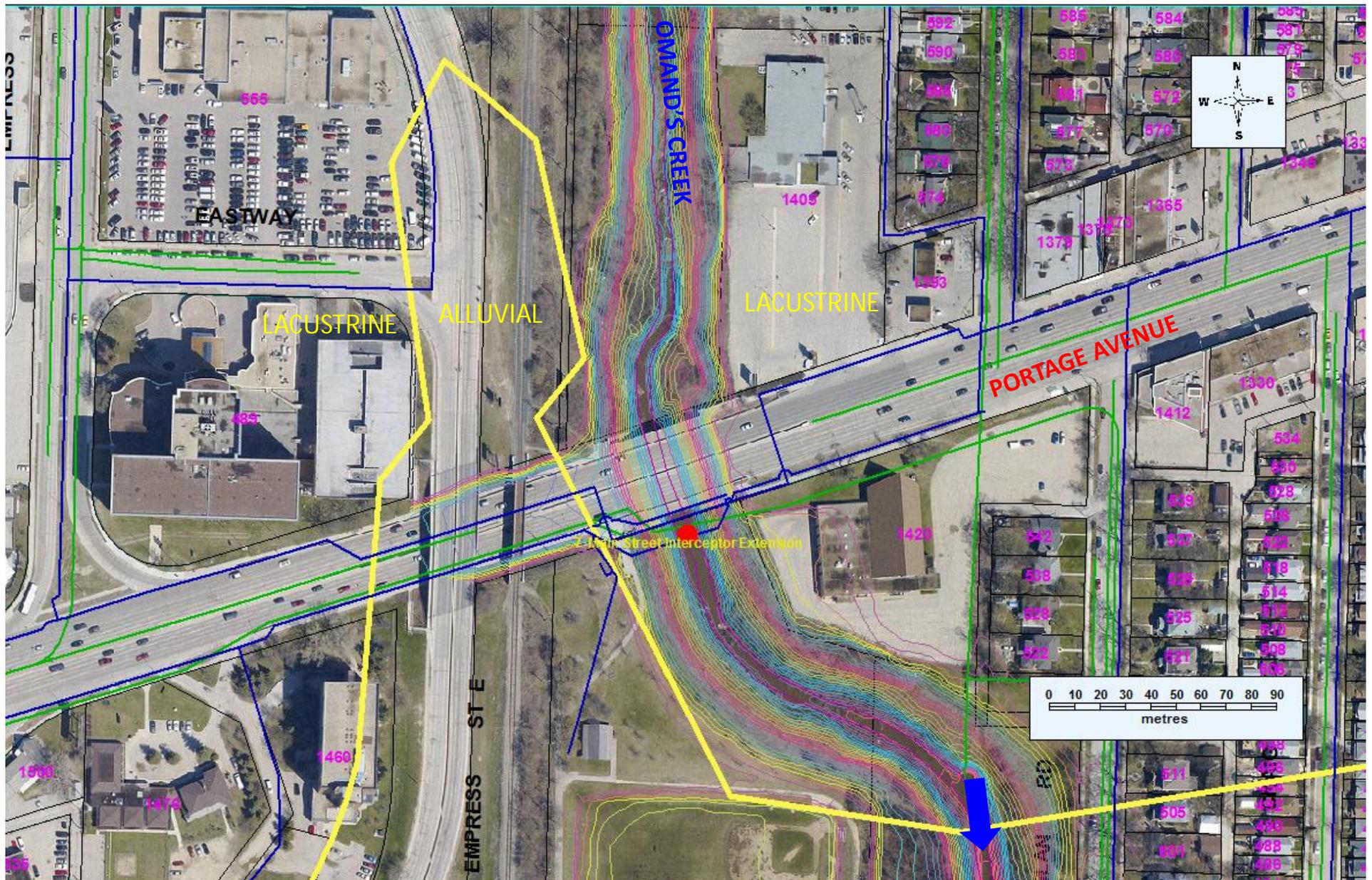
SITE 4: ST. JAMES INTERCEPTOR



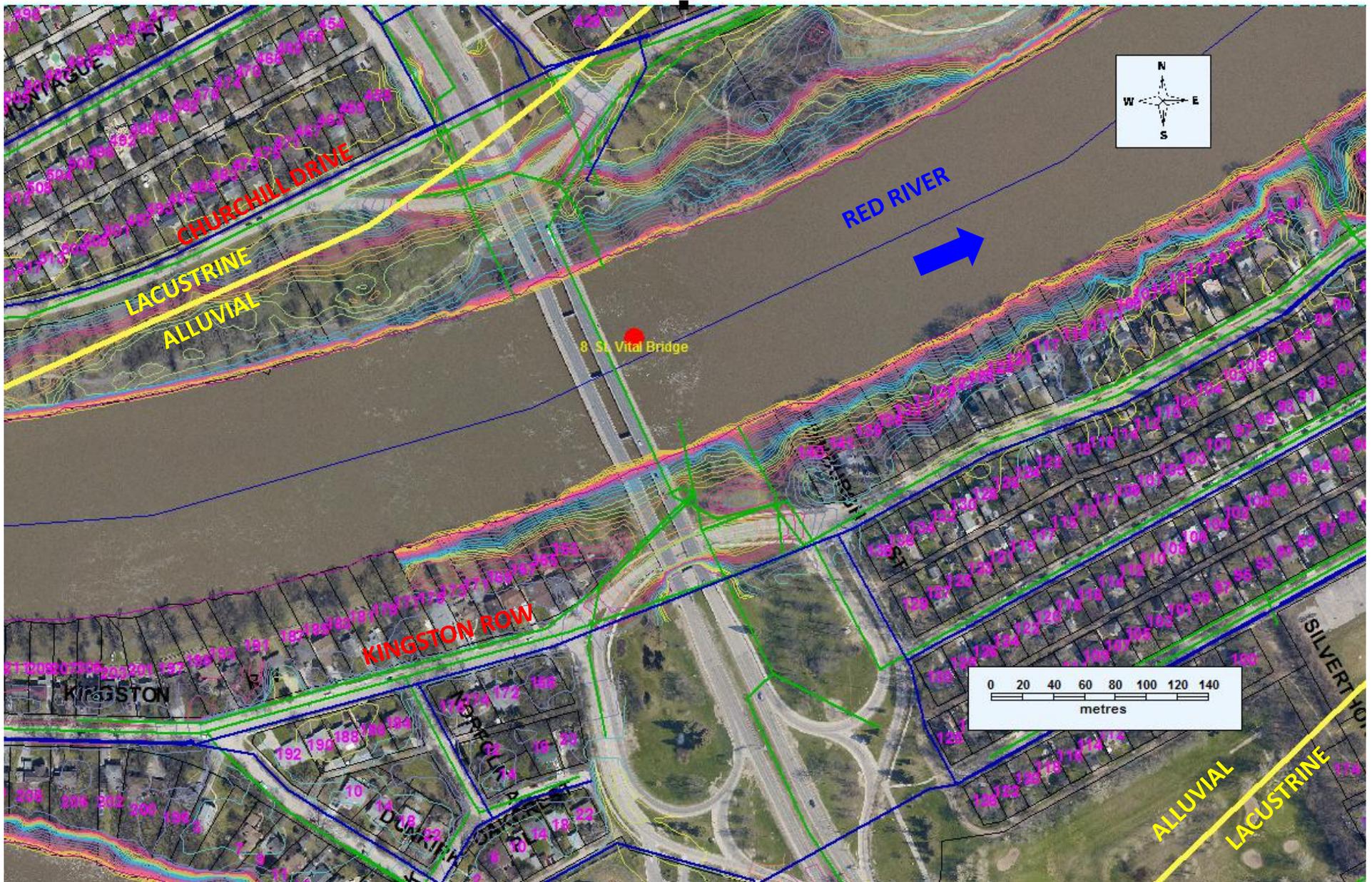
SITE 5: ASSINIBOINE PARK SIPHON



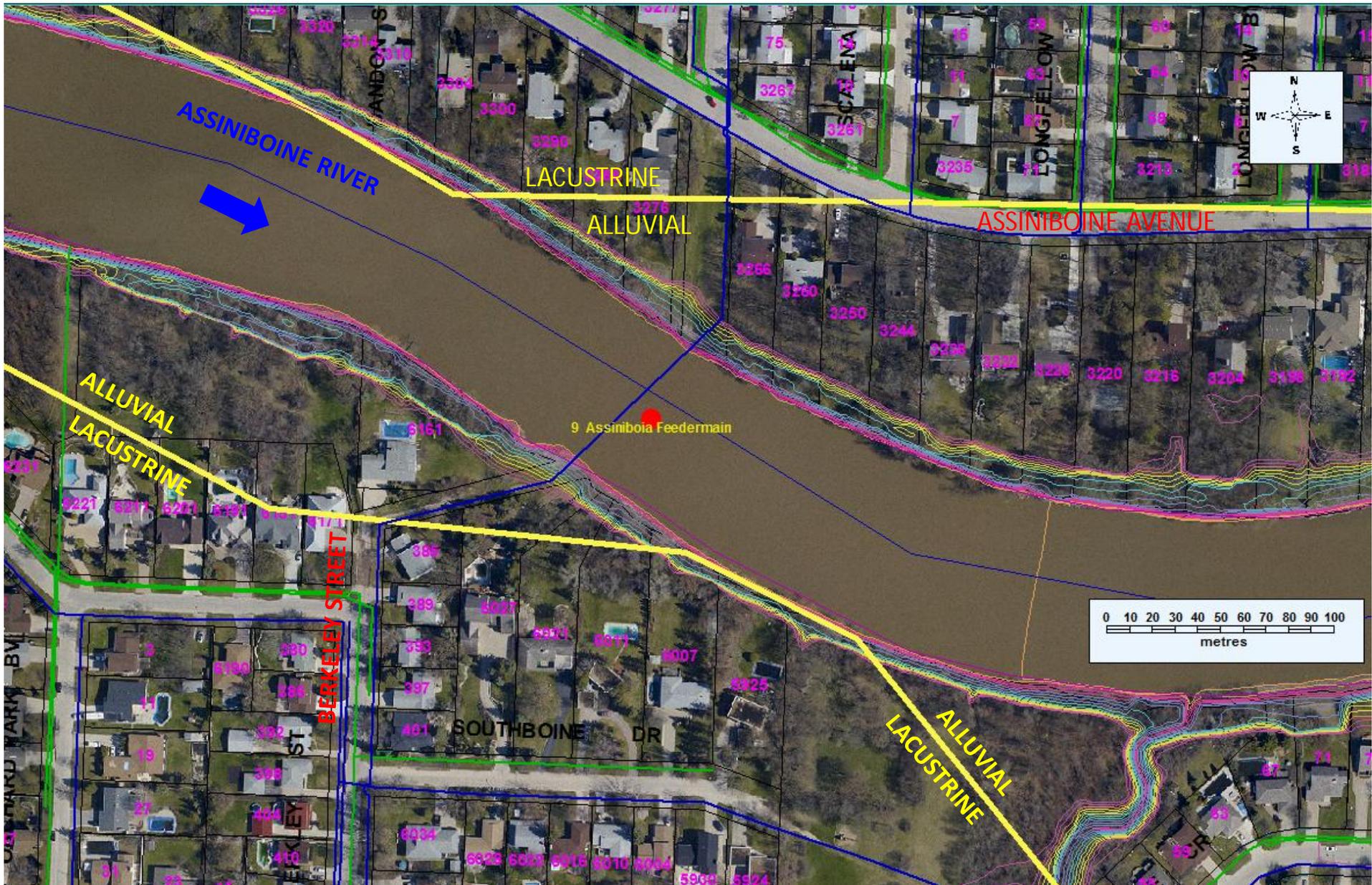
SITE 6: MUNROE POLSON SIPHON



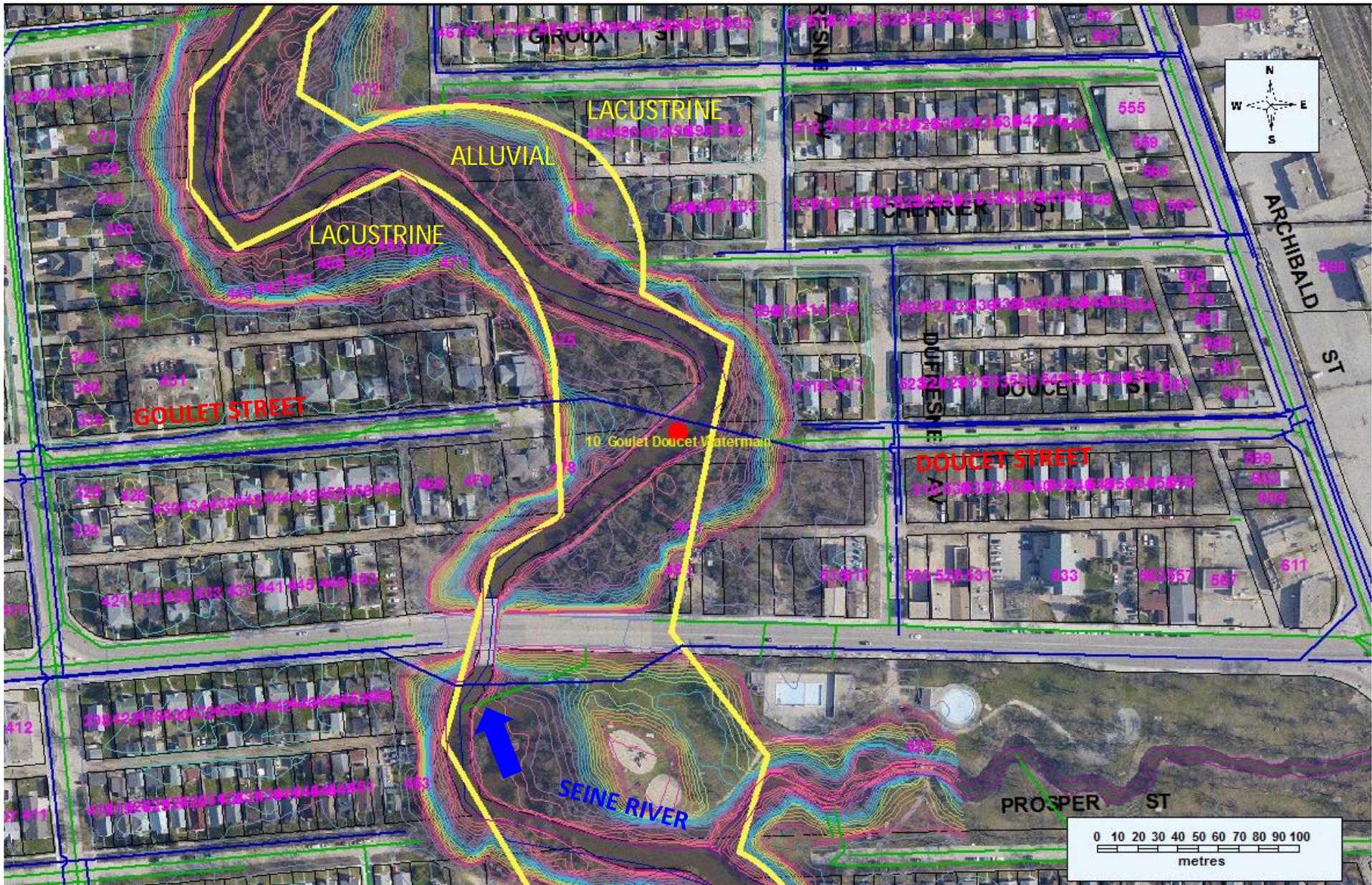
SITE 7: MAIN STREET INTERCEPTOR EXTENSION



SITE 8: ST. VITAL BRIDGE



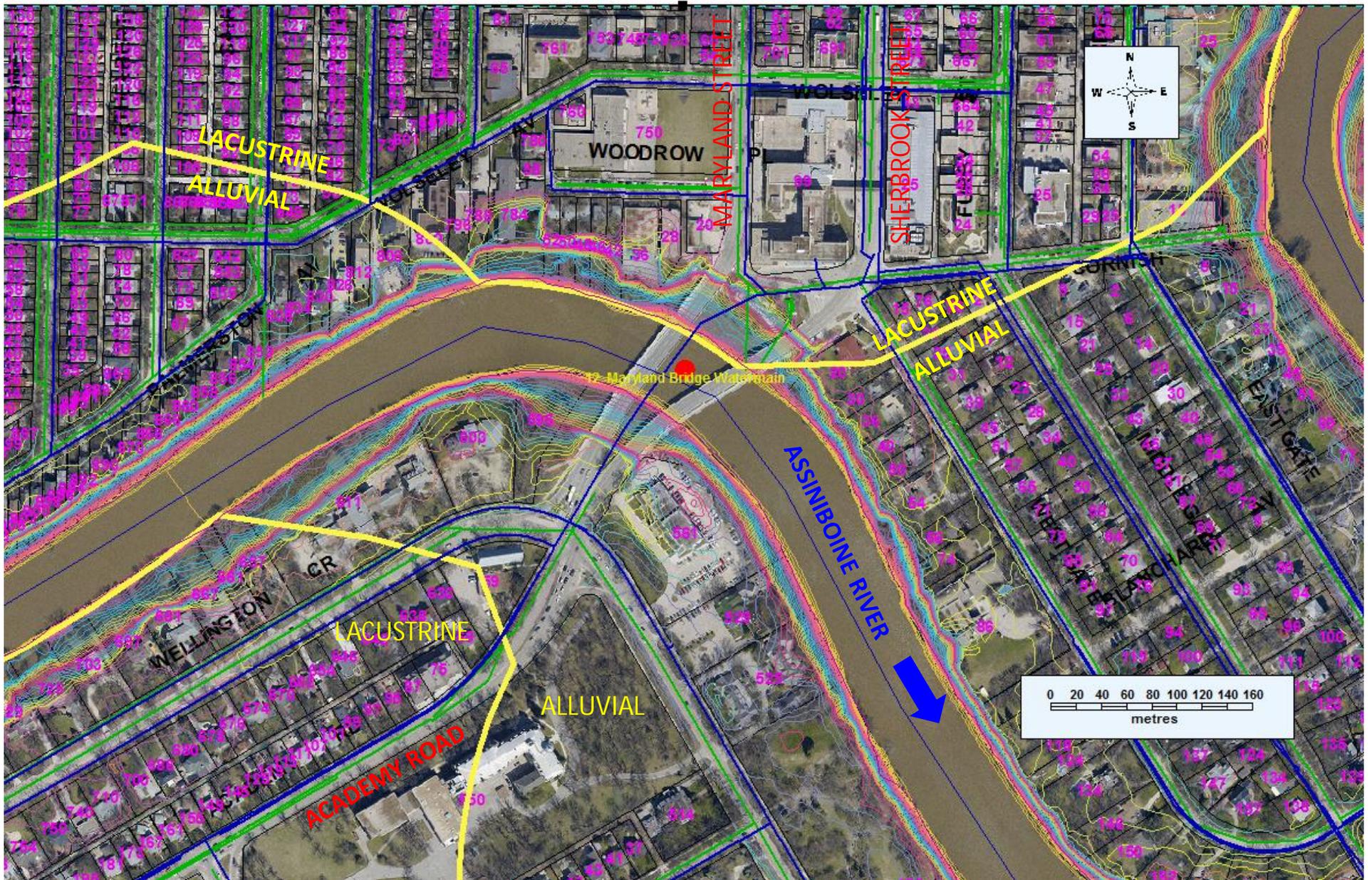
SITE 9: ASSINIBOIA FEEDERMAIN



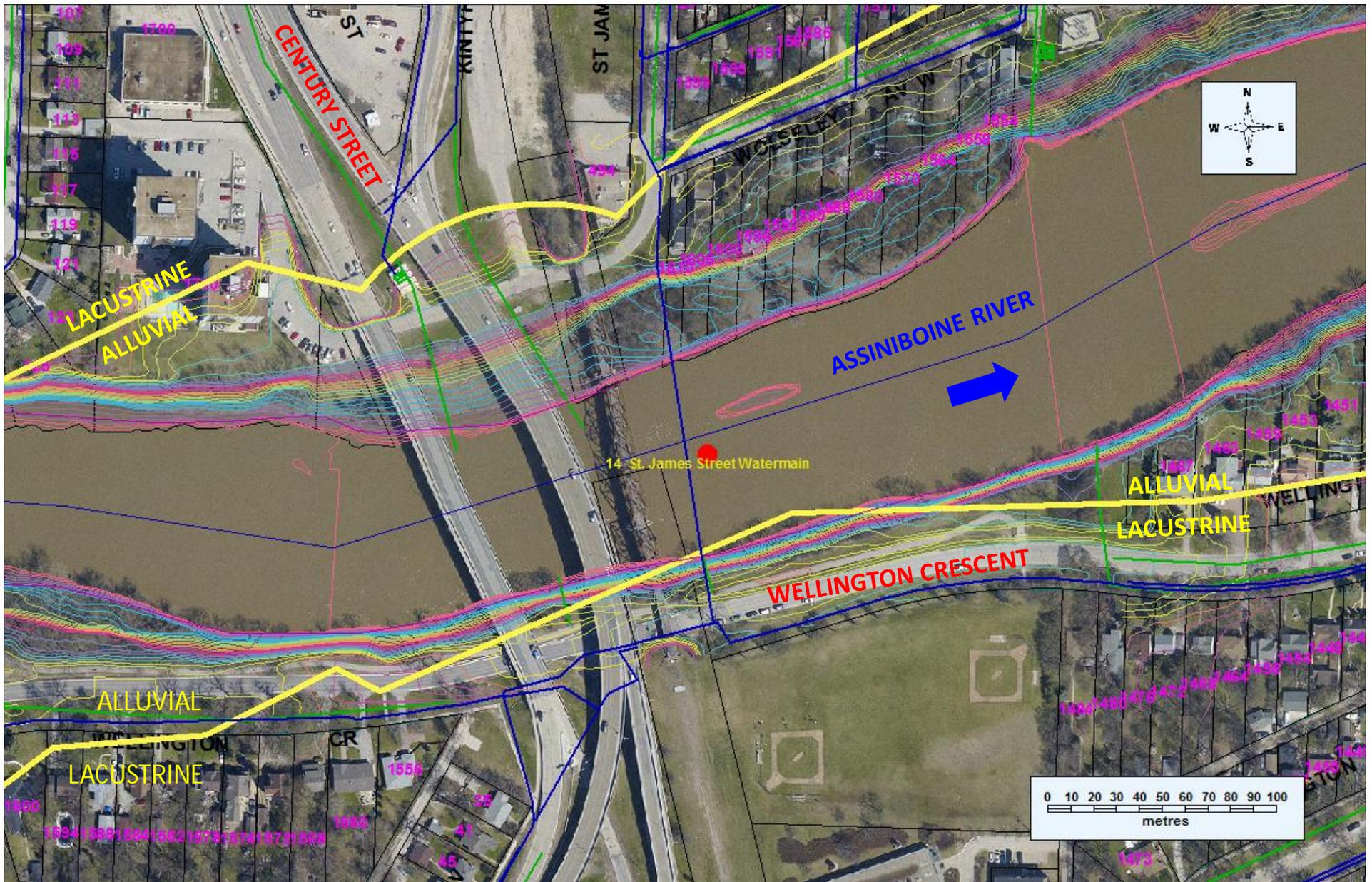
SITE 10: GOULET DOUCET WATERMAIN



SITE 11: KILDONAN REDWOOD FEEDERMAIN



SITE 12: MARYLAND BRIDGE WATERMAIN



SITE 14: ST. JAMES STREET WATERMAIN

**APPENDIX B
RIVER BANK CONDITION
SUMMARY**

**APPENDIX C
PHOTO LISTING**

PHOTOGRAPH DESCRIPTIONS
GEOTECHNICAL SITE INSPECTIONS

City of Winnipeg
 Condition Assessment of High Risk Water and Wastewater River Crossings
 Project number: 60270487

PHOTO NUMBER	SITE NUMBER	SITE NAME	BANK SIDE	DIRECTION FACING	PHOTOGRAPH DESCRIPTION	DATE
1	1	St. James Interceptor	N	N	Street at ROW	June 22, 2012
2	1	St. James Interceptor	N	S	Street at ROW	June 22, 2012
3	1	St. James Interceptor	N	S	Upper bank at ROW	June 22, 2012
4	1	St. James Interceptor	N	S	Mid bank at ROW	June 22, 2012
5	1	St. James Interceptor	N	N	Steepened bank at chambers	June 22, 2012
6	1	St. James Interceptor	N	E	Lower bank - lawn	June 22, 2012
7	1	St. James Interceptor	N	W	Lower bank - lawn and pool	June 22, 2012
8	1	St. James Interceptor	N	W	Bank toe - garden boxes	June 22, 2012
9	1	St. James Interceptor	N	E	Bank toe - mounds	June 22, 2012
10	1	St. James Interceptor	N	E	Toe at outfall	June 22, 2012
11	1	St. James Interceptor	N	W	Toe at outfall	June 22, 2012
12	1	St. James Interceptor	N	NW	Lower bank at first property east of ROW	June 22, 2012
13	1	St. James Interceptor	N	NE	Third and fourth properties east of ROW. Natural condition of landscape. Past erosion, hummocky.	June 22, 2012
14 to 21	1	St. James Interceptor	N	S	Panoramic view of south bank.	June 22, 2012
22 to 24	1	St. James Interceptor	N	N	Second and fourth properties west of ROW. Slight rise at mid bank.	June 22, 2012
25	1	St. James Interceptor	N	N	Photo taken from outfall facing toward road	June 22, 2012
26	1	St. James Interceptor	N	W	River edge	June 22, 2012
27	1	St. James Interceptor	N	E	River edge	June 22, 2012
28	1	St. James Interceptor	S	N	From street along ROW	June 22, 2012
29	1	St. James Interceptor	S	S	From ROW to street	June 22, 2012
30	1	St. James Interceptor	S	S	From ROW along alignment	June 22, 2012
31	1	St. James Interceptor	S	N	Pipe alignment to north shore through house corner	June 22, 2012
32	1	St. James Interceptor	S	N	ROW showing two manhole covers	June 22, 2012
33 to 39	1	St. James Interceptor	S	N	Mid bank	June 22, 2012
40	1	St. James Interceptor	S	E	Mid bank east of ROW at bench	June 22, 2012
41	1	St. James Interceptor	S	W	Two properties east of ROW on bench	June 22, 2012
42	1	St. James Interceptor	S	W	Third property west of ROW. Pronounced bench present.	June 22, 2012
43	1	St. James Interceptor	S	E	Second property west. Erosion and scarp at toe that becomes less pronounced towards ROW	June 22, 2012
44	1	St. James Interceptor	S	E	Fill placed on first property west of ROW	June 22, 2012
45	1	St. James Interceptor	S	E	First property west. Edge of fill at bank toe.	June 22, 2012
46	1	St. James Interceptor	S	W	First property west. Edge of fill at bank toe.	June 22, 2012
47	1	St. James Interceptor	S	E	First property west. Edge of fill at bank toe.	June 22, 2012
48	1	St. James Interceptor	S	E	First property west. Edge of fill at bank toe.	June 22, 2012
49	1	St. James Interceptor	S	W	Minor erosion at toe with about 0.6m high scarp. Flat slope to water edge.	June 22, 2012
50 to 59	1	St. James Interceptor	S	N	Panoramic view of north bank	June 22, 2012
60	1	St. James Interceptor	S	S	First property east of ROW	June 22, 2012
61	1	St. James Interceptor	S	S	Second property east of ROW	June 22, 2012
62	1	St. James Interceptor	S	E	Second property east of ROW with steep bank and gully	June 22, 2012
63	1	St. James Interceptor	S	W	Second property east of ROW with steep bank	June 22, 2012
64	1	St. James Interceptor	S	N	North bank at ROW	June 22, 2012
65	2	Northeast Interceptor	W	E	Potential slope inclinometer casing north of bridge and downslope of path	June 23, 2012
66	2	Northeast Interceptor	W	S	Three potential slope inclinometer casings. Two on abutment apron and one at toe of apron between the two bridges.	June 23, 2012
67	2	Northeast Interceptor	W	E	Settlement of sidewalk pads at abutment wall at north bridge	June 23, 2012
68	2	Northeast Interceptor	W	E	Settlement of sidewalk pads at abutment wall at south bridge	June 23, 2012
69	2	Northeast Interceptor	W	E	Potential slope inclinometer casings	June 23, 2012
70, 71	2	Northeast Interceptor	W	S	Cracks on abutment apron	June 23, 2012

PHOTOGRAPH DESCRIPTIONS
GEOTECHNICAL SITE INSPECTIONS

PHOTO NUMBER	SITE NUMBER	SITE NAME	BANK SIDE	DIRECTION FACING	PHOTOGRAPH DESCRIPTION	DATE
72	2	Northeast Interceptor	W	S	Apron settlement at abutment wall	June 23, 2012
73 to 76	2	Northeast Interceptor	W	W	ROW in golf course	June 23, 2012
77 to 79	2	Northeast Interceptor	W	S	Large trees along ROW	June 23, 2012
80	2	Northeast Interceptor	W	W	Along ROW	June 23, 2012
81	2	Northeast Interceptor	W	S	Hydro pole and large tree south of ROW	June 23, 2012
82 to 84	2	Northeast Interceptor	W	N	Hydro poles and large trees supporting lines that cross the river	June 23, 2012
85 to 89	2	Northeast Interceptor	W	N	Potential slope inclinometer casing north of fence and south of bridge	June 23, 2012
90	2	Northeast Interceptor	W	S	Photo taken from slope inclinometer casing facing toward ROW	June 23, 2012
91	2	Northeast Interceptor	W	S	Leaning trees south of ROW	June 23, 2012
92	2	Northeast Interceptor	W	N	Leaning trees	June 23, 2012
93, 94	2	Northeast Interceptor	W	N	Hydro poles and berm of bike path	June 23, 2012
95	2	Northeast Interceptor	W	W	Photo taken from river edge on ROW	June 23, 2012
96	2	Northeast Interceptor	W	N	River edge north of ROW	June 23, 2012
97	2	Northeast Interceptor	W	S	River edge south of ROW	June 23, 2012
98	2	Northeast Interceptor	W	S	Scarps on ROW	June 23, 2012
99	2	Northeast Interceptor	W	N	River edge looking toward ROW	June 23, 2012
100	2	Northeast Interceptor	W	S	Scarp south of ROW	June 23, 2012
101 to 104	2	Northeast Interceptor	W	N	Scarp south of ROW	June 23, 2012
105 to 111	2	Northeast Interceptor	W	E	Panoramic view of east bank for Site 2	June 23, 2012
112	2	Northeast Interceptor	W	W	Upslope between bridges	June 23, 2012
113 to 128	2	Northeast Interceptor	E	W	Panoramic view of west bank of Site 2	June 23, 2012
129	2	Northeast Interceptor	E	S	Edge of bank from ROW. Rip rap in ROW ends immediately upstream of ROW.	June 23, 2012
130, 131	2	Northeast Interceptor	E	N	Rip rap at edge of bank looking toward bridge from ROW. Outfall pipe present.	June 23, 2012
132	2	Northeast Interceptor	E	S	Erosion upstream of ROW	June 23, 2012
133	2	Northeast Interceptor	E	N	Erosion upstream of ROW with 1.8m-high head scarp	June 23, 2012
134	2	Northeast Interceptor	E	W	Toe along ROW. Photo taken from top of bank (head scarp)	June 23, 2012
135, 136	2	Northeast Interceptor	E	E	Along ROW from top of bank	June 23, 2012
137	2	Northeast Interceptor	E	W	Along ROW at valve chamber	June 23, 2012
138 to 140	2	Northeast Interceptor	E	N	Hydro poles supporting lines that cross the river	June 23, 2012
141	2	Northeast Interceptor	E	W	Effluent on west bank upstream of ROW	June 23, 2012
142 to 144	2	Northeast Interceptor	E	E	Head scarp upstream (south) of ROW, about 1.5m high.	June 23, 2012
145 to 147	2	Northeast Interceptor	E	S	Top of head scarp south of ROW	June 23, 2012
148	2	Northeast Interceptor	E	N	Top of head scarp	June 23, 2012
149	2	Northeast Interceptor	E	N	Approximately 3m vertical bank at river edge	June 23, 2012
150	2	Northeast Interceptor	E	E	Lower bank scarp	June 23, 2012
151	2	Northeast Interceptor	E	S	Toe of bank facing toward ROW	June 23, 2012
152	3	Newton Avenue Forcemain	W	W	Street view at station on pipe alignment	June 27, 2012
153, 154	3	Newton Avenue Forcemain	W	E	Street view at station on pipe alignment	June 27, 2012
155	3	Newton Avenue Forcemain	W	SE	Street view at station on pipe alignment	June 27, 2012
156	3	Newton Avenue Forcemain	W	NE	Street view at station on pipe alignment	June 27, 2012
157	3	Newton Avenue Forcemain	W	E	North side of station	June 27, 2012
158	3	Newton Avenue Forcemain	W	W	North side of station	June 27, 2012
159, 160	3	Newton Avenue Forcemain	W	E	Upper bank downslope of station	June 27, 2012
161 to 164	3	Newton Avenue Forcemain	W	W	Upper bank downslope of station	June 27, 2012
165	3	Newton Avenue Forcemain	W	S	Toe of bank at river edge. Rip rap is concrete rubble.	June 27, 2012
166	3	Newton Avenue Forcemain	W	N	Toe of bank at river edge. Rip rap is concrete rubble.	June 27, 2012
167 to 178	3	Newton Avenue Forcemain	W		Outfall pipe	June 27, 2012
179	3	Newton Avenue Forcemain	W	N	Rip rap north of outfall	June 27, 2012
180	3	Newton Avenue Forcemain	W	S	Outfall pipe	June 27, 2012

PHOTOGRAPH DESCRIPTIONS
GEOTECHNICAL SITE INSPECTIONS

PHOTO NUMBER	SITE NUMBER	SITE NAME	BANK SIDE	DIRECTION FACING	PHOTOGRAPH DESCRIPTION	DATE
181	3	Newton Avenue Forcemain	W	N	Photo taken from first property north facing toward second property north	June 27, 2012
182	3	Newton Avenue Forcemain	W	N	Wall at toe at second property north of ROW	June 27, 2012
183	3	Newton Avenue Forcemain	W	S	First property north taken from second property north	June 27, 2012
184	3	Newton Avenue Forcemain	W	N	Wall at second property north	June 27, 2012
185 to 187	3	Newton Avenue Forcemain	W	W	Stairs at second property in good condition	June 27, 2012
188	3	Newton Avenue Forcemain	W	E	Stairs at second property in good condition	June 27, 2012
189 to 196	3	Newton Avenue Forcemain	W	S	Upper bank at first property north of station	June 27, 2012
197 to 200	3	Newton Avenue Forcemain	W	E	Lower bank at first property north of station	June 27, 2012
201 to 206	3	Newton Avenue Forcemain	W	S	Upper bank at first property south of station	June 27, 2012
207 to 209	3	Newton Avenue Forcemain	W	N	Upper bank at first property south of station	June 27, 2012
210	3	Newton Avenue Forcemain	W	N	Toe at river edge at first property south of station	June 27, 2012
211	3	Newton Avenue Forcemain	W	S	Toe at river edge at first property south of station	June 27, 2012
212	3	Newton Avenue Forcemain	W	E	East bank on alignment	June 27, 2012
213 to 233	3	Newton Avenue Forcemain	W	E	Panoramic view of east bank (Fraser's Grove Road)	June 27, 2012
234	3	Newton Avenue Forcemain	E	E	Photo taken from dike at path along pipe crossing alignment	June 23, 2012
235	3	Newton Avenue Forcemain	E	W	Photo taken from dike facing toward river along pipe crossing alignment	June 23, 2012
236	3	Newton Avenue Forcemain	E	N	Along dike at manholes	June 23, 2012
237	3	Newton Avenue Forcemain	E	S	Along dike at manholes	June 23, 2012
238, 239	3	Newton Avenue Forcemain	E	W	Along pipe crossing alignment	June 23, 2012
240	3	Newton Avenue Forcemain	E	E	Facing manhole along pipe crossing alignment	June 23, 2012
241	3	Newton Avenue Forcemain	E	E	River bank toe at river edge with some erosion and no significant scarps	June 23, 2012
242	3	Newton Avenue Forcemain	E	N	Photo taken from pipe crossing alignment facing downstream along bank toe	June 23, 2012
243	3	Newton Avenue Forcemain	E	S	Head scarp upstream (south of pipe crossing alignment)	June 23, 2012
244	3	Newton Avenue Forcemain	E	N	Facing toward pipe crossing alignment	June 23, 2012
245	3	Newton Avenue Forcemain	E	E	Erosion scarp facing south of pipe crossing alignment	June 23, 2012
246	3	Newton Avenue Forcemain	E	N	Upper bench facing toward pipe crossing alignment	June 23, 2012
247	3	Newton Avenue Forcemain	E	S	Upper bench facing toward pipe crossing alignment	June 23, 2012
248	3	Newton Avenue Forcemain	E	W	Lower slope	June 23, 2012
249	3	Newton Avenue Forcemain	E	E	Lower slope at pipe crossing alignment	June 23, 2012
250 to 265	3	Newton Avenue Forcemain	E	W	Panoramic view of west bank	June 25, 2012
266	4	St. James Interceptor	N	N	Chamber location	June 22, 2012
267	4	St. James Interceptor	N	W	ROW from street	June 22, 2012
268	4	St. James Interceptor	N	W	ROW from chamber doors	June 22, 2012
269, 270	4	St. James Interceptor	N	W	ROW - Lower bank to creek	June 22, 2012
271	4	St. James Interceptor	N	NE	ROW from lower bank	June 22, 2012
272, 273	4	St. James Interceptor	N	N	Upstream lower bank at toe	June 22, 2012
274	4	St. James Interceptor	N	E	Downstream lower bank at toe	June 22, 2012
275, 276	4	St. James Interceptor	N	N	Third property upstream of ROW	June 22, 2012
277	4	St. James Interceptor	N	E	Downstream bank toe	June 22, 2012
278	4	St. James Interceptor	N	E	ROW from bank toe	June 22, 2012
279	4	St. James Interceptor	N	N	ROW upstream from toe	June 22, 2012
280 to 283	4	St. James Interceptor	N	W	Panorama of south bank	June 22, 2012
284	4	St. James Interceptor	N	N	Upstream toe at creek edge	June 22, 2012
285, 286	4	St. James Interceptor	N	N	Mid bank plateau	June 22, 2012
287, 288	4	St. James Interceptor	N	W	Neighbouring feedermain crossing located under building	June 22, 2012
289	4	St. James Interceptor	S	N	Toward Assiniboine Crescent along ROW	June 22, 2012

PHOTOGRAPH DESCRIPTIONS
GEOTECHNICAL SITE INSPECTIONS

PHOTO NUMBER	SITE NUMBER	SITE NAME	BANK SIDE	DIRECTION FACING	PHOTOGRAPH DESCRIPTION	DATE
290 to 294	4	St. James Interceptor	S	N	Panoramic view of south north bank.	June 22, 2012
295 to 299	4	St. James Interceptor	S	N	Various locations between pedestrian bridge to crossing location	June 22, 2012
300	4	St. James Interceptor	S	E	Bank toe at crossing	June 22, 2012
301	4	St. James Interceptor	S	N	ROW from south shore facing north	June 22, 2012
302	4	St. James Interceptor	S	E	Downstream shore	June 22, 2012
303 to 305	4	St. James Interceptor	S	W	Upstream shore	June 22, 2012
306	4	St. James Interceptor	S	W	Pipe (approximately 4-inch diameter) protruding from bank with other debris at probable crossing location	June 22, 2012
307	4	St. James Interceptor	S	N	ROW: line is on ROW	June 22, 2012
308, 309	4	St. James Interceptor	S	W	Upstream at ROW toe	June 22, 2012
310	4	St. James Interceptor	S	E	Downstream of ROW toe	June 22, 2012
311	4	St. James Interceptor	S	W	Upstream of ROW. Metal strapping and other debris at toe	June 22, 2012
312 to 314	4	St. James Interceptor	S	N	Location of crossing at shore taken from road at Ashcroft Point	June 22, 2012
315	5	Assiniboine Park Siphon	N	N	ROW facing toward Portage Avenue from station	June 25, 2012
316	5	Assiniboine Park Siphon	N	S	ROW facing toward river from Portage Avenue	June 25, 2012
317, 318	5	Assiniboine Park Siphon	N	N	Photo take from test hole casing on ROW	June 25, 2012
319	5	Assiniboine Park Siphon	N	S	Test hole casing with photo taken from gate structure	June 25, 2012
320	5	Assiniboine Park Siphon	N	N	ROW, test hole casing, and gate structure	June 25, 2012
321	5	Assiniboine Park Siphon	N	S	Lower bank	June 25, 2012
322	5	Assiniboine Park Siphon	N	S	ROW alignment	June 25, 2012
323	5	Assiniboine Park Siphon	N	N	ROW with photo taken from lower bank	June 25, 2012
324 to 334	5	Assiniboine Park Siphon	N	S	Panoramic view of south bank.	June 25, 2012
335 to 337	5	Assiniboine Park Siphon	N	E	Mid bank on ROW	June 25, 2012
338	5	Assiniboine Park Siphon	N	W	First property east at mid bank	June 25, 2012
339 to 341	5	Assiniboine Park Siphon	N	W	Mid bank on ROW	June 25, 2012
342	5	Assiniboine Park Siphon	N	W	Bank toe with photo taken from ROW	June 25, 2012
343	5	Assiniboine Park Siphon	N	E	Bank toe with photo taken from ROW	June 25, 2012
344	5	Assiniboine Park Siphon	N	W	Photo taken from first property east	June 25, 2012
345	5	Assiniboine Park Siphon	N	E	Photo taken from first property east facing toward ROW and outfall	June 25, 2012
346, 347	5	Assiniboine Park Siphon	N	W	Sand on mid bank of first and second property east of ROW	June 25, 2012
348	5	Assiniboine Park Siphon	N	W	Steep bank on second property east of ROW with sand along shore	June 25, 2012
349	5	Assiniboine Park Siphon	N	E	Sand on mid bank	June 25, 2012
350	5	Assiniboine Park Siphon	N	N	Scarp at mid bank on third property east of ROW	June 25, 2012
351	5	Assiniboine Park Siphon	N	E	Photo taken from first property west of ROW facing toward ROW	June 25, 2012
352 to 356	5	Assiniboine Park Siphon	N	N	Steep lower bank west of ROW with some rubble and pit run fill on first property west of ROW	June 25, 2012
357	5	Assiniboine Park Siphon	N	E	Marsh grass at bank toe at river edge	June 25, 2012
358	5	Assiniboine Park Siphon	N	W	Marsh grass at bank toe at river edge	June 25, 2012
359	5	Assiniboine Park Siphon	N	W	Upper bank of first property west of ROW	June 25, 2012
360	5	Assiniboine Park Siphon	N	S	Outfall pipe at river edge	June 25, 2012
361	5	Assiniboine Park Siphon	S	S	ROW into Assiniboine Park Zoo	June 22, 2012
362, 363	5	Assiniboine Park Siphon	S	N	ROW toward river. Manhole in foreground on road.	June 22, 2012
364	5	Assiniboine Park Siphon	S	N	Manhole and gate cover.	June 22, 2012
365	5	Assiniboine Park Siphon	S	S	Manhole and gate cover facing toward road and into zoo.	June 22, 2012
366	5	Assiniboine Park Siphon	S	N	Approximately 4-inch steel casing	June 22, 2012
367	5	Assiniboine Park Siphon	S	N	Path to river on ROW	June 22, 2012

PHOTOGRAPH DESCRIPTIONS
GEOTECHNICAL SITE INSPECTIONS

PHOTO NUMBER	SITE NUMBER	SITE NAME	BANK SIDE	DIRECTION FACING	PHOTOGRAPH DESCRIPTION	DATE
368	5	Assiniboine Park Siphon	S	S	Path to street on ROW on bank	June 22, 2012
369 to 377	5	Assiniboine Park Siphon	S	N	Panoramic view of north bank	June 22, 2012
370 to 381	5	Assiniboine Park Siphon	S	S	Boulders in armored ditch leading to river. Water running through rocks.	June 22, 2012
382, 383	5	Assiniboine Park Siphon	S	W	Upstream along toe of bench. No erosion. Deposition occurring.	June 22, 2012
384	5	Assiniboine Park Siphon	S	W	Toe at crossing location facing upstream.	June 22, 2012
385	5	Assiniboine Park Siphon	S	W	Lower toe slump block about 5m west of crossing	June 22, 2012
386	6	Munroe Polson Siphon	W	E	Street location at station	June 27, 2012
387	6	Munroe Polson Siphon	W	W	Street location at station	June 27, 2012
388	6	Munroe Polson Siphon	W	N	Street location at station	June 27, 2012
389	6	Munroe Polson Siphon	W	S	Street location at station	June 27, 2012
390 to 393	6	Munroe Polson Siphon	W	W	Upper bank at station with potential slope inclinometer casing	June 27, 2012
394 to 397	6	Munroe Polson Siphon	W	E	Upper bank with photo taken from potential slope inclinometer casing	June 27, 2012
398 to 402	6	Munroe Polson Siphon	W	E	Upper bank with photo taken from station	June 27, 2012
403, 404	6	Munroe Polson Siphon	W	S	Lower bank	June 27, 2012
405	6	Munroe Polson Siphon	W	E	Outfall and toe of river bank	June 27, 2012
406	6	Munroe Polson Siphon	W	N	River bank toe facing downstream	June 27, 2012
407	6	Munroe Polson Siphon	W	S	River bank toe facing upstream	June 27, 2012
408	6	Munroe Polson Siphon	W	E	East bank at pipe crossing alignment	June 27, 2012
409 to 434	6	Munroe Polson Siphon	W	E	Panoramic view of east bank (Henderson Highway)	June 27, 2012
435	6	Munroe Polson Siphon	W	S	Toe at outfall with river bank recession. Photo taken from first property north of crossing.	June 27, 2012
436	6	Munroe Polson Siphon	W	N	Toe at first and second properties north of crossing	June 27, 2012
437	6	Munroe Polson Siphon	W	S	Toe erosion at first property north of crossing. Photo taken from second property north of crossing.	June 27, 2012
438	6	Munroe Polson Siphon	W	N	Concrete rubble used as rip rap at toe north of pipe crossing	June 27, 2012
439	6	Munroe Polson Siphon	W	S	Recession of river bank at grouted rip rap	June 27, 2012
440	6	Munroe Polson Siphon	W	N	Recession of river bank at grouted rip rap	June 27, 2012
441	6	Munroe Polson Siphon	W	S	Photo taken from first property south of pipe crossing. Toe armor present at first property south of crossing with no armor at properties farther south.	June 27, 2012
442	6	Munroe Polson Siphon	W	N	Armored toe at first property south of station	June 27, 2012
443 to 449	6	Munroe Polson Siphon	W	E	Photo taken from third property south of station showing east bank instabilities	June 27, 2012
450, 451	6	Munroe Polson Siphon	W	N	River bank toe with leaning tree at second and third properties south of station	June 27, 2012
452 to 461	6	Munroe Polson Siphon	W	N	Upper bank at first property north of station	June 27, 2012
462 to 472	6	Munroe Polson Siphon	W	S	Upper bank at first property south of station	June 27, 2012
473 to 475	6	Munroe Polson Siphon	E	E	Along ROW facing toward Henderson Highway	June 26, 2012
476 to 486	6	Munroe Polson Siphon	E	W	ROW and neighbouring property backyard immediately south of ROW	June 26, 2012
487 to 506	6	Munroe Polson Siphon	E	W	Panoramic view of west bank	June 26, 2012
507 to 512	6	Munroe Polson Siphon	E	S	River bank instabilities at second and third properties south	June 26, 2012
513	6	Munroe Polson Siphon	E	N	Rip at toe on ROW	June 26, 2012
514	6	Munroe Polson Siphon	E	N	Bank on ROW	June 26, 2012
517	6	Munroe Polson Siphon	E	N	Outfall pipe	June 26, 2012
518 to 523	6	Munroe Polson Siphon	E	E	Grouted rip rap around outfall with cracking at north side. No cracking on south side.	June 26, 2012
524	6	Munroe Polson Siphon	E	N	Rip rap at toe north of outfall	June 26, 2012

PHOTOGRAPH DESCRIPTIONS
GEOTECHNICAL SITE INSPECTIONS

PHOTO NUMBER	SITE NUMBER	SITE NAME	BANK SIDE	DIRECTION FACING	PHOTOGRAPH DESCRIPTION	DATE
525	6	Munroe Polson Siphon	E	S	Rip rap at toe north of outfall	June 26, 2012
526 to 529	6	Munroe Polson Siphon	E	E	First property north of ROW	June 26, 2012
530	6	Munroe Polson Siphon	E	E	Photo taken from outfall facing upslope toward valve chamber	June 26, 2012
531 to 534	6	Munroe Polson Siphon	E	SE	River bank on ROW	June 26, 2012
535 to 539	6	Munroe Polson Siphon	E	N	River bank on ROW with fill placed at top of bank	June 26, 2012
540 to 546	6	Munroe Polson Siphon	E	N	Fill placed at top of slope	June 26, 2012
547 to 552	6	Munroe Polson Siphon	E		Sono tube from south to north with contents of crushed limestone. No concrete or reinforcing steel present.	June 26, 2012
553 to 556	6	Munroe Polson Siphon	E	NE	Pad of granular fill	June 26, 2012
557	6	Munroe Polson Siphon	E	SE	Edge of fill	June 26, 2012
558	6	Munroe Polson Siphon	E	N	ROW at mid bank downslope of chamber looking onto first property north	June 26, 2012
1231	6	Munroe Polson Siphon	E	W	Gravel piles at top of bank on ROW	August 15, 2012
1232	6	Munroe Polson Siphon	E	SW	Backyard of first property south of ROW. Concrete pad cast left of loader.	August 15, 2012
1233	6	Munroe Polson Siphon	E	E	Along ROW	August 15, 2012
1234	6	Munroe Polson Siphon	E	SE	Edge of fill at top of bank dressed with field stones and topsoil with concrete grade beam at toe. Sono tubes contain concrete and reinforcing steel.	August 15, 2012
1235	6	Munroe Polson Siphon	E	N	Edge of fill at top of bank dressed with field stones and topsoil with concrete grade beam at toe. Sono tubes contain concrete and reinforcing steel.	August 15, 2012
1236	6	Munroe Polson Siphon	E	E	Sono tube containing concrete and reinforcing steel	August 15, 2012
559 to 561	7	Main Street Interceptor Extension	W	S	Armoring on west bank and centreline of channel	June 25, 2012
562 to 568	7	Main Street Interceptor Extension	W	E	Panoramic view of east bank	June 25, 2012
569	7	Main Street Interceptor Extension	W	N	Armor on west bank	June 25, 2012
570, 571	7	Main Street Interceptor Extension	W	E	Armor on east bank	June 25, 2012
572 to 574	7	Main Street Interceptor Extension	E	W	Panoramic view of west bank	June 25, 2012
575 to 579	7	Main Street Interceptor Extension	E	W	Panoramic closer view of west bank	June 25, 2012
580, 581	7	Main Street Interceptor Extension	E	S	East bank vegetation	June 25, 2012
582, 583	7	Main Street Interceptor Extension	E	N	Tri-lock blocks armoring at channel edge	June 25, 2012
584	7	Main Street Interceptor Extension	E	N	Grouted limestone border at channel edge	June 25, 2012
585 to 587	7	Main Street Interceptor Extension	E	N	Bridge structure concrete	June 25, 2012
588 to 590	7	Main Street Interceptor Extension	E	S	Armoring on east bank	June 25, 2012
591	8	St. Vital Bridge	N	NE	Forcemain alignment	June 25, 2012
592	8	St. Vital Bridge	N	S	Magmeter enclosure	June 25, 2012
593	8	St. Vital Bridge	N	S	Forcemain elbow mounted on bridge	June 25, 2012
594	8	St. Vital Bridge	N	N	Forcemain elbow mounted on bridge	June 25, 2012
595	8	St. Vital Bridge	N	S	Forcemain on bridge	June 25, 2012
596	8	St. Vital Bridge	N	N	Forcemain on bridge	June 25, 2012
597	8	St. Vital Bridge	N	W	Rip rap armor at toe. Grouted rip rap drain channels disconnected from main channel.	June 25, 2012
598	8	St. Vital Bridge	N	N	East drain channel	June 25, 2012
599	8	St. Vital Bridge	N	E	East drain channel	June 25, 2012
600	8	St. Vital Bridge	N	W	West drain channel	June 25, 2012
601	8	St. Vital Bridge	N	N	West drain channel	June 25, 2012
602	8	St. Vital Bridge	N	E	Rip rap armor	June 25, 2012
603 to 605	8	St. Vital Bridge	N	E	Mid bank between east bridge piers	June 25, 2012
606 to 613, 615	8	St. Vital Bridge	N	S	Panoramic view of south bank.	June 25, 2012
614	8	St. Vital Bridge	N	E	Steep bank east of rip rap	June 25, 2012
616 to 625	8	St. Vital Bridge	S	N	Panoramic view of north bank	June 25, 2012
626	8	St. Vital Bridge	S	W	Forcemain outlet manhole in foreground. Raised manhole in background is magmeter chamber.	June 25, 2012

PHOTOGRAPH DESCRIPTIONS
GEOTECHNICAL SITE INSPECTIONS

PHOTO NUMBER	SITE NUMBER	SITE NAME	BANK SIDE	DIRECTION FACING	PHOTOGRAPH DESCRIPTION	DATE
627	8	St. Vital Bridge	S	E	Photo taken from magmeter chamber toward forcemain outlet	June 25, 2012
628	8	St. Vital Bridge	S	N	Outfall manholes	June 25, 2012
629	8	St. Vital Bridge	S	N	50mm diameter outlet for magmeter leak trial. Crews were to coat and re-insulate.	June 25, 2012
630, 631	8	St. Vital Bridge	S	W	Slope inclinometer casing west of west bridge	June 25, 2012
632	8	St. Vital Bridge	S	N	Pipe mounted on bridge	June 25, 2012
633	8	St. Vital Bridge	S	N	Pipe enters ground at first pier from water edge	June 25, 2012
634	8	St. Vital Bridge	S	W	Alignment of forcemain	June 25, 2012
635 to 638	8	St. Vital Bridge	S	E	Lower bank with rip rap armor at toe	June 25, 2012
639	8	St. Vital Bridge	S	W	Rip rap at toe	June 25, 2012
640, 641	8	St. Vital Bridge	S	E	Steep bank and sparse rip rap at bank toe downstream of pier	June 25, 2012
642, 643	8	St. Vital Bridge	S	S	Oversteepened bank at outfall locations	June 25, 2012
644	8	St. Vital Bridge	S	W	Oversteepened bank at outfall locations	June 25, 2012
645	8	St. Vital Bridge	S	E	Oversteepened bank at outfall locations	June 25, 2012
646	8	St. Vital Bridge	S	E	Upper bank of oversteepened bank	June 25, 2012
647	8	St. Vital Bridge	S	E	Separation in drain channel joint at east side of east bridge	June 25, 2012
648	8	St. Vital Bridge	S	W	Separation and offset at drain channel joint at west side of west bridge	June 25, 2012
649, 650	8	St. Vital Bridge	S	W	Patch of channel drain cracks	June 25, 2012
651	8	St. Vital Bridge	S	N	West channel drain	June 25, 2012
652	8	St. Vital Bridge	S	E	Downslope of slope inclinometer casing with sparse rip rap at bank toe	June 25, 2012
653	8	St. Vital Bridge	S	S	Minor surficial displacement and slumping	June 25, 2012
654	8	St. Vital Bridge	S	S	Erosion adjacent to west drain channel	June 25, 2012
655 to 658	8	St. Vital Bridge	S	S, W, E	Slope Inclinometer 3 located west of west bridge	June 25, 2012
659	8	St. Vital Bridge	S	S	West drain offset	June 25, 2012
660	9	Assiniboia Feedermain	N	S	Assiniboine Avenue at Rouge Road from road	June 22, 2012
661	9	Assiniboia Feedermain	N	N	Street location	June 22, 2012
662	9	Assiniboia Feedermain	N	S	Right-of-way	June 22, 2012
663	9	Assiniboia Feedermain	N	S	Toe of bank	June 22, 2012
664	9	Assiniboia Feedermain	N	W	Steepened edge	June 22, 2012
665	9	Assiniboia Feedermain	N	E	Plateau and then steepened bank to river at neighbouring property.	June 22, 2012
666	9	Assiniboia Feedermain	N	N	Right-of-way (ROW) from river edge	June 22, 2012
667 to 677	9	Assiniboia Feedermain	N	S	Panoramic view of south bank.	June 22, 2012
678	9	Assiniboia Feedermain	N	W	River edge	June 22, 2012
679	9	Assiniboia Feedermain	N	E	River edge	June 22, 2012
680	9	Assiniboia Feedermain	N	W	East of right-of-way with steepened mid bank	June 22, 2012
681	9	Assiniboia Feedermain	N	E	East of right-of-way with steepened mid bank	June 22, 2012
682, 683	9	Assiniboia Feedermain	N	N	Toe of bank with potential scarp or toe erosion	June 22, 2012
684	9	Assiniboia Feedermain	N	W	West of right-of-way at third property from ROW. Lower bench and evidence of past slope failure.	June 22, 2012
685	9	Assiniboia Feedermain	N	NW	Slope at mid bank. First property west of ROW.	June 22, 2012
686	9	Assiniboia Feedermain	N	E	Toe scarp and erosion. See Photo 682, 683.	June 22, 2012
687	9	Assiniboia Feedermain	N	W	First property west.	June 22, 2012
688	9	Assiniboia Feedermain	S	N	From street to ROW	June 22, 2012
689	9	Assiniboia Feedermain	S	N	From street along ROW	June 22, 2012
690 to 700	9	Assiniboia Feedermain	S	N	Panoramic view of north bank	June 22, 2012
701	9	Assiniboia Feedermain	S	S	ROW from river	June 22, 2012

PHOTOGRAPH DESCRIPTIONS
GEOTECHNICAL SITE INSPECTIONS

PHOTO NUMBER	SITE NUMBER	SITE NAME	BANK SIDE	DIRECTION FACING	PHOTOGRAPH DESCRIPTION	DATE
702	9	Assiniboia Feedermain	S	W	First property west of ROW (upstream) at mid bank	June 22, 2012
703	9	Assiniboia Feedermain	S	E	First property east of ROW (downstream)	June 22, 2012
704	9	Assiniboia Feedermain	S	E	First property west of ROW at toe. Steepened toe and berm. Facing downstream.	June 22, 2012
705	9	Assiniboia Feedermain	S	W	First property west of ROW at toe. Steepened toe and berm. Facing upstream.	June 22, 2012
706	9	Assiniboia Feedermain	S	E	Scarp face downstream of ROW	June 22, 2012
707	9	Assiniboia Feedermain	S	W	Scarp face upstream of ROW. About 1.5m high upslope of berm at toe.	June 22, 2012
708 to 711	9	Assiniboia Feedermain	S	E	First property east of ROW at toe	June 22, 2012
712	9	Assiniboia Feedermain	S	E	Slump block at toe. Facing downstream.	June 22, 2012
713	9	Assiniboia Feedermain	S	W	Slump block at toe. Facing upstream.	June 22, 2012
714	10	Goulet Doucet Watermain	W	W	Alignment of watermain from end of road barrier	June 26, 2012
715, 716	10	Goulet Doucet Watermain	W	E	Alignment of watermain	June 26, 2012
717	10	Goulet Doucet Watermain	W	W	Upper bank on alignment	June 26, 2012
718	10	Goulet Doucet Watermain	W	E	Lower bank facing toward location of bend in pipe	June 26, 2012
719	10	Goulet Doucet Watermain	W	W	Notebook is approximate location of bend in watermain as scaled from GIS drawing. Slightly raised berm at this location suggests trench backfill mounding and accurate pipe location.	June 26, 2012
720	10	Goulet Doucet Watermain	W	E	Alignment leading toward river. Notebook at at approximate pipe bend location.	June 26, 2012
721 to 725	10	Goulet Doucet Watermain	W	W	Upper bank with large spreading tree on approximate watermain location (Photo 724)	June 26, 2012
726 to 728	10	Goulet Doucet Watermain	W	N	Alignment at upper bank taken from the south	June 26, 2012
729 to 734	10	Goulet Doucet Watermain	W	N	Alignment at upper bank taken from the south. Notebook at pipe bend.	June 26, 2012
735 to 738	10	Goulet Doucet Watermain	W	W	Lower bench on alignment facing toward pipe bend	June 26, 2012
739 to 743	10	Goulet Doucet Watermain	W	E	Lower bench on alignment facing toward river	June 26, 2012
744 to 749	10	Goulet Doucet Watermain	W	E	Panoramic view of east bank toward Rue Doucet	June 26, 2012
750	10	Goulet Doucet Watermain	W	E	Toe at alignment taken from upstream location	June 26, 2012
751 to 754	10	Goulet Doucet Watermain	W	N	Minor erosion and 1m high steepened bank at river edge on pipe crossing alignment	June 26, 2012
755	10	Goulet Doucet Watermain	W	S	Toe of river bank	June 26, 2012
756	10	Goulet Doucet Watermain	E	E	Along pipe alignment on Rue Doucet	June 26, 2012
757	10	Goulet Doucet Watermain	E	W	Along pipe alignment on Rue Doucet	June 26, 2012
758	10	Goulet Doucet Watermain	E	W	Upper bank transition to lower bench along pipe alignment	June 26, 2012
759	10	Goulet Doucet Watermain	E	W	Photo taken from toe of upper bank	June 26, 2012
760, 761	10	Goulet Doucet Watermain	E	E	Upper bank on alignment	June 26, 2012
762	10	Goulet Doucet Watermain	E	W	Opposite bank on alignment	June 26, 2012
763 to 766	10	Goulet Doucet Watermain	E	W	Panoramic view of west bank	June 26, 2012
767	10	Goulet Doucet Watermain	E	S	Toe of bank taken from pipe alignment location	June 26, 2012
768	10	Goulet Doucet Watermain	E	N	Toe of bank taken from pipe alignment location	June 26, 2012
769 to 771	10	Goulet Doucet Watermain	E	NE	Lower bench with photo taken from upstream location and facing pipe crossing alignment	June 26, 2012

PHOTOGRAPH DESCRIPTIONS
GEOTECHNICAL SITE INSPECTIONS

PHOTO NUMBER	SITE NUMBER	SITE NAME	BANK SIDE	DIRECTION FACING	PHOTOGRAPH DESCRIPTION	DATE
772 to 778	10	Goulet Doucet Watermain	E	S	Lower bench with photo taken from downstream location and facing pipe crossing alignment	June 26, 2012
779 to 783	10	Goulet Doucet Watermain	E	N	Toe of upper bank and lower bank taken downstream	June 26, 2012
784 to 786	10	Goulet Doucet Watermain	E	E	Upper bank with photo taken from south (upstream) of pipe crossing alignment	June 26, 2012
787 to 790	10	Goulet Doucet Watermain	E	N	Upper bank on alignment taken from south (upstream)	June 26, 2012
791	10	Goulet Doucet Watermain	E	N	First property north of pipe crossing alignment	June 26, 2012
792	10	Goulet Doucet Watermain	E	E	Outfall pipe in line with centreline of Rue Doucet from sewer manhole at top of slope with 300mm diameter	June 26, 2012
793	10	Goulet Doucet Watermain	E	W	Opposite bank close view on pipe crossing alignment	June 26, 2012
796	11	Kildonan Redwood Feedermain	W	E	Along alignment of feedermain	June 26, 2012
797	11	Kildonan Redwood Feedermain	W	W	Along alignment of feedermain	June 26, 2012
798	11	Kildonan Redwood Feedermain	W	E	Alignment at upper bank with grouted rip rap	June 26, 2012
799	11	Kildonan Redwood Feedermain	W	W	Upper bank with grouted rip rap and concrete drain channel on feedermain alignment	June 26, 2012
800, 801	11	Kildonan Redwood Feedermain	W	E	On feedermain crossing alignment across river at toe of west bank	June 26, 2012
802 to 805	11	Kildonan Redwood Feedermain	W	W	Photo taken from river edge facing upslope on feedermain crossing alignment	June 26, 2012
806 to 813	11	Kildonan Redwood Feedermain	W	E	Panoramic view of east bank (Glenwood Crescent) north (downstream) of bridge	June 26, 2012
814 to 823	11	Kildonan Redwood Feedermain	W	E	Panoramic view of east bank (Glenwood Crescent) south (upstream) of bridge	June 26, 2012
824	11	Kildonan Redwood Feedermain	W	S	Bridge abutment	June 26, 2012
825, 826	11	Kildonan Redwood Feedermain	W	S	Toe at river edge	June 26, 2012
827	11	Kildonan Redwood Feedermain	W	S	Upper bank north of bridge	June 26, 2012
828 to 840	11	Kildonan Redwood Feedermain	W		Grouted rip rap on north side of bridge. Patches evident at cracks. Crack separation and offset present.	June 26, 2012
841 to 844, 847	11	Kildonan Redwood Feedermain	W		Grouted rip rap on south side of bridge. Not as many cracks or as severe cracks as on north side.	June 26, 2012
845, 846	11	Kildonan Redwood Feedermain	W		Slope inclinometer casing on south side of bridge with flush mount in grouted rip rap	June 26, 2012
848	11	Kildonan Redwood Feedermain	W	N	Toe of river bank	June 26, 2012
849, 850	11	Kildonan Redwood Feedermain	W	S	Toe of river bank	June 26, 2012
851	11	Kildonan Redwood Feedermain	W	S	Limestone blocks under bridge along path	June 26, 2012
852	11	Kildonan Redwood Feedermain	W	W	North concrete drain channel	June 26, 2012
853	11	Kildonan Redwood Feedermain	W	S	Upper limestone blocks under bridge along path	June 26, 2012
854	11	Kildonan Redwood Feedermain	W	S	Upper limestone blocks under bridge along path	June 26, 2012
855	11	Kildonan Redwood Feedermain	W	N	Upper limestone blocks under bridge along path	June 26, 2012
856, 863	11	Kildonan Redwood Feedermain	E	W	On pipe crossing alignment facing toward river	June 26, 2012
857, 862	11	Kildonan Redwood Feedermain	E	E	On pipe crossing alignment facing toward river	June 26, 2012
858 to 860, 864	11	Kildonan Redwood Feedermain	E	W	Chamber locations	June 26, 2012
861	11	Kildonan Redwood Feedermain	E	SW	Bridge abutment	June 26, 2012
865, 867	11	Kildonan Redwood Feedermain	E	E	On alignment	June 26, 2012
866, 868	11	Kildonan Redwood Feedermain	E	W	On alignment	June 26, 2012
869	11	Kildonan Redwood Feedermain	E	E	On alignment	June 26, 2012

PHOTOGRAPH DESCRIPTIONS
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 Project number: 60270487

PHOTO NUMBER	SITE NUMBER	SITE NAME	BANK SIDE	DIRECTION FACING	PHOTOGRAPH DESCRIPTION	DATE
870	11	Kildonan Redwood Feedermain	E	E	Cable conduit on underside of bridge disconnected	June 26, 2012
871 to 877	11	Kildonan Redwood Feedermain	E	E	Mid to upper bank under bridge	June 26, 2012
878	11	Kildonan Redwood Feedermain	E	N	Erosion downstream of bridge	June 26, 2012
879	11	Kildonan Redwood Feedermain	E	S	Erosion upstream of bridge	June 26, 2012
880	11	Kildonan Redwood Feedermain	E	N	Erosion downstream of bridge with erosion scarp 2.4m high	June 26, 2012
881	11	Kildonan Redwood Feedermain	E	NE	Concrete overlain with river sediment	June 26, 2012
882	11	Kildonan Redwood Feedermain	E	S	Erosion and near vertical bank about 2.7m high	June 26, 2012
883	11	Kildonan Redwood Feedermain	E	S	Large diameter cables south of bridge	June 26, 2012
884	11	Kildonan Redwood Feedermain	E	N	Erosion on both side of bridge	June 26, 2012
885 to 887	11	Kildonan Redwood Feedermain	E	N	Top of bank at first property south of bridge	June 26, 2012
888	11	Kildonan Redwood Feedermain	E	S	Top of bank at first property south of bridge	June 26, 2012
889 to 891	11	Kildonan Redwood Feedermain	E	S	Top of bank at first property south of bridge	June 26, 2012
892 to 894	11	Kildonan Redwood Feedermain	E	N	Alignment of feedermain	June 26, 2012
895 to 898	11	Kildonan Redwood Feedermain	E	N	First property north of bridge	June 26, 2012
899 to 902	11	Kildonan Redwood Feedermain	E	N	Second property north of bridge	June 26, 2012
903	11	Kildonan Redwood Feedermain	E	S	Photo taken from second property north of pipe crossing alignment	June 26, 2012
904	11	Kildonan Redwood Feedermain	E	N	Photo taken from second property north of pipe crossing alignment	June 26, 2012
905	11	Kildonan Redwood Feedermain	E	S	Large trees north of bridge on first property north of pipe crossing alignment	June 26, 2012
906	11	Kildonan Redwood Feedermain	E	N	Steep eroded bank at first property north of pipe crossing alignment	June 26, 2012
907 to 909	11	Kildonan Redwood Feedermain	E	S	Steep eroded bank at first property north of pipe crossing alignment	June 26, 2012
910	11	Kildonan Redwood Feedermain	E	S	Photo taken from first property north facing toward feedermain and bridge	June 26, 2012
911	11	Kildonan Redwood Feedermain	E	E	Upper bank on feedermain alignment	June 26, 2012
912 to 922	11	Kildonan Redwood Feedermain	E	W	Panoramic view of west bank north of bridge	June 26, 2012
923 to 934	11	Kildonan Redwood Feedermain	E	W	Panoramic view of west bank south of bridge	June 26, 2012
935 to 947	12	Maryland Bridge Watermain	N	S	Panoramic view of south bank.	June 25, 2012
948	12	Maryland Bridge Watermain	N	N	Along pipe alignment	June 25, 2012
949	12	Maryland Bridge Watermain	N	S	Along pipe alignment	June 25, 2012
950	12	Maryland Bridge Watermain	N	E	East of alignment	June 25, 2012
951	12	Maryland Bridge Watermain	N	N	Along pipe alignment	June 25, 2012
952	12	Maryland Bridge Watermain	N	S	Along pipe alignment	June 25, 2012
953	12	Maryland Bridge Watermain	N	E	Along edge of river	June 25, 2012
954	12	Maryland Bridge Watermain	N	W	Along edge of river	June 25, 2012
955	12	Maryland Bridge Watermain	N	E	Edge of river	June 25, 2012
956	12	Maryland Bridge Watermain	N	E	Lower bank from west bridge	June 25, 2012
957	12	Maryland Bridge Watermain	N	W	From ROW facing upstream	June 25, 2012
958	12	Maryland Bridge Watermain	N	E	From ROW facing downstream	June 25, 2012
959 to 963	12	Maryland Bridge Watermain	N	N, S, E	Gully erosion at east bridge pier	June 25, 2012
964	12	Maryland Bridge Watermain	N	W	Toe from east bridge	June 25, 2012
965	12	Maryland Bridge Watermain	N	N	Erosion at east bridge	June 25, 2012
966	12	Maryland Bridge Watermain	N	W	Mid bank from east bridge	June 25, 2012
967	12	Maryland Bridge Watermain	N	S	South bank, closer view	June 25, 2012
968	12	Maryland Bridge Watermain	N	E	Toe of bank at west bridge	June 25, 2012
969 to 987	12	Maryland Bridge Watermain	S	N	Panoramic view of north bank	June 25, 2012
988 to 990	12	Maryland Bridge Watermain	S	N	Panoramic view of north bank taken from east bridge	June 25, 2012
991	12	Maryland Bridge Watermain	S	W	Facing toward pipe crossing alignment	June 25, 2012
992 to 999	12	Maryland Bridge Watermain	S	E, N, S	Gully erosion at east bridge	June 25, 2012
1000	12	Maryland Bridge Watermain	S	W	Stepped erosion at east bridge	June 25, 2012
1001	12	Maryland Bridge Watermain	S	N	Along alignment	June 25, 2012
1002	12	Maryland Bridge Watermain	S	S	Along alignment	June 25, 2012
1003	12	Maryland Bridge Watermain	S	N	Along alignment	June 25, 2012
1004	12	Maryland Bridge Watermain	S	S	Upstream of alignment	June 25, 2012

PHOTOGRAPH DESCRIPTIONS
GEOTECHNICAL SITE INSPECTIONS

PHOTO NUMBER	SITE NUMBER	SITE NAME	BANK SIDE	DIRECTION FACING	PHOTOGRAPH DESCRIPTION	DATE
1005 to 1007	12	Maryland Bridge Watermain	S	N, W, S	Small erosion gully at west bridge	June 25, 2012
1008	12	Maryland Bridge Watermain	S	E	Toe at watermain crossing alignment	June 25, 2012
1009	13	North Kildonan Feedermain	W	W	Along ROW facing toward Main Street	June 23, 2012
1010	13	North Kildonan Feedermain	W	E	Along ROW facing toward river	June 23, 2012
1011, 1012	13	North Kildonan Feedermain	W		On John Black Avenue	June 23, 2012
1013	13	North Kildonan Feedermain	W	W	ROW at upper bank	June 23, 2012
1014 to 1022	13	North Kildonan Feedermain	W	W	Panoramic view of west bank	June 23, 2012
1023	13	North Kildonan Feedermain	W	E	First property south of ROW. Vacant with 0.6m-high stone retaining wall remaining. Photo taken from top of wall.	June 23, 2012
1024	13	North Kildonan Feedermain	W	N	First property south of ROW. Stone retaining wall.	June 23, 2012
1025 to 1034	13	North Kildonan Feedermain	W	S, N, W	Stone retaining wall in good condition	June 23, 2012
1035 to 1037	13	North Kildonan Feedermain	W	N	West bank facing north to ROW taken from first property south of ROW	June 23, 2012
1038	13	North Kildonan Feedermain	W	N	First property north of ROW. Lower bank.	June 23, 2012
1039, 1040	13	North Kildonan Feedermain	W	N	First property north of ROW. Lower bank. Grouted rip rap at toe and dock.	June 23, 2012
1041	13	North Kildonan Feedermain	W	S	Toe and river edge along ROW and first property north and south of ROW	June 23, 2012
1042 to 1046	13	North Kildonan Feedermain	W	W	Second and third properties north of ROW	June 23, 2012
1047	13	North Kildonan Feedermain	W	N	Second property north of ROW. Toe and river edge.	June 23, 2012
1048 to 1060	13	North Kildonan Feedermain	W	E	Panoramic view of east bank for Site 13	June 23, 2012
1061	13	North Kildonan Feedermain	W	N	Bank toe and river edge north (downstream) of ROW	June 23, 2012
1062, 1063	13	North Kildonan Feedermain	W	S	Bank toe and river edge facing south (upstream) with ROW in foreground	June 23, 2012
1064	13	North Kildonan Feedermain	W	S	Culvert extending from bank at river edge immediately south of grouted rip rap. Culvert is corrugated metal pipe about 0.6m in diameter and partly crushed.	June 23, 2012
1065	13	North Kildonan Feedermain	W	S	Shoreline erosion at ROW	June 23, 2012
1066, 1067	13	North Kildonan Feedermain	W	N	Bank toe at ROW	June 23, 2012
1068 to 1070	13	North Kildonan Feedermain	W	E, N, W	Potential slope inclinometer casing at bank toe	June 23, 2012
465	13	North Kildonan Feedermain	W	N	Steep lower toe with slump block. Shallow scarps at toe.	June 23, 2012
466	13	North Kildonan Feedermain	W	S	Lower bank south of ROW and north of bridge	June 23, 2012
467	13	North Kildonan Feedermain	W	S	Large poplar trees on ROW	June 23, 2012
1074 to 1080	13	North Kildonan Feedermain	E	E	Starting from manhole proceeding east along ROW toward Henderson Highway parallel to river	June 23, 2012
1081 to 1091	13	North Kildonan Feedermain	E	W	Proceeding west along ROW toward manhole parallel to river	June 23, 2012
1092	13	North Kildonan Feedermain	E	S	Upstream from ROW at edge of river bank	June 23, 2012
1093	13	North Kildonan Feedermain	E	N	Downstream from ROW at edge of river bank	June 23, 2012
1094, 1095	13	North Kildonan Feedermain	E	N	Facing toward ROW downstream	June 23, 2012
1096	13	North Kildonan Feedermain	E	S	Facing toward bridge along bank toe	June 23, 2012
1097	13	North Kildonan Feedermain	E	S	Facing toward ROW	June 23, 2012
1098	13	North Kildonan Feedermain	E	S	Facing toward ROW upstream	June 23, 2012
1099	13	North Kildonan Feedermain	E	S	River bank toe at ROW	June 23, 2012
1100 to 1103	13	North Kildonan Feedermain	E	E	Head scarp at river bank toe	June 23, 2012
1104 to 1115	13	North Kildonan Feedermain	E	W	Panoramic view of west bank of Site 13	June 23, 2012
1116	14	St. James Street Watermain	N	N	Along watermain alignment from corner of Wolseley Avenue and St. James Street	June 25, 2012
1117	14	St. James Street Watermain	N	S	Along watermain alignment from corner of Wolseley Avenue and St. James Street	June 25, 2012
1118	14	St. James Street Watermain	N	S	Along pipe alignment adjacent to 1610 Wolseley Avenue	June 25, 2012

PHOTOGRAPH DESCRIPTIONS
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PHOTO NUMBER	SITE NUMBER	SITE NAME	BANK SIDE	DIRECTION FACING	PHOTOGRAPH DESCRIPTION	DATE
1119	14	St. James Street Watermain	N	N	Along pipe alignment adjacent to 1610 Wolseley Avenue	June 25, 2012
1120, 1121	14	St. James Street Watermain	N	N	Upper bank at back edge of 1610 Wolseley Avenue. Approximately 3.7m high.	June 25, 2012
1122	14	St. James Street Watermain	N	E	Downstream view of upper bank face	June 25, 2012
1123	14	St. James Street Watermain	N	S	Along watermain alignment	June 25, 2012
1124	14	St. James Street Watermain	N	W	Top of bank	June 25, 2012
1125	14	St. James Street Watermain	N	E	Second property east of ROW with steep slope and cut lawn	June 25, 2012
1126	14	St. James Street Watermain	N	E	Top of bank at 1610 Wolseley Avenue	June 25, 2012
1127, 1128	14	St. James Street Watermain	N	W	Top of upper bank	June 25, 2012
1129	14	St. James Street Watermain	N	E	Top of upper bank	June 25, 2012
1130 to 1133	14	St. James Street Watermain	N	E	Lower bank bench with sand deposit and berm at river edge	June 25, 2012
1134	14	St. James Street Watermain	N	W	Near-vertical bank toe with photo taken from fourth property east of ROW	June 25, 2012
1135, 1136	14	St. James Street Watermain	N	W	Lower bank bench with photo taken from seventh property east of ROW	June 25, 2012
1137 to 1157	14	St. James Street Watermain	N	S	Panoramic view of south bank.	June 25, 2012
1158 to 1160	14	St. James Street Watermain	N	E	Lower bank and river edge with photo 1158 (105) taken from railway pier	June 25, 2012
1161	14	St. James Street Watermain	S	W	Shoes hanging from wire at road near crossing	June 25, 2012
1162	14	St. James Street Watermain	S	N	Watermain alignment at road with manhole cover in eastbound lane (south lane)	June 25, 2012
1163	14	St. James Street Watermain	S	E	Adjacent to alignment and parallel to Wellington Crescent	June 25, 2012
1164	14	St. James Street Watermain	S	W	Adjacent to alignment and parallel to Wellington Crescent	June 25, 2012
1165, 1166	14	St. James Street Watermain	S	S	Along watermain alignment with manhole cover in eastbound lane (south lane)	June 25, 2012
1167 to 1180	14	St. James Street Watermain	S	N	Panoramic view of north bank	June 25, 2012
1181	14	St. James Street Watermain	S	E	Erosion at bank toe with photo taken from railway bridge	June 25, 2012
1182	14	St. James Street Watermain	S	W	Concrete sandbag armoring along railway bridge bank toe	June 25, 2012
1183 to 1186	14	St. James Street Watermain	S	S	First culvert outfall east of railway bridge. Culvert is 400mm diameter corrugated metal pipe	June 25, 2012
1187 to 1193	14	St. James Street Watermain	S	S	Second clay tile outfall east of railway with 470mm outside diameter and 390mm inside diameter	June 25, 2012
1194	14	St. James Street Watermain	S	S	Scarp and tension crack on watermain alignment	June 25, 2012
1195	14	St. James Street Watermain	S	W	Tension crack	June 25, 2012
1196 to 1200	14	St. James Street Watermain	S	W	Slump block	June 25, 2012
1201, 1202	14	St. James Street Watermain	S	S	Tension cracks	June 25, 2012
1203 to 1205	14	St. James Street Watermain	S	N	Slump block east of pipe crossing alignment	June 25, 2012
1206 to 1209	14	St. James Street Watermain	S	W	Slump blocks and trees toppled into river	June 25, 2012
1210	14	St. James Street Watermain	S	E	East of ROW with undercut bank, overhanging roots, and slump block.	June 25, 2012
1211 to 1214	14	St. James Street Watermain	S	W	Toppled trees and slump blocks on watermain alignment	June 25, 2012
1215	14	St. James Street Watermain	S	S	Exposed face of failure surface at location of slump block and toppled tree (first toppled tree east of railway crossing)	June 25, 2012
1216	14	St. James Street Watermain	S	E	Second toppled tree east of railway crossing taken from location of first toppled tree	June 25, 2012
1217	14	St. James Street Watermain	S	W	Shoreline point extending into river channel consisting of railway concrete sandbag shoreline armor	June 25, 2012

PHOTOGRAPH DESCRIPTIONS
GEOTECHNICAL SITE INSPECTIONS

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 Project number: 60270487

PHOTO NUMBER	SITE NUMBER	SITE NAME	BANK SIDE	DIRECTION FACING	PHOTOGRAPH DESCRIPTION	DATE
1218 to 1230	14	St. James Street Watermain	S	E	Photos taken from St. James Bridge	June 25, 2012
1237	14	St. James Street Watermain	S	E	Toppled trees and slump blocks on watermain alignment with lower bank exposed due to reduced river level	August 15, 2012
1238, 1239	14	St. James Street Watermain	S	E	Slump block toppled between June 25 and August 15, 2012	August 15, 2012
1240	14	St. James Street Watermain	S	W	Slump block and toppled trees	August 15, 2012

Appendix C

Geotechnical Slope Stability Analysis

Technical Memorandum

To **Marv McDonald** Page **1**

CC

Subject **Geotechnical Slope Stability Analysis
 High Risk Water and Wastewater River Crossings**

From **Alex Hill**

Date **March 24, 2016** Project Number **60270487 (400.01.03.22)**

1. Stability Assessment

1.1 General

Based on the findings of the site inspection completed between June 22 and June 27, 2012, five sites were selected to perform a preliminary slope stability analysis. The subject sites were selected based on the observed conditions and the importance of the potentially impacted assets. The five locations are shown in Table 1.

Table 1: Riverbank Slope Stability Analysis

Site No.	Location	Riverbank	River
4	St James Interceptor	South	Sturgeon
5	Assiniboine Park Siphon	South	Assiniboine
6	Munroe Polson Siphon	East	Red
11	Kildonan Redwood Feedermain	East	Red
14	St James Street Watermain	South	Assiniboine

1.2 Methodology

1.2.1 Stability Analysis

Slope stability models were developed using GeoStudio 2007. The riverbank geometries were established based on recent Lidar survey provided by the City, recent bathymetric data and where available, Record Drawings. The soil stratigraphy for the stability models was derived from geological maps and available test hole information. Assumptions were necessary to facilitate the analysis where local or detailed information was limited. The pipe location at each crossing has been taken from Record Drawings, and attempts were made to infer its profile within the slope stability models.

River elevations used within the slope stability models were based on information sourced from the City of Winnipeg’s online database (<http://www.winnipeg.ca/publicworks/pwddata/riverlevels/>)

accessed September to October 2012). River elevations were adjusted to reflect high and low water events as shown in Table 3: River Elevations

Upon establishing a slope stability model for each location, assessment was performed using Morgenstern-Price's general method of slices based on a limit equilibrium approach. More advanced methods (such as finite element analysis) were not used for this study as the uncertainties associated with material parameters, soil stratigraphy and piezometric conditions would not justify the rigorous approach of a more complex analysis method.

As part of the analysis the following slip surfaces were considered of interest and presented graphically as Figure 1. A Factor of Safety (FS) was assigned to each of the following;

- **Critical Slip Surface (CS):** is defined as a slip surface that encompasses part of the river bank which would likely compromise the global stability of river bank. Only slip surfaces with a depth of 0.5m or greater have been assessed.
- **Global Slip Surface (GS):** is defined as a slip surface that largely encompasses the slope soil mass, and has an entry and exit point at or just beyond the slope crest and toe.
- **Global Slip Surface Engaging Pipe (GS+P):** is defined as a slip surface that meets the criteria of a global slip surface and encompasses part of the buried pipe.
- **Toe Slip Surface (TS):** is defined as a slip surface that is localised to the toe of the slope, which has a minimum depth of 0.5m. At some locations the FS of this slip surface may be lower than the critical or global FS. Instability at the toe of the slope may reduce the FS for the global or critical slip surfaces. Retrogressive failures starting at the toe may also work towards the riverbank.

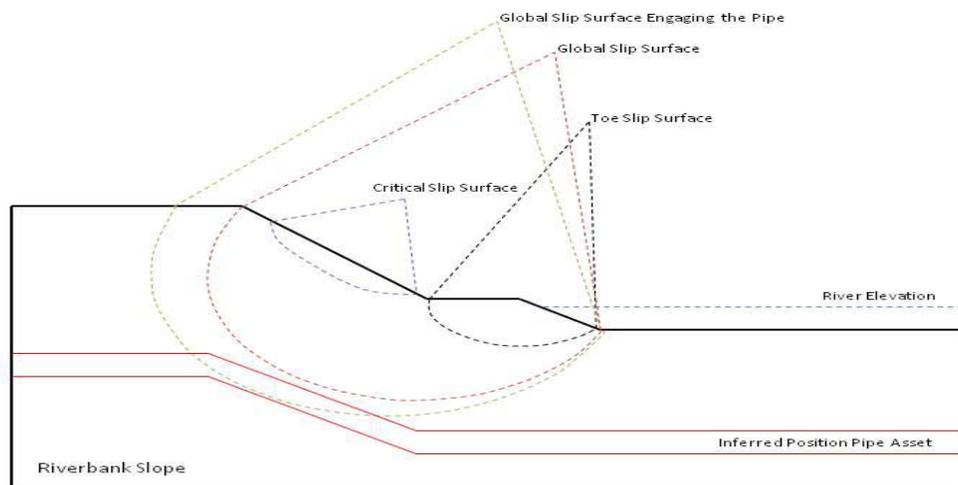


Figure 1: Assessed Slip Surfaces within Analysis

Values of FS close to 1.0 suggest that the riverbank slope is of marginal stability (i.e., all resisting forces and gravitational forces within the riverbank slope are in critical balance).

Acceptable FS can be defined between a range of 1.3 and 1.5 depending on several factors including but not limited to associated impact of instability, risk management approach and related cost to improve the stability.

1.2.2 Soil Parameters

Soil strength parameters used in the stability analysis are presented in Table 2. In the absence of laboratory testing, material parameters have been selected based on local knowledge and past experience which are considered locally acceptable values.

Table 2: Soil Strength Parameters for Stability Analysis

Stratum	Moist Bulk Unit Weight (kN/m ³)	Internal Angle of Friction (Degrees)	Cohesion (kPa)
Alluvium deposit	17	26	0
Lacustrine clay	17	17	5
Glacial Till	21	30	10

Residual soil strength values have not been assigned within the slope stability models due to the absence of accurate soil profile data and topographic survey data to provide confirmation of the field observations.

1.2.3 Surface and Groundwater Conditions

Based on river elevation information obtained from the City of Winnipeg's online database, river elevations used in the assessment for each location are presented in Table 3 below.

Table 3: River Elevations

River Elevation (m)					
River Conditions	Site 4- St James Interceptor	Site 5- Assiniboine Park Siphon	Site 6- Munroe-Polson Siphon	Site 11- Kildonan Redwood Feedermain	Site 14- St James Street Watermain
Summer	228.25	225.90	221.43	223.10	225.00
Slightly Elevated	228.75	226.50	223.69	223.78	225.50
Elevated	229.25	227.00	224.50 – 225.00	224.50	226.00

In the absence of groundwater information at each crossing location, the groundwater condition prevailing at the river bank has been assumed to be approximately 2 to 3 m below ground surface and matching the river level at the water edge.

1.3 Results and Discussion

The analysis results are tabulated in Appendix A and summarized in Section 1.4. Computer outputs for the stability model are presented graphically in Appendix B, and the results are discussed below.

1.3.1 Site 4: St James Interceptor (Sturgeon Creek) - South Bank

The calculated FS for selected slip surfaces (CS, GS, and GS+P) are presented in Figure 2 for the anticipated range of river water levels. The CS results indicate a case of imminent instability dependent upon the river water level, and is manifested by oversteepened river bank slopes. These CS's are primarily surficial slides which would be limited to shallow slough and can be addressed by undertaking grading works to introduce flatter slopes. The calculated FS for slip surfaces designated as GS and GS+P vary between 1.0 and 1.2. While the slope can be considered marginally stable, the presence of critical assets within the river bank call for improved river bank stability to attain acceptable FS.

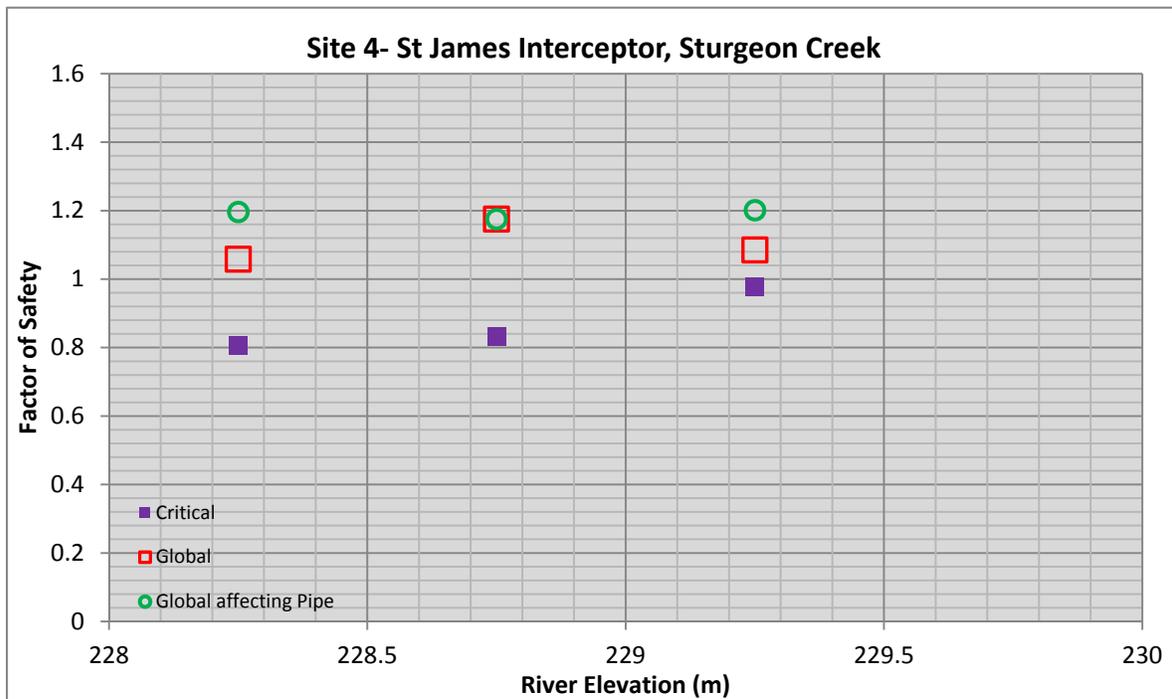


Figure 2: Factor of Safety relating to the Current Condition of Site 4

A stability output model illustrating the trace of the global slip surface engaging the pipe at the lowest calculated FS is shown in Figure 3 determined at a river elevation of 228.25m.

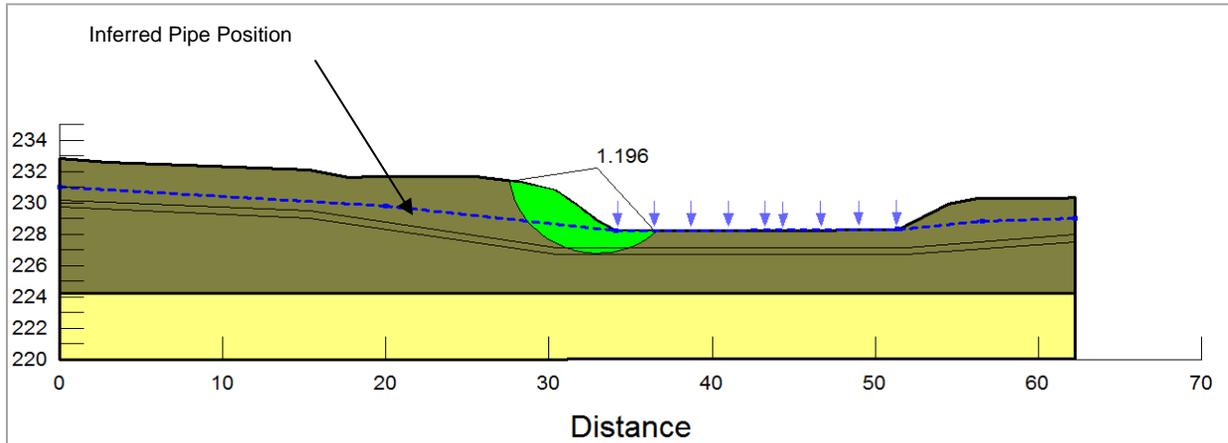


Figure 3: Site 4 St James Interceptor Global Slip Surface Engaging the Pipe (GS+P)

1.3.2 Site 5: Assiniboine Park Siphon (Assiniboine River) - South Bank

The calculated FS values for the riverbank, presented in Figure 4, correlates closely to the observations made in the field which had noted the presence of a slump block near the toe of the slope. The presence of a slump block and evidences of recent movement would suggest that the riverbank has a FS close to or less than 1.0 (imminent instability).

The presented graphical results indicate the trace of the GS and GS+P theoretical slip surfaces are likely to result in global instability that engage the pipe given the depth of the impacted mass. Retrogressive failure at the toe is also further likely to reduce the overall global stability of the riverbank, as potential failure surfaces migrate upslope.



Figure 4: Slope Stability Factor of Safety Relating to the Current Condition of Site 5

While a number of slip surfaces have FS values slightly lower than 0.8, in reality the actual FS are likely higher considering the stabilizing effect of the extensive vegetation growth. A stability output model illustrating the position of the global slip surface engaging the pipe at the lowest FS is shown in Figure 5, determined at a river elevation of 226.5 m.

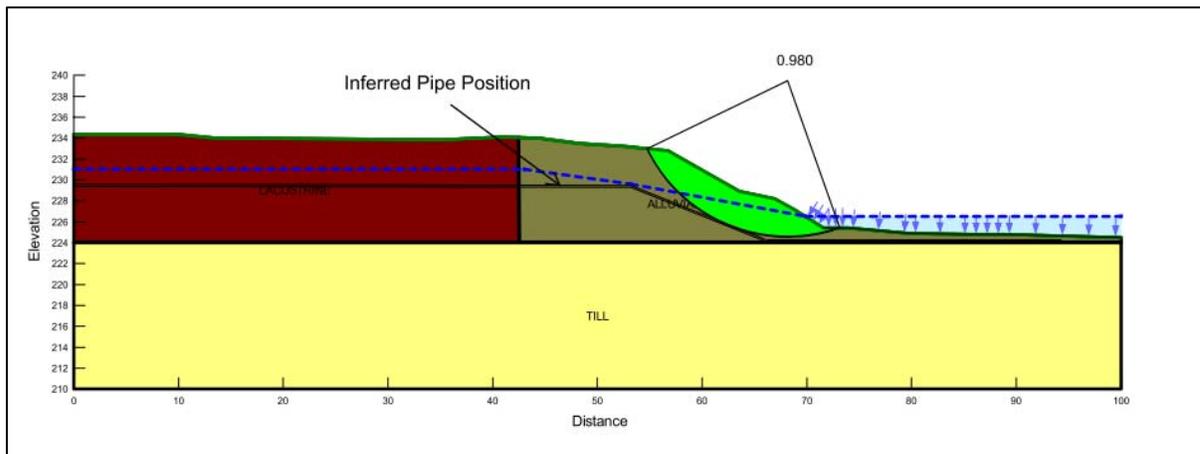


Figure 5: Site 5 Global Slip Surface Engaging the Pipe (GS+P)

1.3.3 Site 6: Munroe Polson (Red River) - East Bank

The results of the slope stability analysis presented in Figure 6 indicate that the calculated FS lies typically between 1.3 to 1.4. These values are considered an acceptable level of stability. The reported FS values are for the theoretical slip surfaces which represent CS, TS, GS and GS+P.

The field inspection largely supports the results of the slope stability where there was an absence of observations relating to instability within the immediate vicinity of the pipe crossing. Nevertheless, visual inspection of adjacent properties upstream of the Right of Way (ROW) have shown significant evidence of movement which has included head scarps, trees dislocation and topsoil creep which are not present along the riverbank slope of the pipe crossing. This is believed to be related to the recent removal of tree growth allowing for surficial movement within the upper riverbank materials.

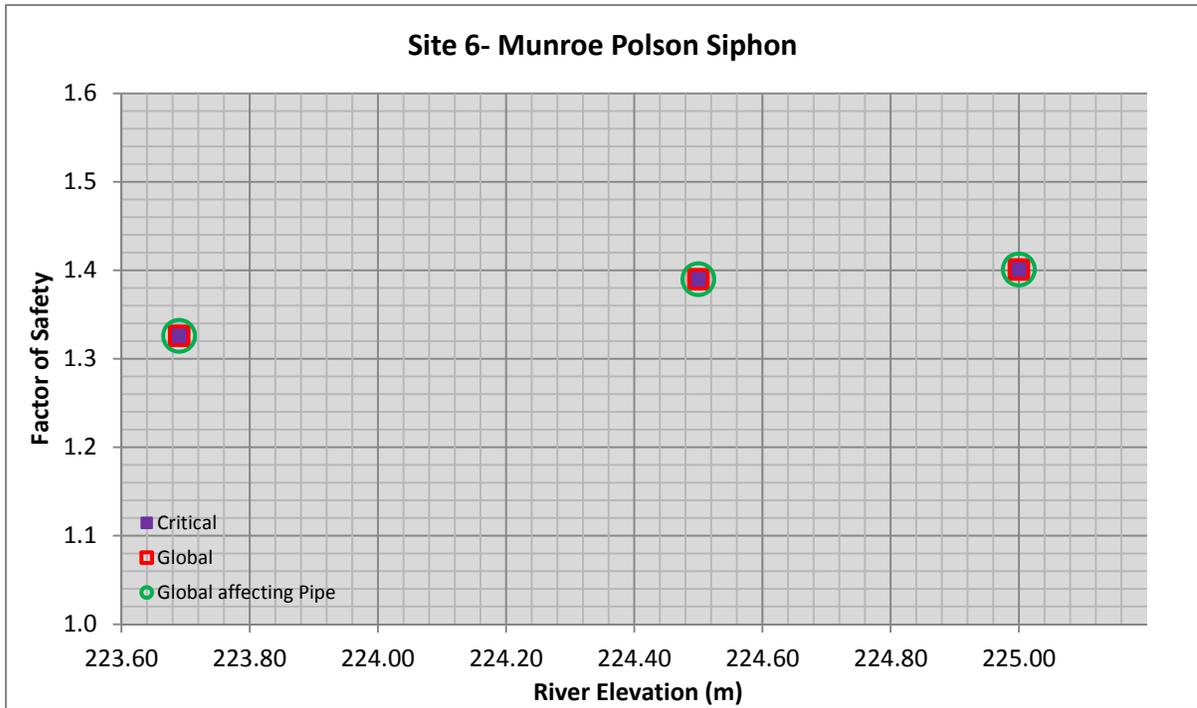


Figure 6: Slope Stability Factor of Safety Relating to the Current Condition to Site 5

A stability output model illustrating the position of the global slip surface engaging the pipe at the lowest FS is shown in Figure 7 determined at a river elevation of 223.69m.

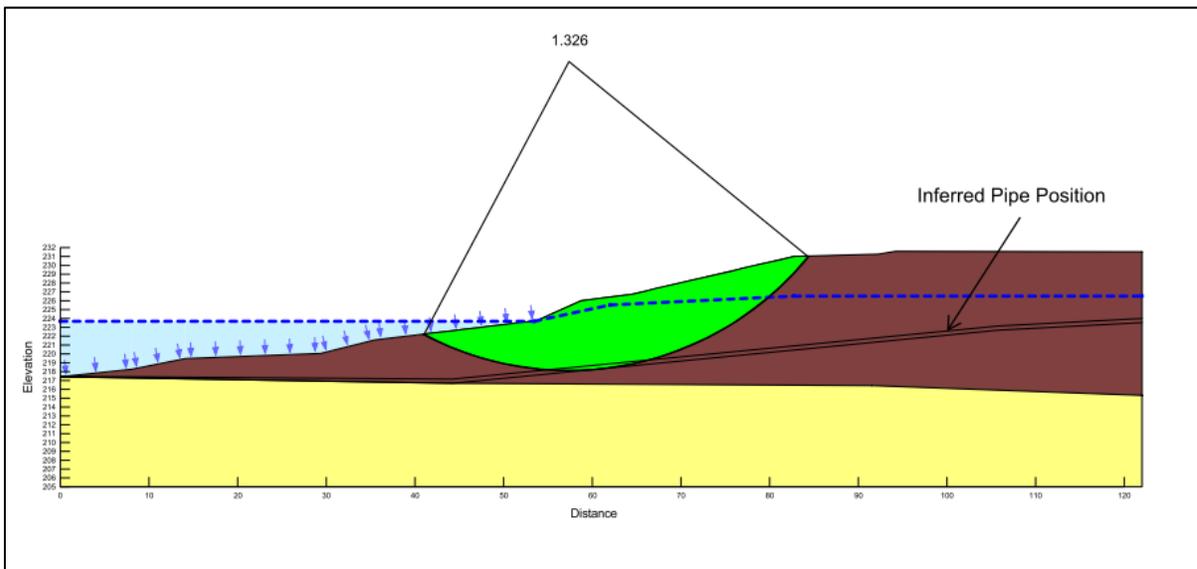


Figure 7: Munroe Polson Siphon- Global Slip Surface Engaging Pipe (GS+P)

1.3.4 Site 11: Kildonan Redwood Feedermain (Red River) - East Bank

The results of the slope stability assessment appear to confirm observations made during the field inspections in regard to the overall stability of the riverbank slope. Limited or no evidence of instability along the upper and mid slope portion of the riverbank was noted, however a large percentage of observations regarding instability were concentrated at the toe of the riverbank. The calculated FS for instability at the toe of the slope has been calculated at less than 0.8, whereas the FS for global instability has been calculated at less than 0.8, whereas the FS for global instability has been calculated between 1.0 and 1.2. This is largely due to the near vertical face at the toe of the riverbank created through erosion. The calculated FS for slip surfaces engaging the pipe has been calculated between 1.3 and 1.6. The FS for critical slip surfaces largely mirror values associated with toe instability of the riverbank slope. The results of the analysis are presented graphically in Figure 8.

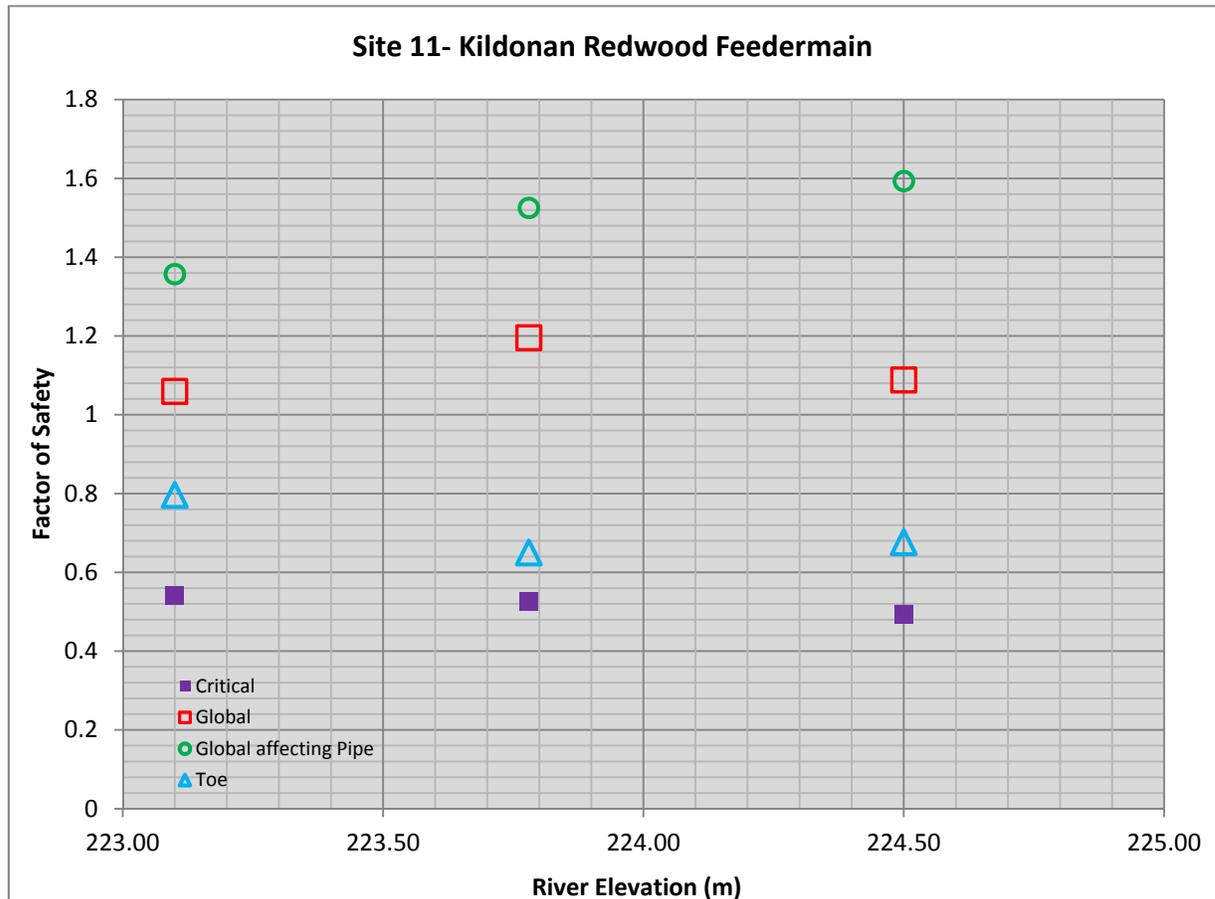


Figure 8: Slope Stability Factor of Safety Relating to the Current Conditions of Site 11

Reduction in overall global stability of the riverbank may occur should retrogressive failure at the toe extend further upslope and potentially engage the pipe. Placement of rock (rip-rap) and reshaping at the toe of the riverbank slope may control toe erosion, and thus reduce the potential for continued or retrogressive slips.

A stability output model illustrating the position of the global slip surface engaging the pipe at the lowest FS is shown in Figure 9 determined at a river elevation of 223.10m.

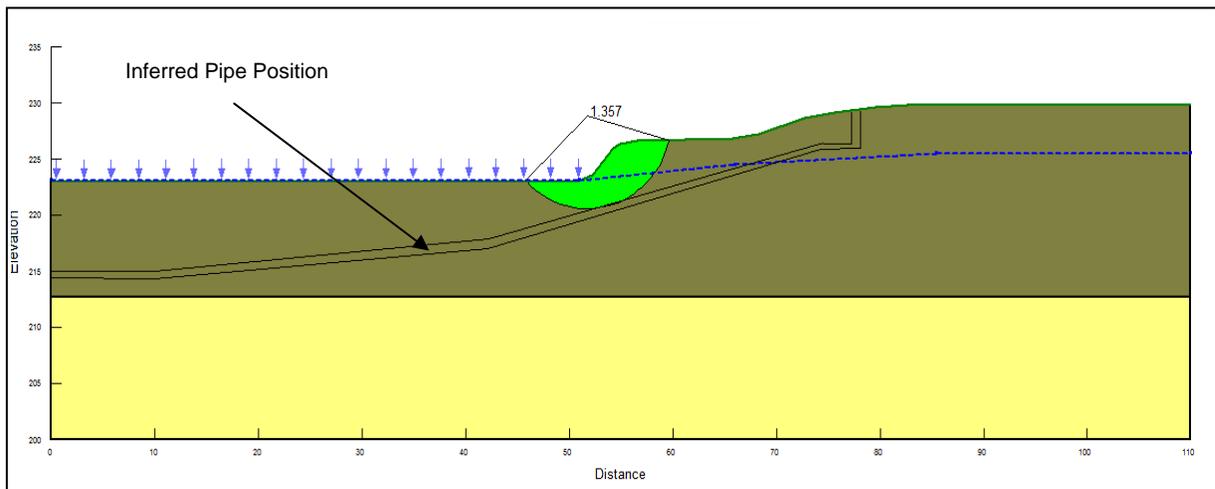


Figure 9: Site 11, Global Slip Surface Engaging the Pipe (GS+P)

1.3.5 Site 14: St James Street Watermain (Assiniboine River) - South Bank

Bathymetric survey information was not available for the crossing location; therefore the channel depth and profile have been assumed based on field observations and measurements made during construction of the pipe crossing. In the absence of record drawings, these approximate field measurements were also used to locate the pipe position and invert within the slope stability models in order to assess the GS+P.

The calculated FS for the critical case and the toe case are equal, and have been calculated as less than 0.8. These failure surfaces are largely a result of the near vertical face at the toe of the riverbank. The FS for global instability has been calculated between 0.8 and 0.9, and the calculated FS for slip surfaces engaging the pipe has been calculated between 1.1 and 1.2. The results of the analysis are presented graphically in Figure 10.

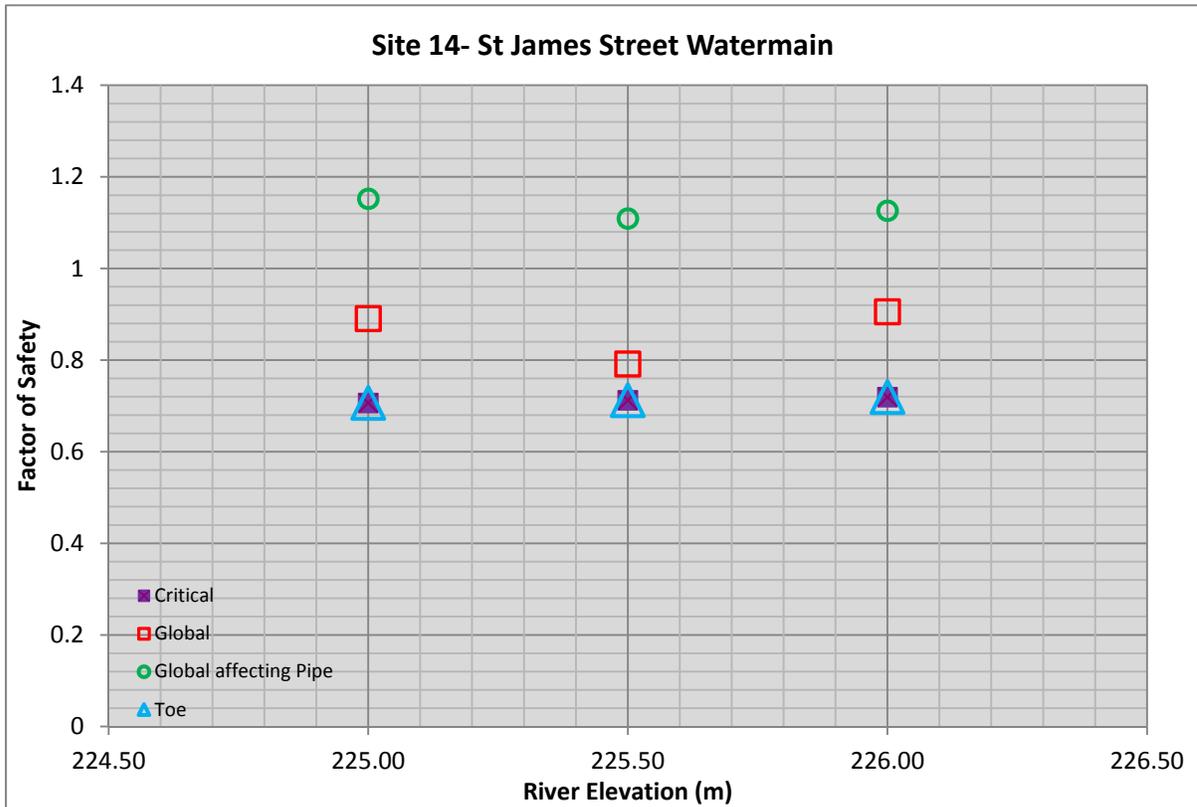


Figure 10: Slope Stability Factor of Safety Relating to the Current Conditions of Site 14

A stability output model illustrating the position of the global slip surface engaging the pipe at the lowest FS is shown in Figure 11, determined at a river elevation of 225.50 m.

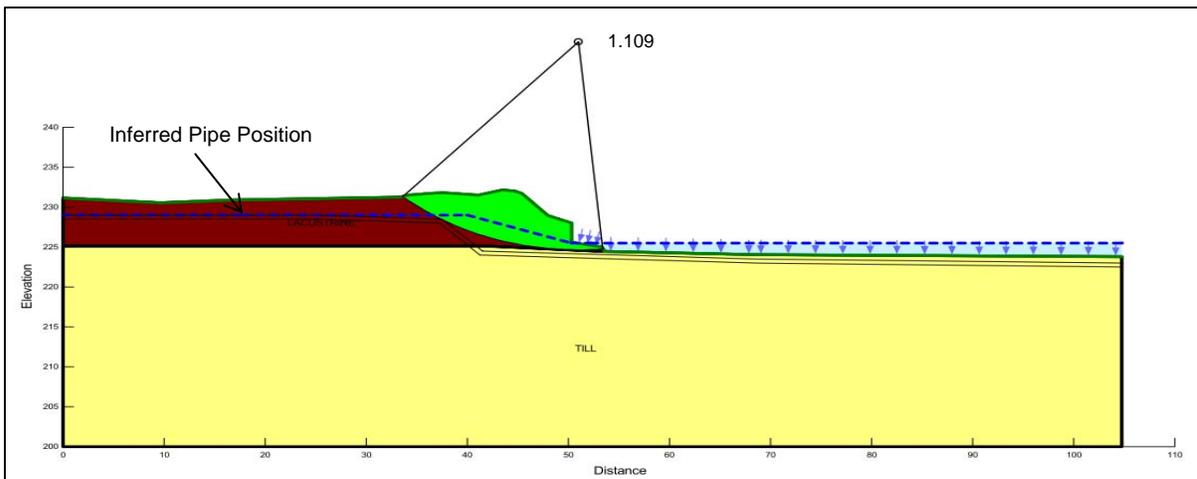


Figure 11: Site 14, Global Slip Surface Engaging the Pipe (GS+P)

1.4 Summary

The calculated stability results from each pipe crossing discussed in previous sections has been presented graphically in Figure 12 and summarized in Table 4. Table 4 presents the sites according to a relative ranking system considering the reported site observations, slope geometries and the calculated FS for GS and GS+P slip surfaces. The relative ranking system is structured as No.1 is the relatively higher risk site and No.5 is the relatively lower risk site among the considered five sites.

Table 4: Ranked Riverbank Slopes

Ranking	Site Location	River Elevation (m)	Global Stability	Global Stability Affecting Pipe	Comments
1	Site 14- St James Street Watermain	225.00- 226.00	0.8- 0.9	1.1- 1.2	Given the relatively low FS values, combined with an absence of accurate and detailed information about channel profile and pipe location, Site 14 should be considered a high priority.
3	Site 5- Assiniboine Park Siphon	225.90- 227.00	0.9- 1.0	0.9- 1.0	Presence of slump block near toe of slope indicating recent movement within the riverbank slope, and potential for future movement
4	Site 11- Kildonan Redwood Feedermain	223.10- 224.50	1.1- 1.2	1.4- 1.6	Extensive toe erosion noted during field inspection; however slope instability features further up slope have not been identified.
2	Site 4- St James Interceptor	227.75- 228.75	1.1- 1.2	1.1- 1.2	Oversteepened river bank slopes for given alluvial soils resulting in relative low FS values. However limited evidence of slope instability has been noted along the slope. The slope is considered to be marginally stable.
5	Site 6- Munroe Polson Siphon	221.43- 225.00	1.3- 1.4	1.3 1.4	No evidence of slope instability or movement observed directly along the slope at pipe crossing.

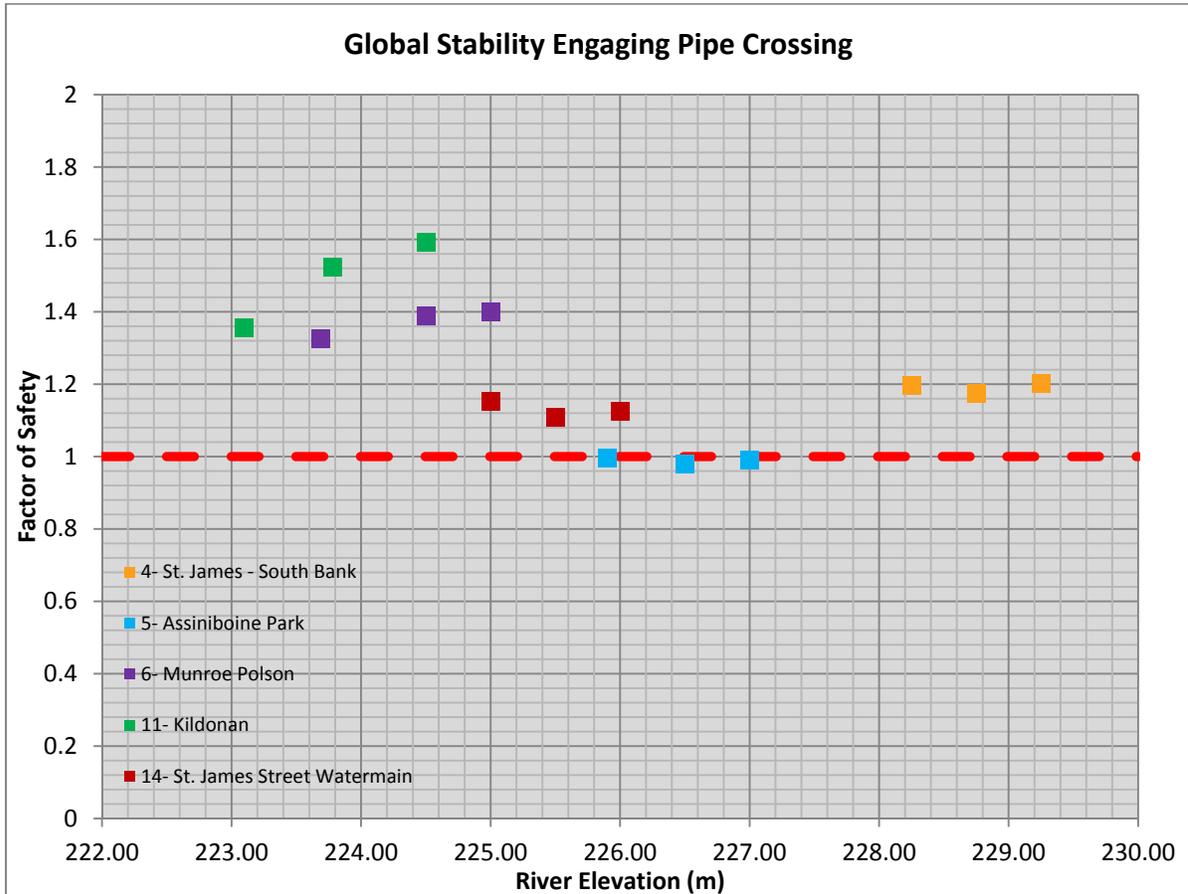


Figure 12: Global Stability Engaging Pipe Crossing

1.5 Conclusions and Recommendations

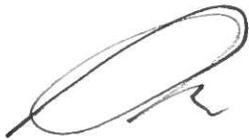
Based on the results of preliminary slope stability assessment for the five riverbank locations, the following can be concluded;

- Analysis has been performed based on limited topographical information, assumed soils data, and approximate pipe invert and positional information. The results should therefore be viewed as preliminary;
- Failure surfaces which engage the pipe have been determined from slope stability analysis, with three out of the five sites having FS values near to 1.0, with the remaining two sites having an FS of between 1.3 and 1.4;

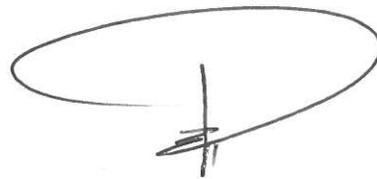
- Based on the results of the preliminary slope stability analysis and field observations, collectively the crossing locations have been ranked based on the potential risk to the integrity of the pipe beneath the crossing and overall stability of the riverbank. This exercise allows for a more proactive approach in terms of asset management.

If you have any questions, please do not hesitate to contact the undersigned.

Respectfully submitted,



Alexander Hill, B.Sc.
Geotechnical Engineering



Zeyad Shukri Al-Hayazai, M.Sc., P. Eng.
Senior Geotechnical Engineer

**APPENDIX A
TABULATED STABILITY ANALYSIS RESULTS**

Condition Assessment of High Risk Water and Wastewater River Crossings

City of Winnipeg

Stability Analysis

Project Number: 60270487

Soil Units:

Soil	γ (kN/m ³)	ϕ	C (kPa)
Intact Alluvial	17	26	0
Residual Alluvial	-	-	-
Intact Lacustrine	17	17	5
Residual Lacustrine	-	-	-
Till	21	30	10

√ Notes
Indicates specific application of geometric model

Stability Results

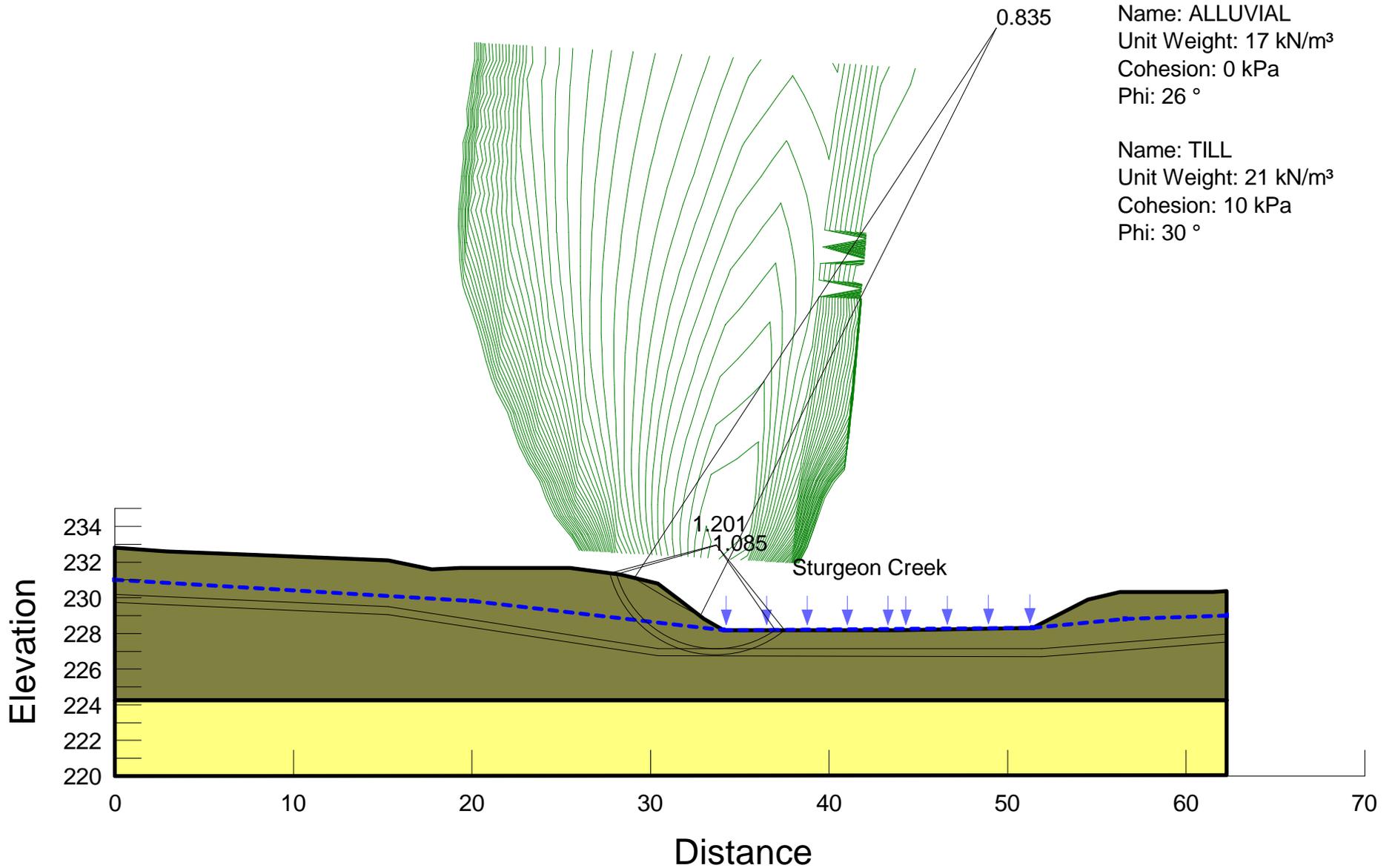
Stability Case	Till Head (m)	River Level (m)	SCARP PRESENT ON ALIGNMENT	RIP RAP AT RIVER BANK TOE	LOWER BANK INSTABILITIES EVIDENT	Pipe Layout	Geometry Cross Section			Grid-Radius							File Name		
							As-Built Drawings	Lidar Survey	Bathymetric Survey	Critical	Slip #	Global	Slip #	Global affecting Pipe	Slip #	Toe		Slip #	
4- St. James - South Bank																			
Case 2-1 : Site # 4 - South Bank	224.25	228.25	Yes	Yes	Yes	Yes			√		0.805	11468	1.058	337	1.196	338	NP	NP	4-ST James at Sturgeon Creek- South Side - 002
Case 2-2 : Site # 4 - South Bank	224.25	228.75	Yes	Yes	Yes	Yes			√		0.831	14071	1.175	988	1.175	988	NP	NP	4-ST James at Sturgeon Creek- South Side - 002
Case 2-3 : Site # 4 - South Bank	224.25	229.25	Yes	Yes	Yes	Yes			√		0.978	19976	1.085	988	1.201	989	NP	NP	4-ST James at Sturgeon Creek- South Side - 002
5- Assiniboine Park																			
Case 5-1 : Site # 5 - South Bank	224.0	225.90	Yes	No	Yes	Yes	√	√			0.866	1021	0.995	10461	0.996	10462	0.866	1021	5-Assiniboine Park - South Side - 005
Case 5-2 : Site # 5 - South Bank	224.0	226.50	Yes	No	Yes	Yes	√	√			0.839	1021	0.977	12041	0.980	12042	0.839	1021	5-Assiniboine Park - South Side - 005
Case 5-3 : Site # 5 - South Bank	224.0	227.00	Yes	No	Yes	Yes	√	√			0.918	1021	0.992	10462	0.992	10462	0.918	1021	5-Assiniboine Park - South Side - 005
6- Munroe Polson																			
Case 6-1 : Site # 6 - East Bank	217.01-215.33	223.69	No	Yes	No	Yes			√		1.326	32348	1.326	32348	1.326	32348	NP	NP	6-Munroe - East Side - 006
Case 6-2 : Site # 6 - East Bank	217.01-215.33	224.50	No	Yes	No	Yes			√		1.390	30820	1.390	30820	1.390	30820	NP	NP	6-Munroe - East Side - 006
Case 6-3 : Site # 6 - East Bank	217.01-215.33	225.00	No	Yes	No	Yes			√		1.401	30818	1.401	30818	1.401	30819	NP	NP	6-Munroe - East Side - 006
11- Kildonan																			
Case 1-1 : Site # 11 - East Bank	212.7	223.10	Yes	Yes	Yes	No			√		0.542	46174	1.059	715	1.357	719	0.796	44730	11- Kildonan - East Side - 002
Case 1-2 : Site # 11 - East Bank	212.7	223.78	Yes	Yes	Yes	No			√		0.525	35153	1.195	2245	1.525	3830	0.65	30501	11- Kildonan - East Side - 002
Case 1-3 : Site # 11 - East Bank	212.7	224.50	Yes	Yes	Yes	No			√		0.493	13212	1.088	2245	1.593	718	0.676	35247	11- Kildonan - East Side - 002
14- St. James Street Watermain																			
										Channel Elevation (m)									
Case 4-1 : Site # 14 - South Bank	225.14-223.78	225.00	Yes	Yes	Yes	Yes			√	224	0.706	21370	0.890	7039	1.152	21218	0.706	21370	14- St James Street Watermain - South Side - 004
Case 4-2 : Site # 14 - South Bank	225.14-223.78	225.50	Yes	Yes	Yes	Yes			√	224	0.712	27746	0.791	3877	1.109	27542	0.712	27746	14- St James Street Watermain - South Side - 004
Case 4-3 : Site # 14 - South Bank	225.14-223.78	226.00	Yes	Yes	Yes	Yes			√	224	0.719	38865	0.905	5458	1.126	18056	0.719	38865	14- St James Street Watermain - South Side - 004

**APPENDIX B
FIGURES OF STABILTY ANALYSIS**

Condition Assessment of High Risk Water and Wastewater River Crossings

Title: Site 4- St James (South Bank)

Name: Case 2-1



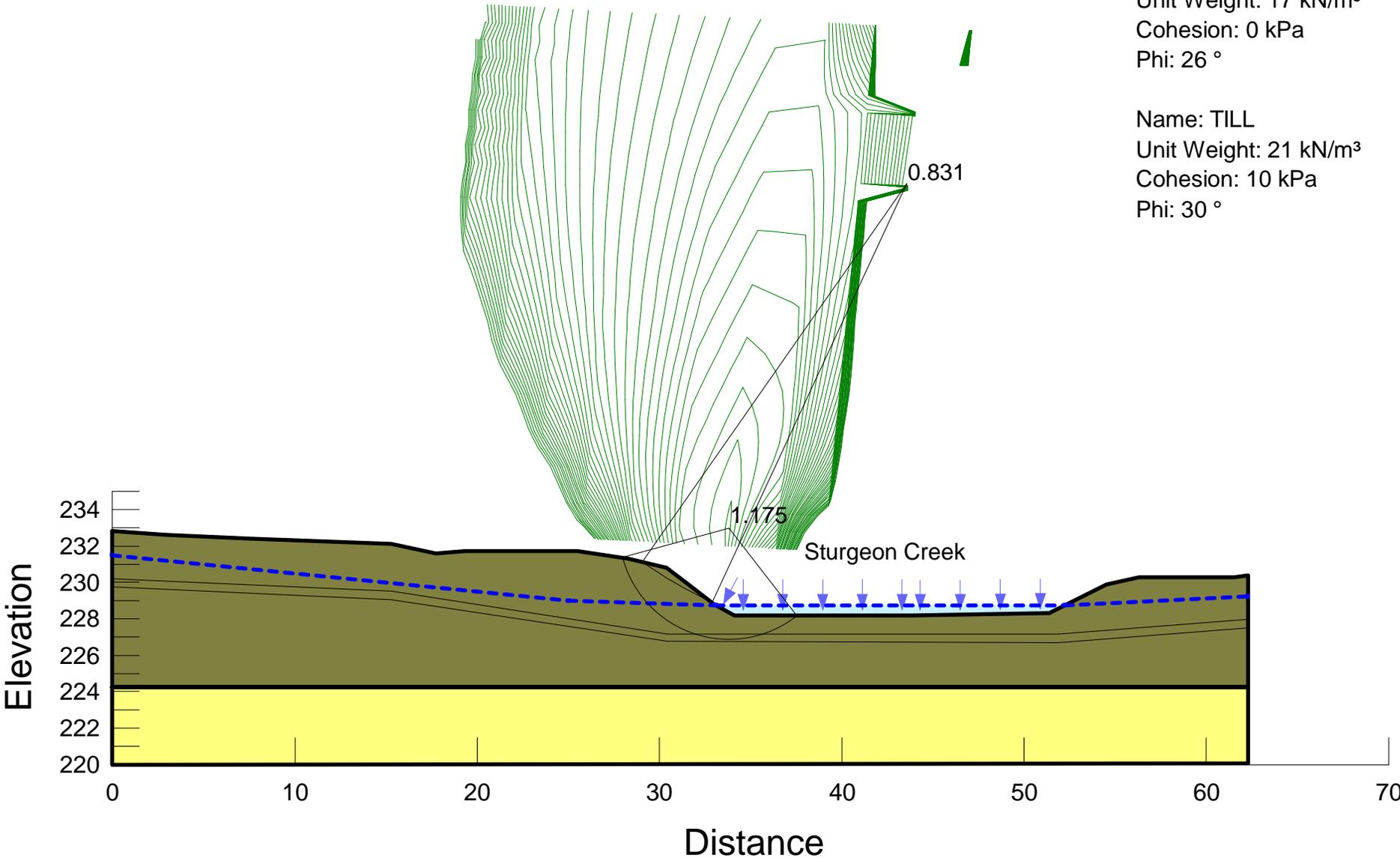
Condition Assessment of High Risk Water and Wastewater River Crossings

Title: Site 4- St James (South Bank)

Name: Case 2-2

Name: ALLUVIAL
Unit Weight: 17 kN/m³
Cohesion: 0 kPa
Phi: 26 °

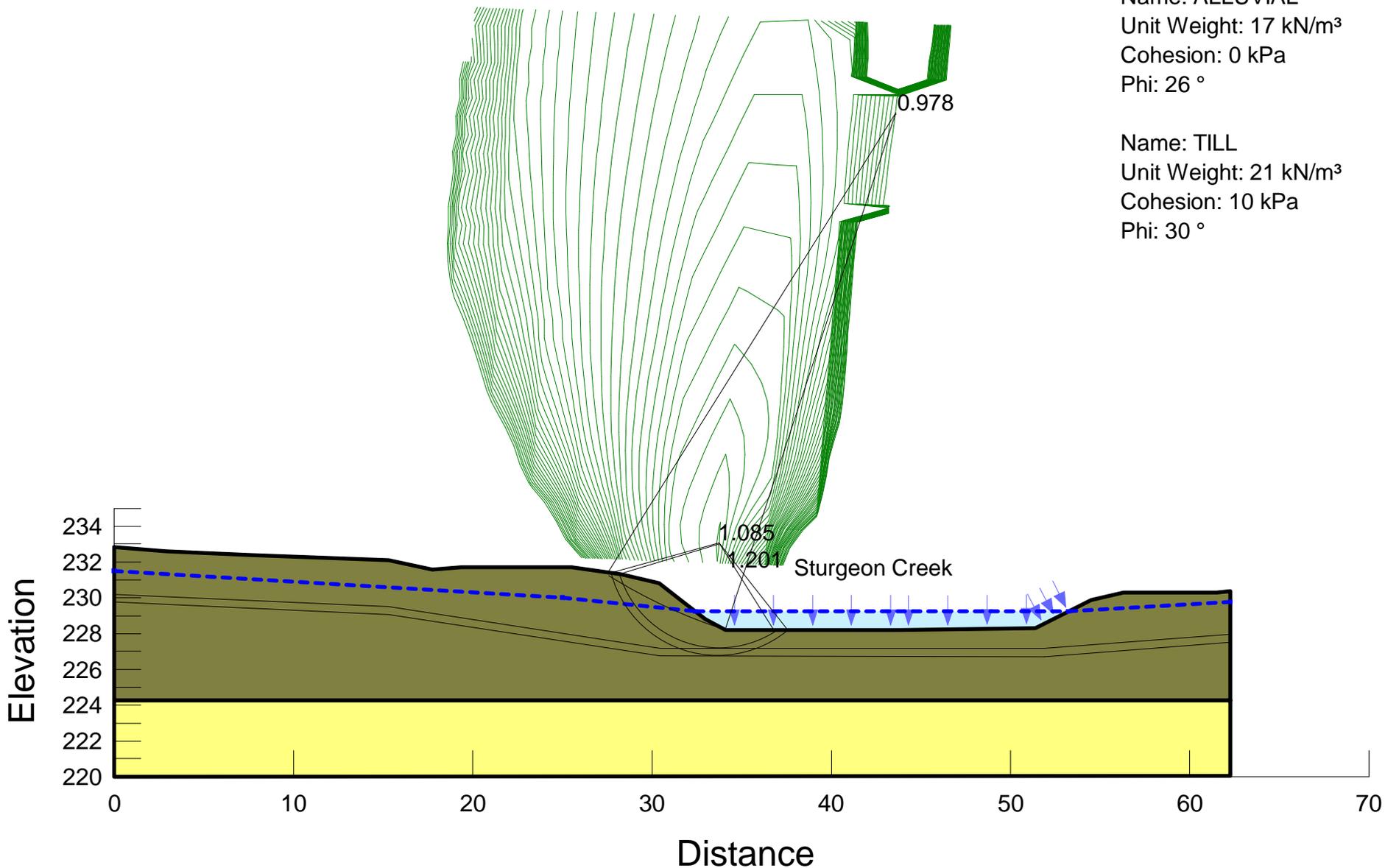
Name: TILL
Unit Weight: 21 kN/m³
Cohesion: 10 kPa
Phi: 30 °



Condition Assessment of High Risk Water and Wastewater River Crossings
Title: Site 4- St James (South Bank)
Name: Case 2-3

Name: ALLUVIAL
Unit Weight: 17 kN/m³
Cohesion: 0 kPa
Phi: 26 °

Name: TILL
Unit Weight: 21 kN/m³
Cohesion: 10 kPa
Phi: 30 °



Condition Assessment of High Risk Water and Wastewater River Crossings

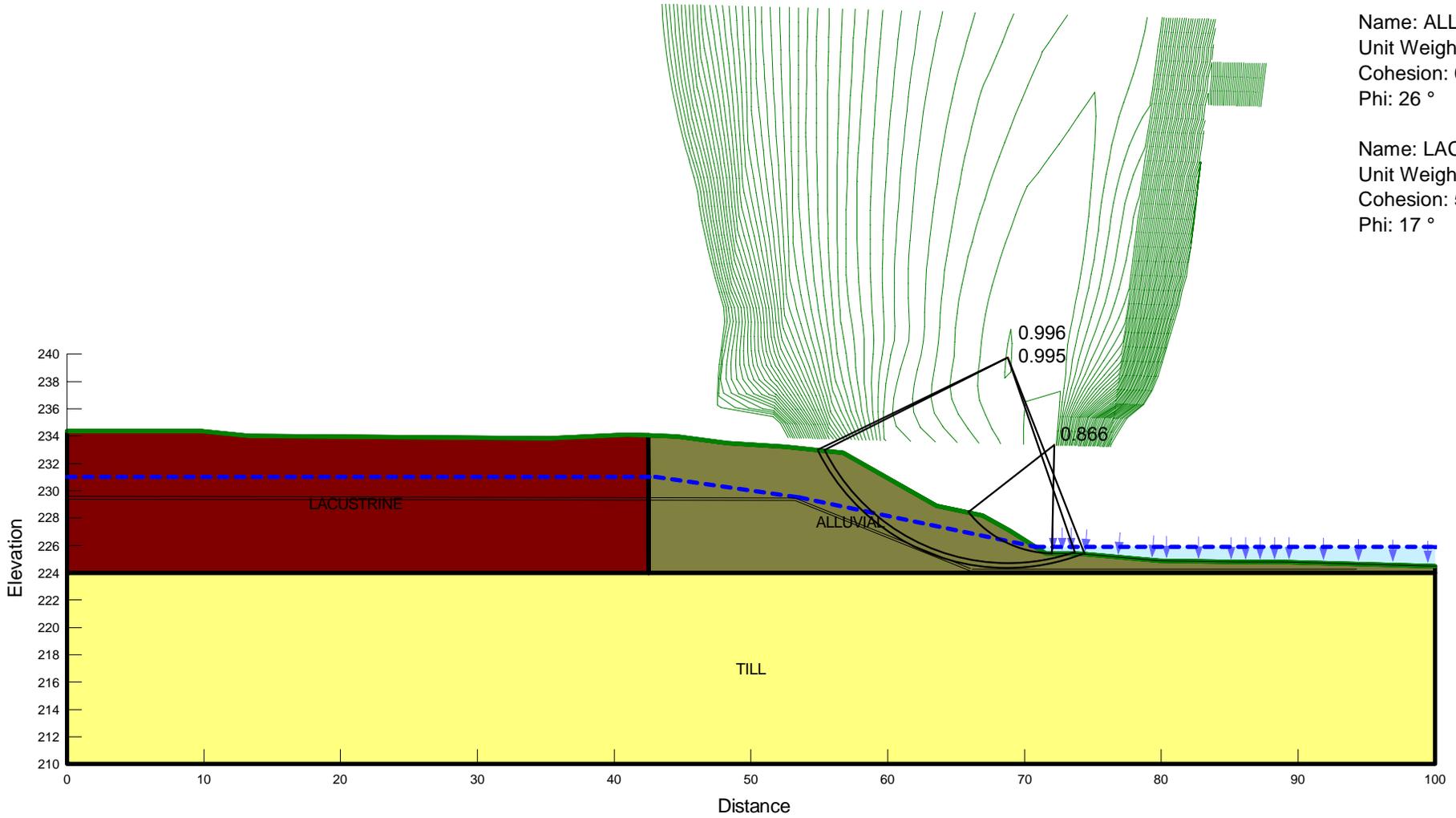
Title: Site 5- Assiniboine Park Siphon

Name: Case 5-1

Name: TILL
Unit Weight: 21 kN/m³
Cohesion: 10 kPa
Phi: 30 °

Name: ALLUVIAL
Unit Weight: 17 kN/m³
Cohesion: 0 kPa
Phi: 26 °

Name: LACUSTRINE
Unit Weight: 17 kN/m³
Cohesion: 5 kPa
Phi: 17 °



Condition Assessment of High Risk Water and Wastewater River Crossings

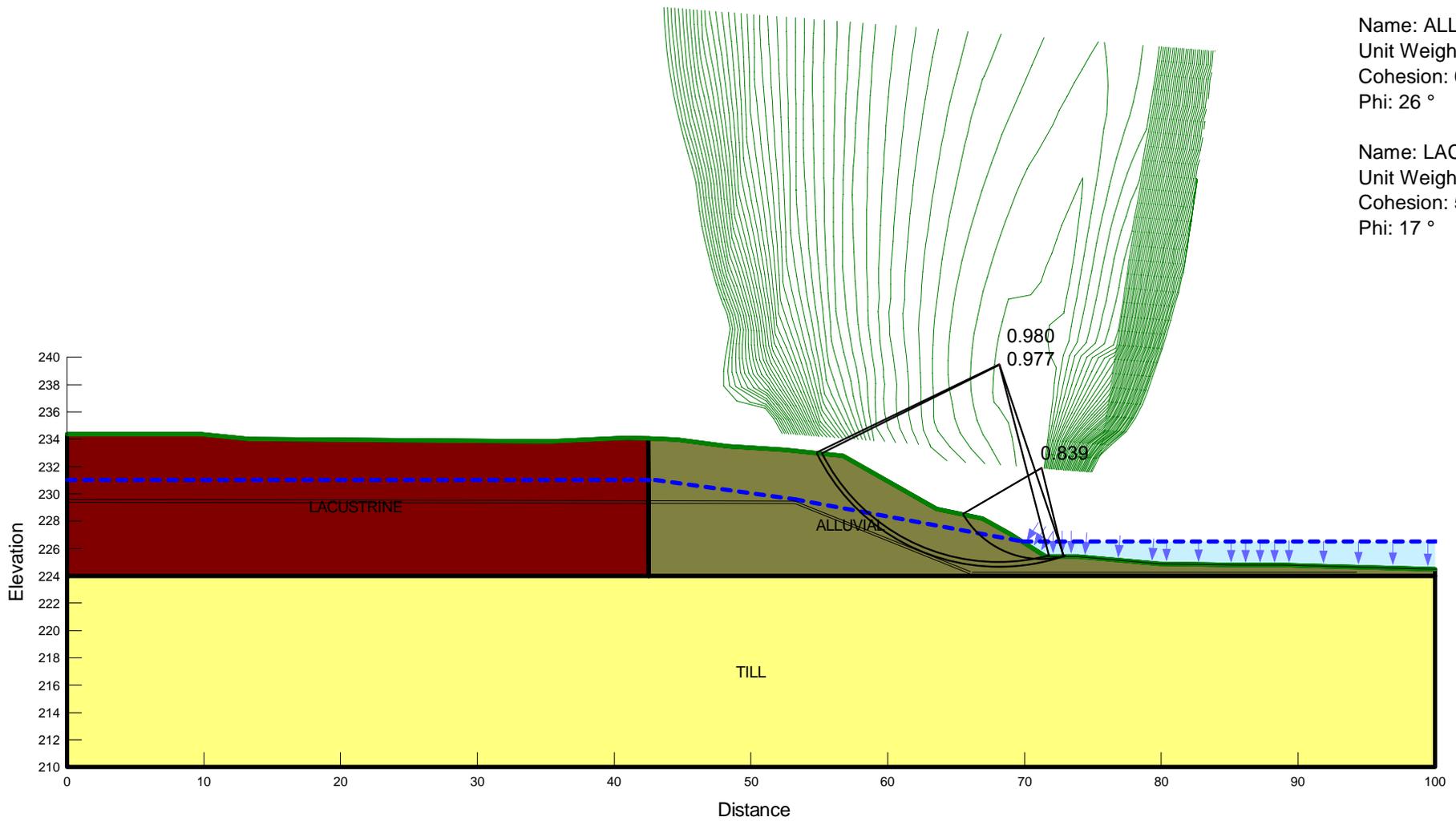
Title: Site 5- Assiniboine Park Siphon

Name: Case 5-2

Name: TILL
Unit Weight: 21 kN/m³
Cohesion: 10 kPa
Phi: 30 °

Name: ALLUVIAL
Unit Weight: 17 kN/m³
Cohesion: 0 kPa
Phi: 26 °

Name: LACUSTRINE
Unit Weight: 17 kN/m³
Cohesion: 5 kPa
Phi: 17 °



Condition Assessment of High Risk Water and Wastewater River Crossings

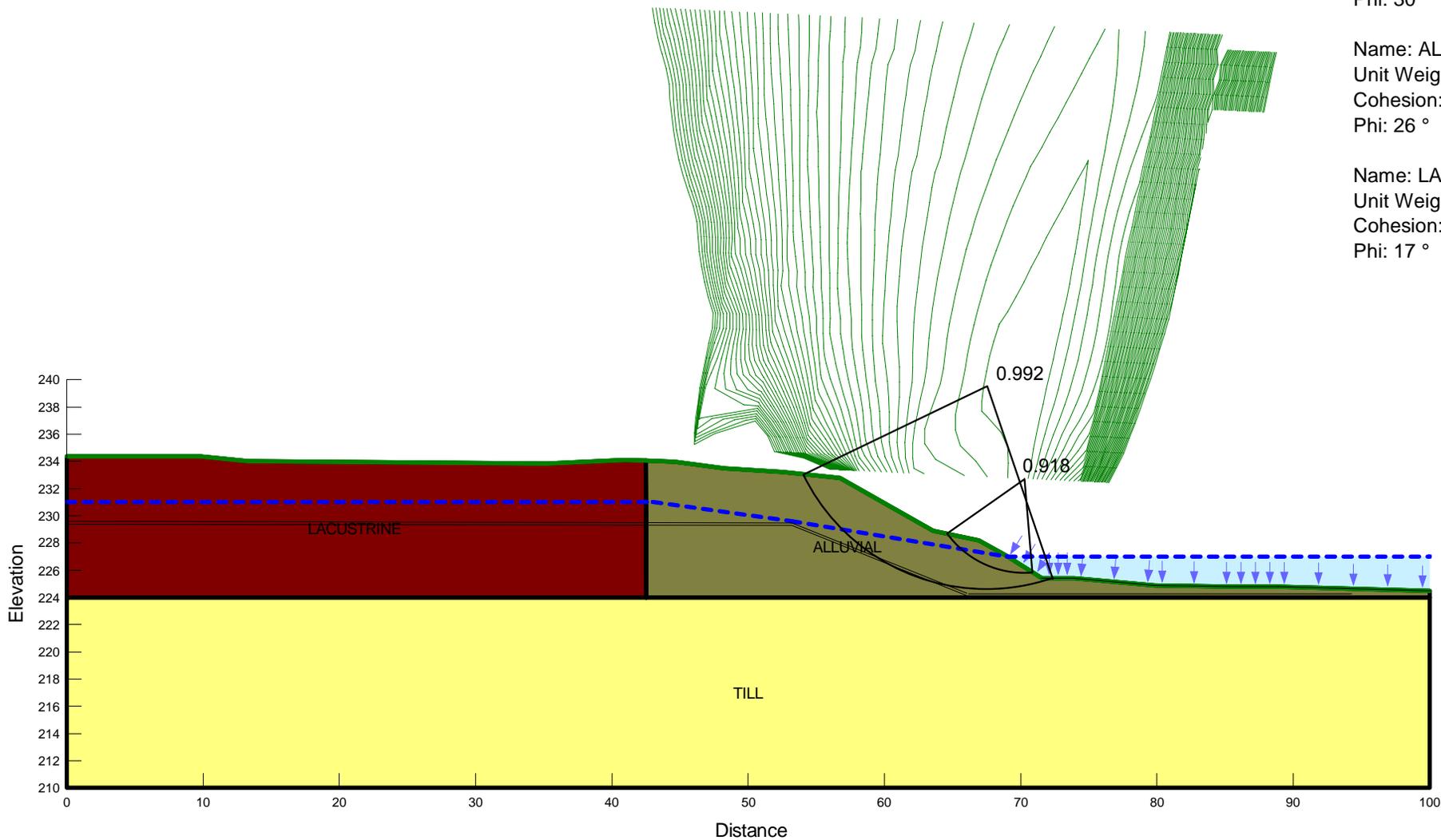
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Name: Case 5-3

Name: TILL
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Cohesion: 10 kPa
Phi: 30 °

Name: ALLUVIAL
Unit Weight: 17 kN/m³
Cohesion: 0 kPa
Phi: 26 °

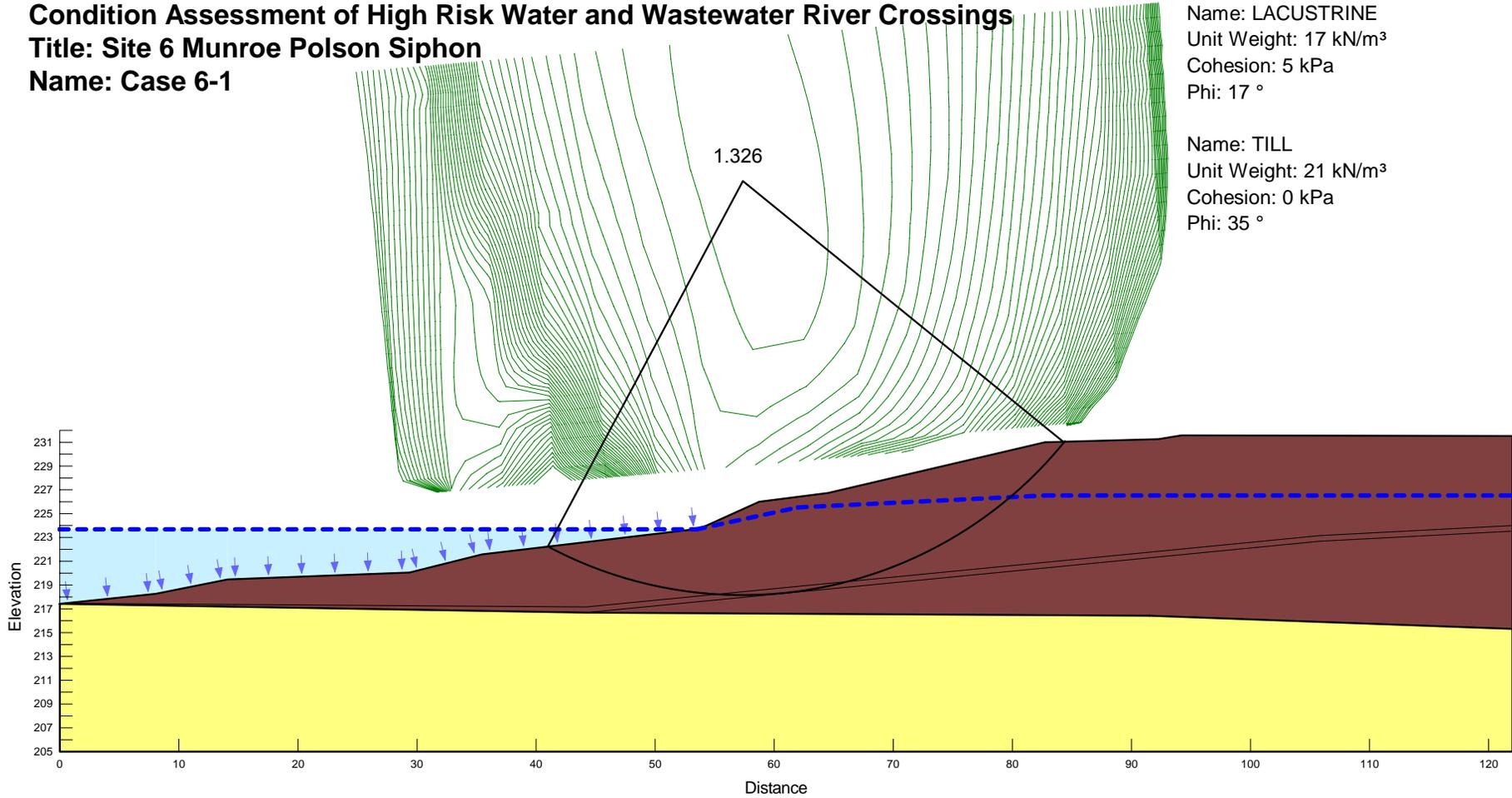
Name: LACUSTRINE
Unit Weight: 17 kN/m³
Cohesion: 5 kPa
Phi: 17 °



Condition Assessment of High Risk Water and Wastewater River Crossings
Title: Site 6 Munroe Polson Siphon
Name: Case 6-1

Name: LACUSTRINE
Unit Weight: 17 kN/m³
Cohesion: 5 kPa
Phi: 17 °

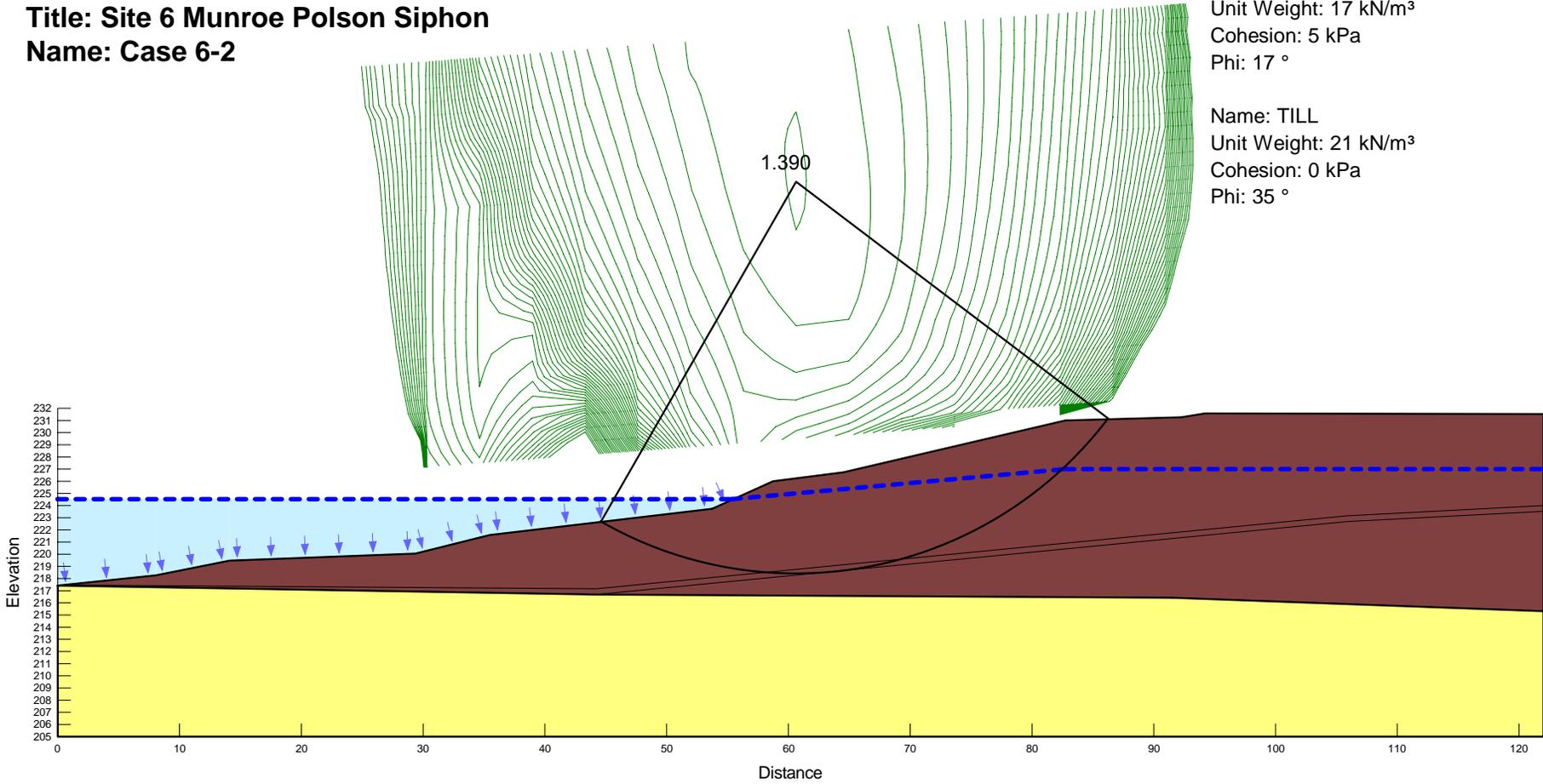
Name: TILL
Unit Weight: 21 kN/m³
Cohesion: 0 kPa
Phi: 35 °



Condition Assessment of High Risk Water and Wastewater River Crossings
Title: Site 6 Munroe Polson Siphon
Name: Case 6-2

Name: LACUSTRINE
Unit Weight: 17 kN/m³
Cohesion: 5 kPa
Phi: 17 °

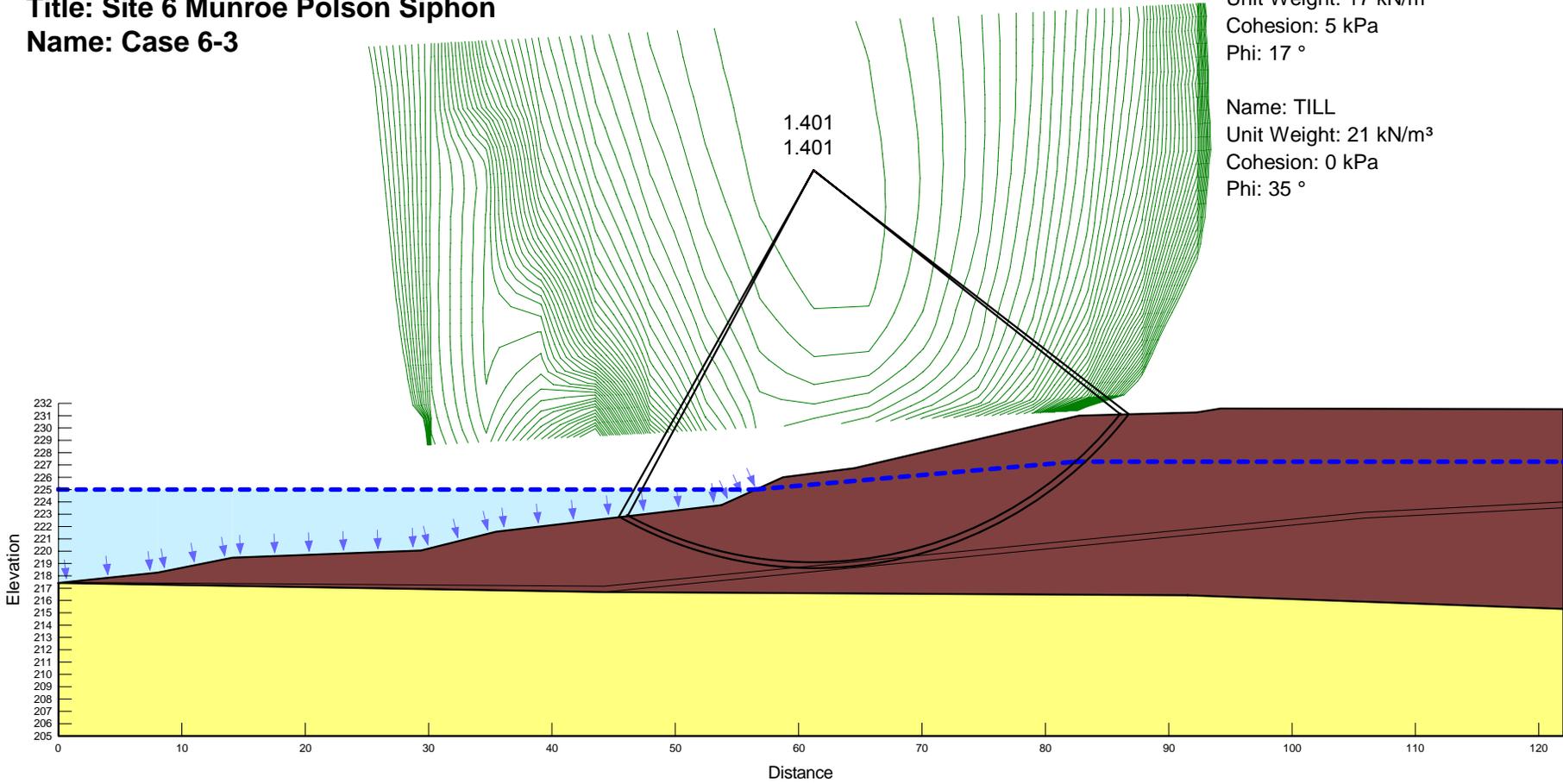
Name: TILL
Unit Weight: 21 kN/m³
Cohesion: 0 kPa
Phi: 35 °



Condition Assessment of High Risk Water and Wastewater River Crossings
Title: Site 6 Munroe Polson Siphon
Name: Case 6-3

Name: LACUSTRINE
Unit Weight: 17 kN/m³
Cohesion: 5 kPa
Phi: 17 °

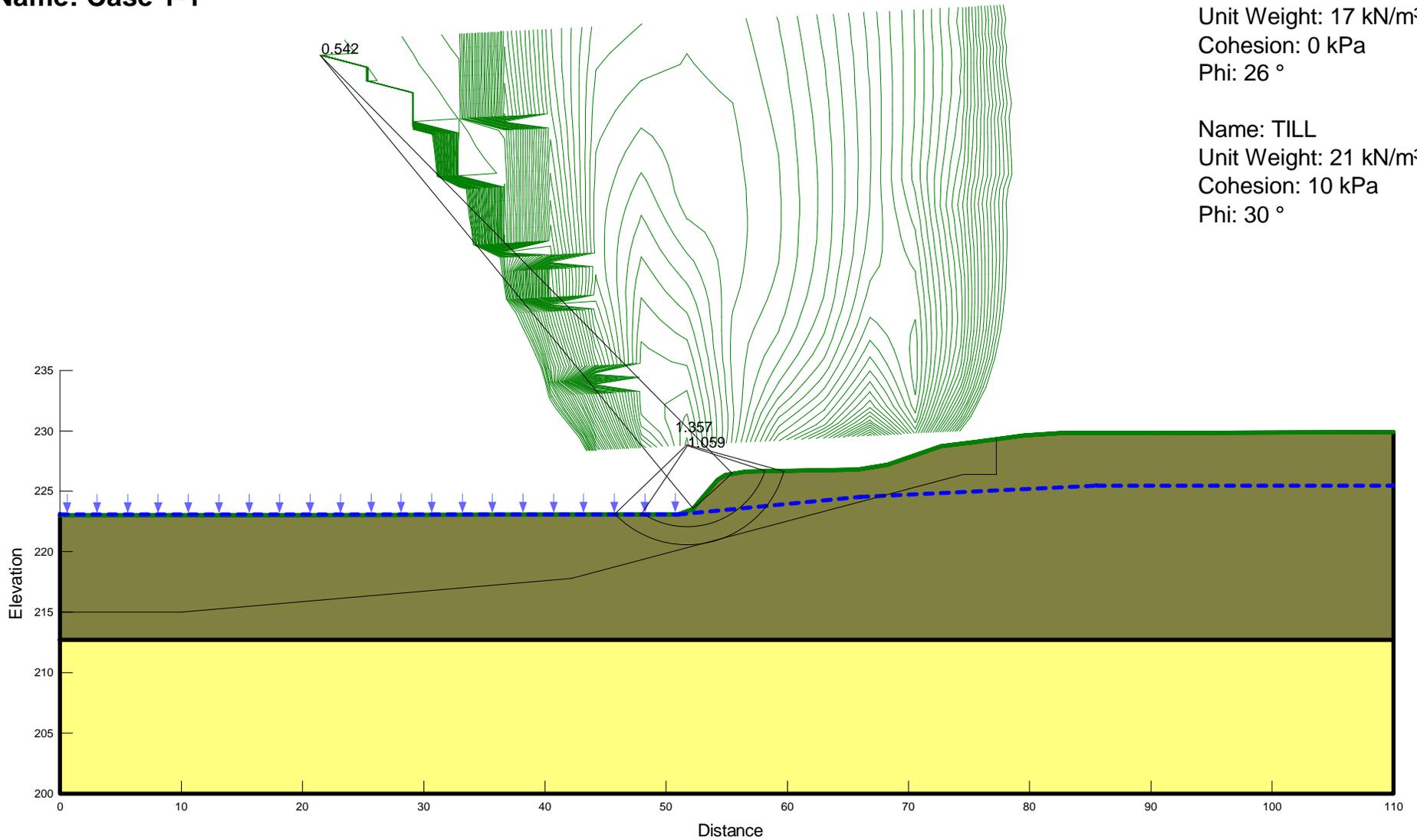
Name: TILL
Unit Weight: 21 kN/m³
Cohesion: 0 kPa
Phi: 35 °



Condition Assessment of High Risk Water and Wastewater River Crossings
Title: Site 11 Kildonan Redwood Feedermain
Name: Case 1-1

Name: ALLUVIAL
Unit Weight: 17 kN/m³
Cohesion: 0 kPa
Phi: 26 °

Name: TILL
Unit Weight: 21 kN/m³
Cohesion: 10 kPa
Phi: 30 °



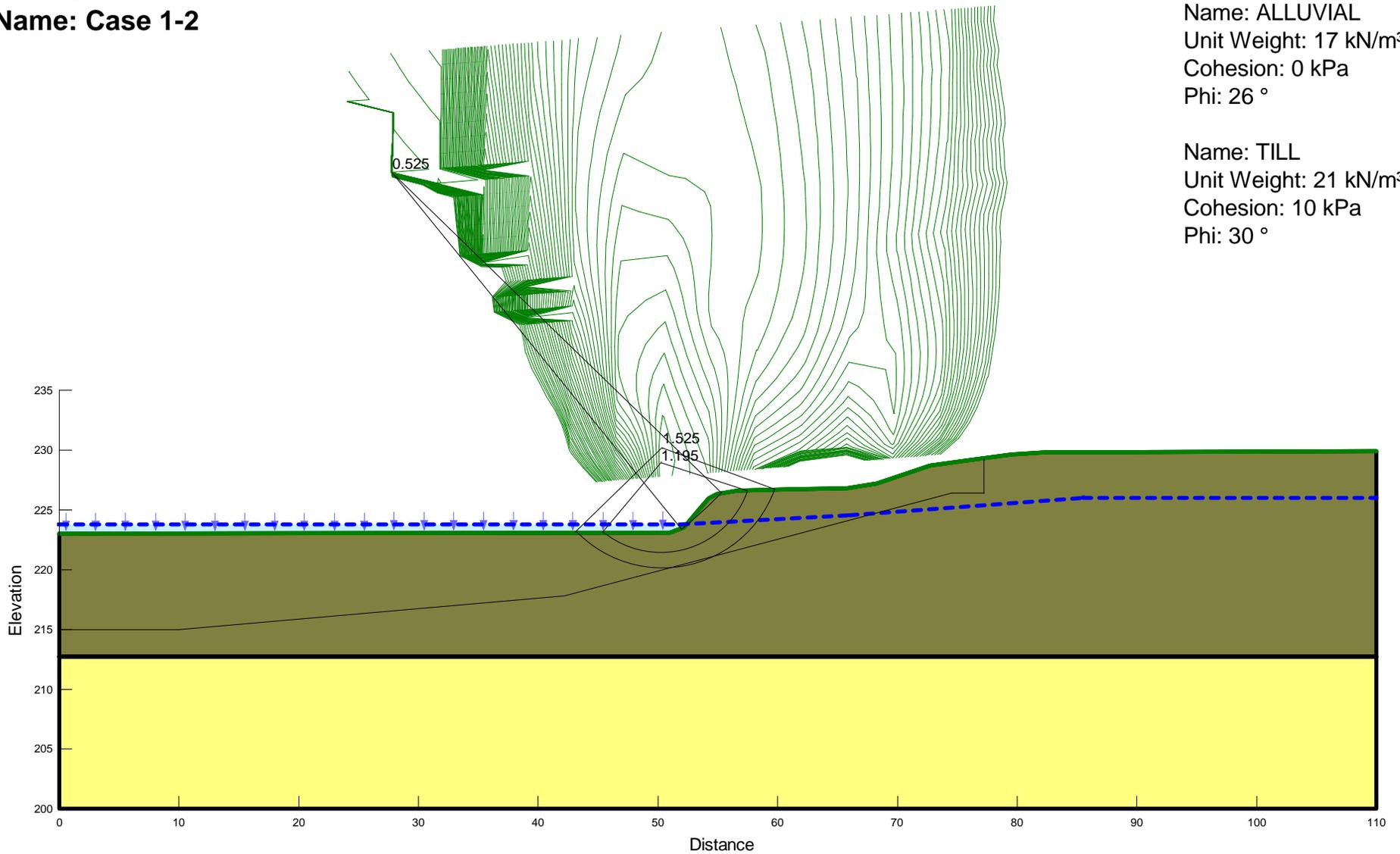
Condition Assessment of High Risk Water and Wastewater River Crossings

Title: Site 11 Kildonan Redwood Feedermain

Name: Case 1-2

Name: ALLUVIAL
Unit Weight: 17 kN/m³
Cohesion: 0 kPa
Phi: 26 °

Name: TILL
Unit Weight: 21 kN/m³
Cohesion: 10 kPa
Phi: 30 °



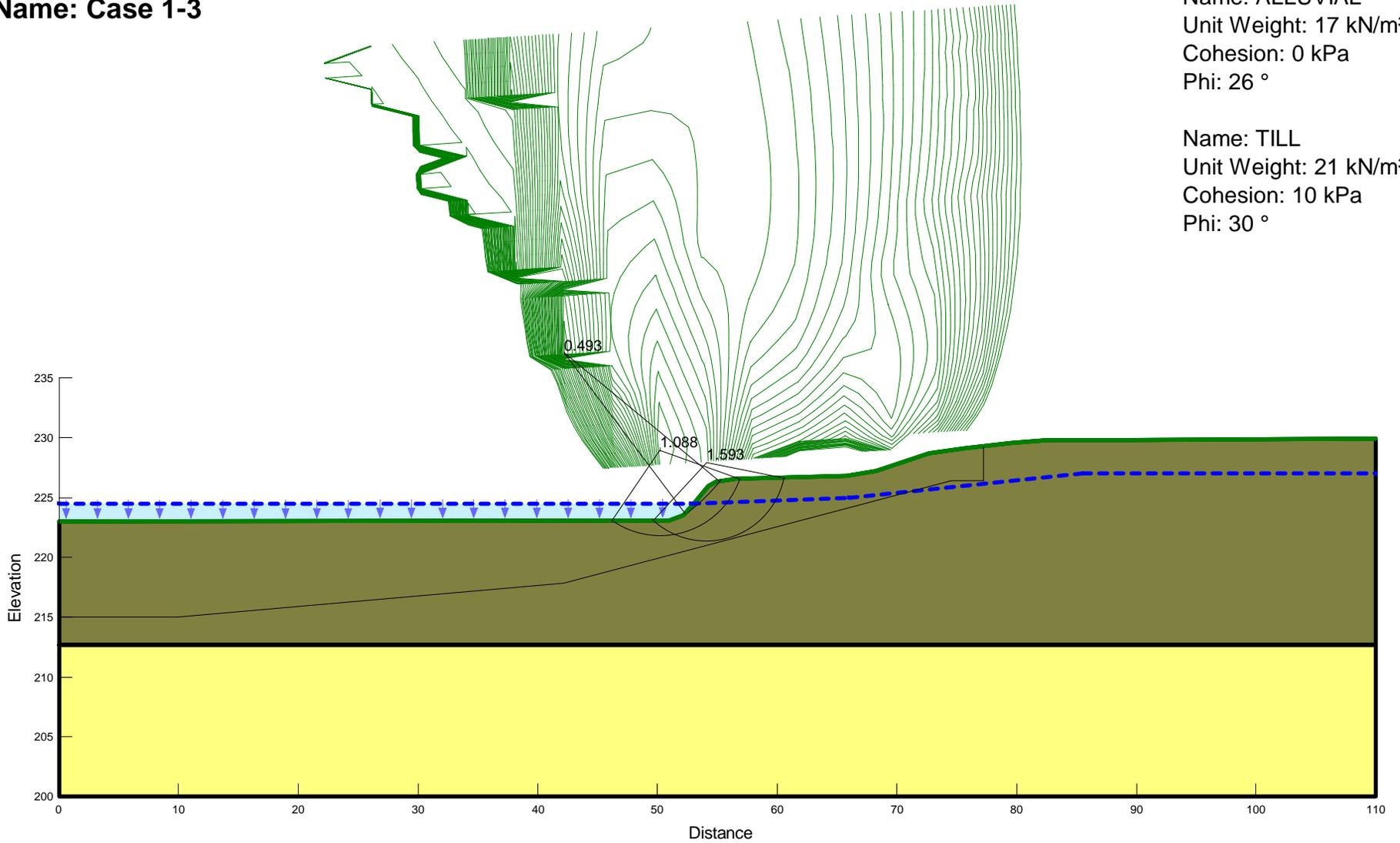
Condition Assessment of High Risk Water and Wastewater River Crossings

Title: Site 11 Kildonan Redwood Feedermain

Name: Case 1-3

Name: ALLUVIAL
Unit Weight: 17 kN/m³
Cohesion: 0 kPa
Phi: 26 °

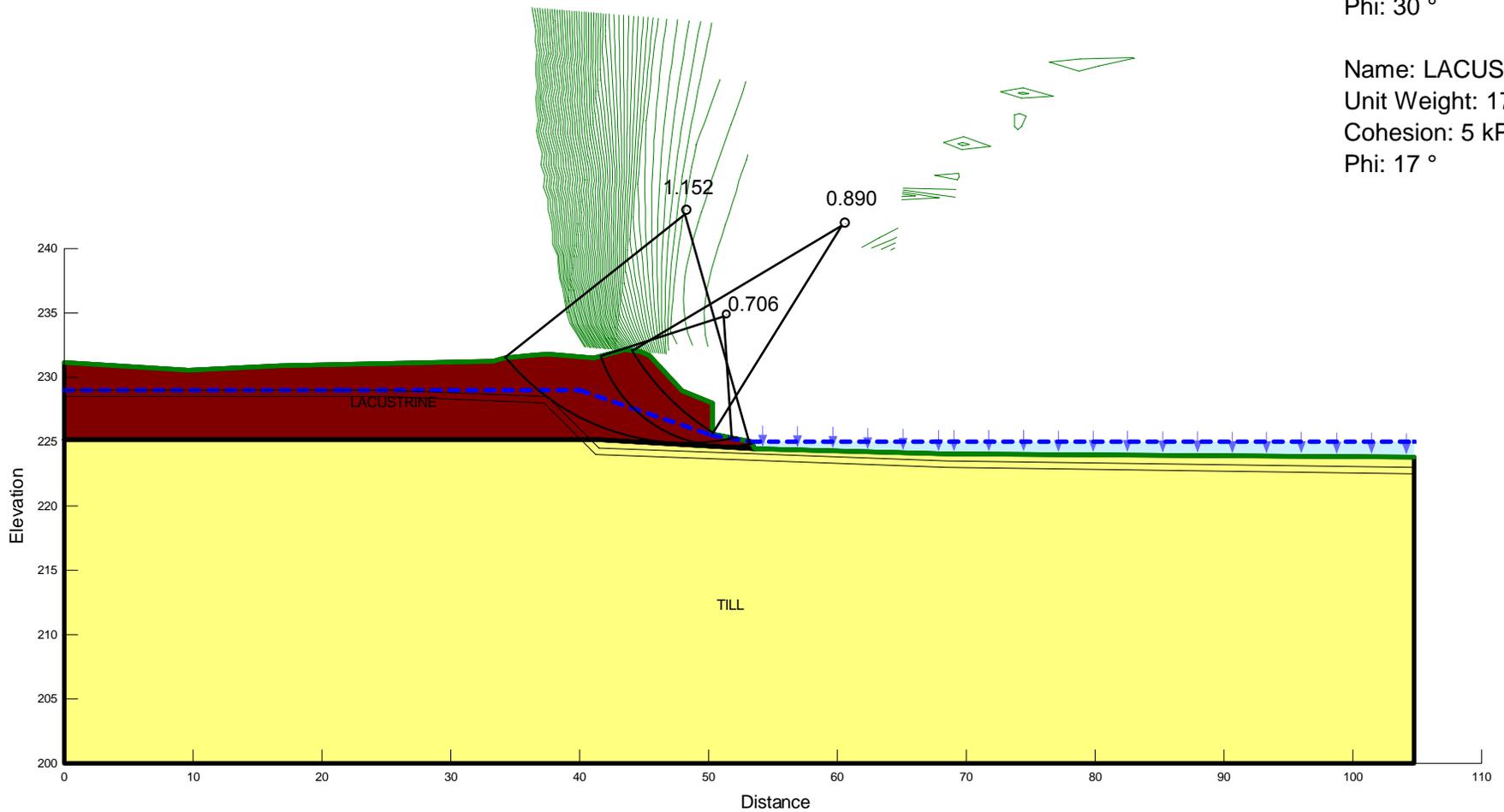
Name: TILL
Unit Weight: 21 kN/m³
Cohesion: 10 kPa
Phi: 30 °



Condition Assessment of High Risk Water and Wastewater River Crossings
Title: Site 14 St James Street Watermain
Name: Case 4-1

Name: TILL
Unit Weight: 21 kN/m³
Cohesion: 10 kPa
Phi: 30 °

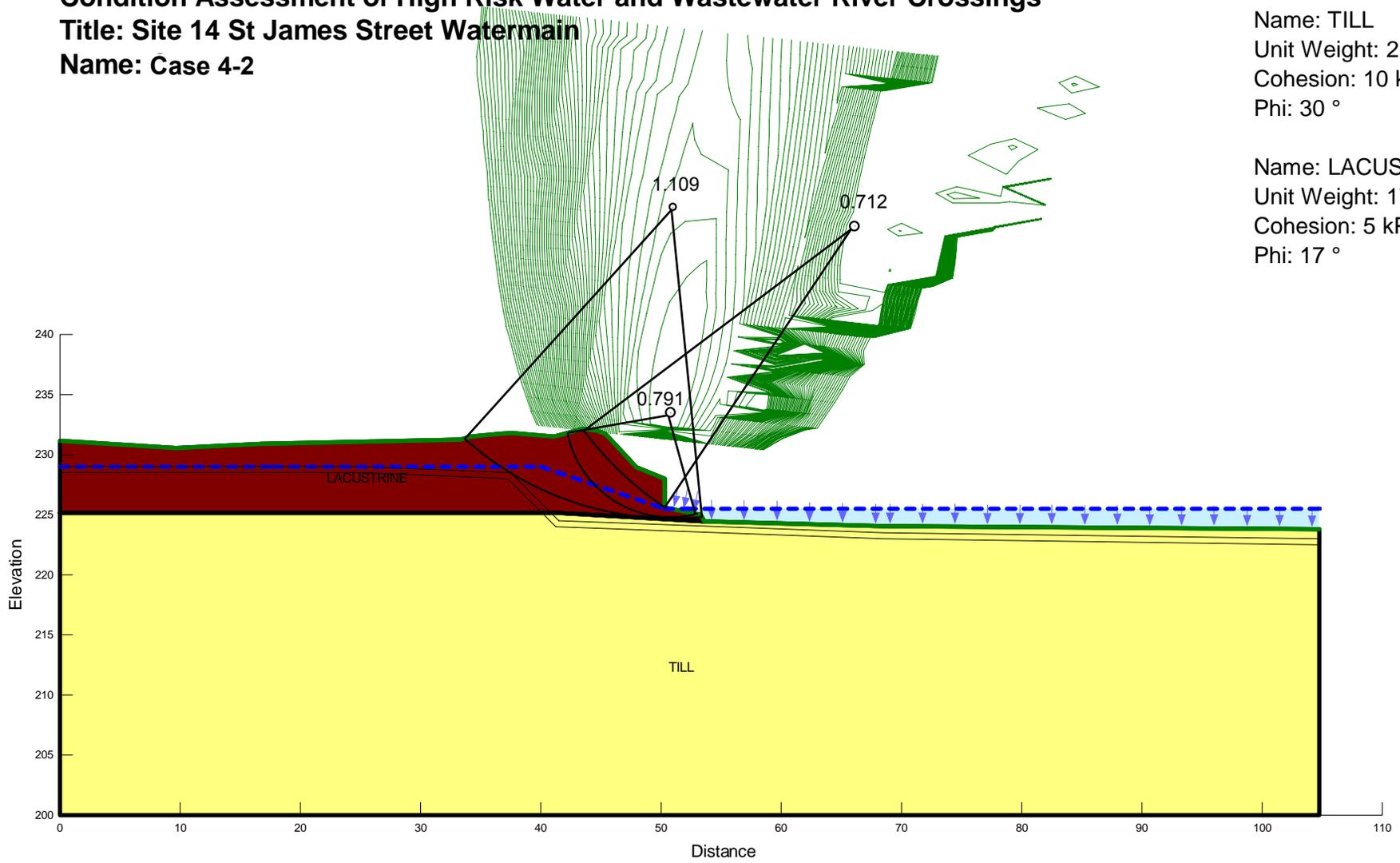
Name: LACUSTRINE
Unit Weight: 17 kN/m³
Cohesion: 5 kPa
Phi: 17 °



Condition Assessment of High Risk Water and Wastewater River Crossings
Title: Site 14 St James Street Watermain
Name: Case 4-2

Name: TILL
Unit Weight: 21 kN/m³
Cohesion: 10 kPa
Phi: 30 °

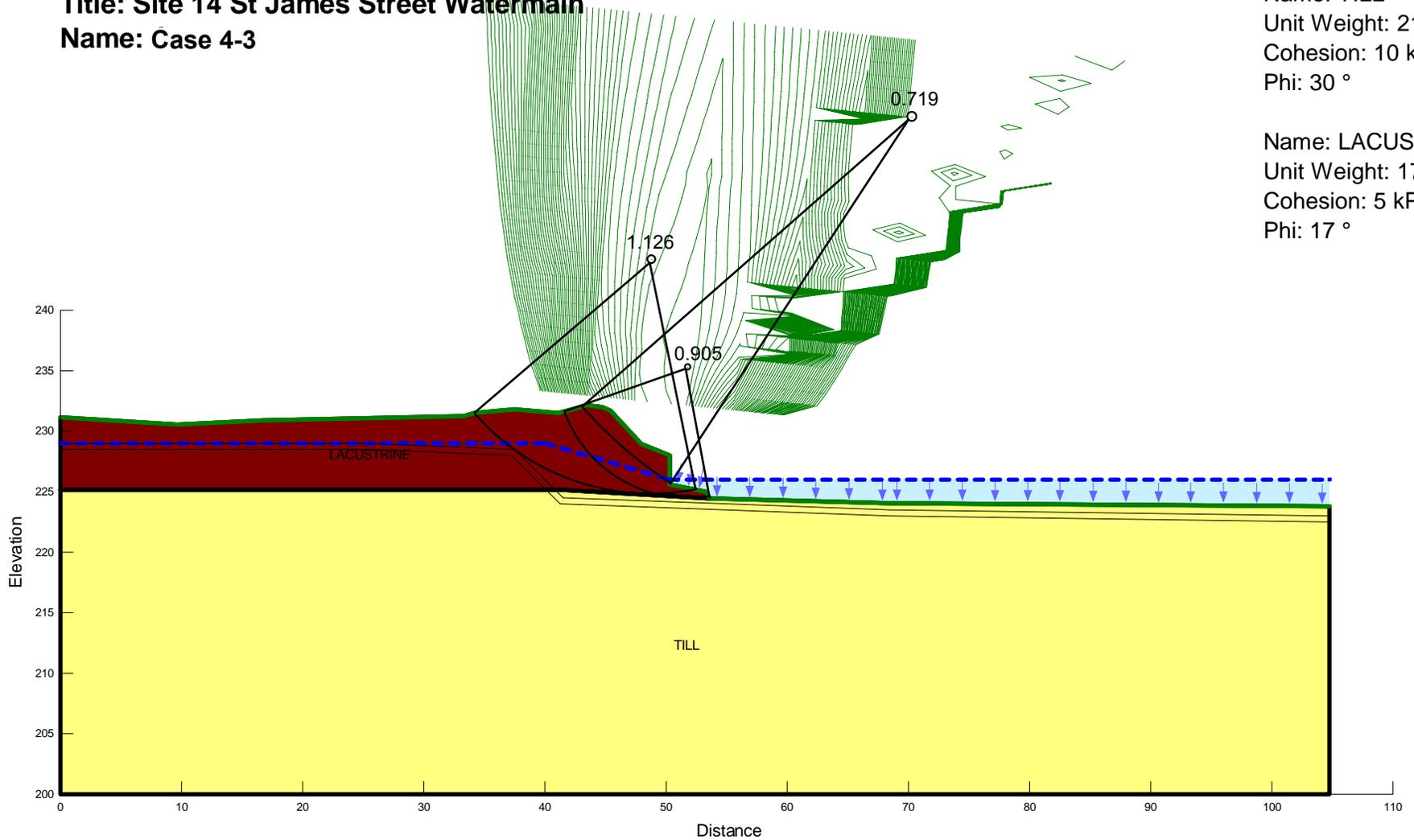
Name: LACUSTRINE
Unit Weight: 17 kN/m³
Cohesion: 5 kPa
Phi: 17 °



Condition Assessment of High Risk Water and Wastewater River Crossings
Title: Site 14 St James Street Watermain
Name: Case 4-3

Name: TILL
Unit Weight: 21 kN/m³
Cohesion: 10 kPa
Phi: 30 °

Name: LACUSTRINE
Unit Weight: 17 kN/m³
Cohesion: 5 kPa
Phi: 17 °



Appendix D

PICA Inspection Reports

City of Winnipeg 2015: 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

Condition Assessment Report, Standard Analysis



PICA – Pipeline Inspection & Condition Analysis Corporation
(A Subsidiary of Russell NDT Holdings Ltd.)

RFT ILI Tool

250mm (10in) Cast Iron & Steel

Assiniboine River Crossing

Between Misericordia Health Center & Wellington Crescent

Winnipeg, MB

PICA Project: Winnipeg 8007

Inspection Date:	October 20, 2015
Report Submission:	June 6, 2016 (Rev 2.0); February 2, 2016 (Rev 1.0)
Operators:	P. Ryhanen, G. Bouchard, B. Senka, G. Hubbard, D. Barron, D. Burton
Analysts:	E. Zake, J. Kim
Reviewers:	J. Regala
Report Revision:	2.0

CONFIDENTIAL

City of Winnipeg: 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

Condition Assessment Report, Standard Analysis

Executive Summary

PICA, under contract with AECOM, inspected a number of Water River Crossings for the City of Winnipeg using Remote Field Testing (RFT) Technology between October 19 and 24, 2015. This report documents the results of the RFT inspection of the 250mm (10in) Maryland Bridge Watermain, which crosses the Assiniboine River. The inspected portion spanned between two newly installed access wyes at the Misericordia Health Centre and the south end of the Maryland Bridge, near Wellington Cr (refer to the line map on page 4). The inspection was conducted on October 20th, 2015. The results reported here document PICA's findings.

As noted in the supplied pipeline drawings, this watermain is comprised of approximately 46m of cast iron (CI) and 121m of steel piping.

In general, the RFT analysis found the inspected section to be in “*fair to poor*” condition, with 82% of all pipes measuring less than 74% remaining wall (RW). A total of 441 localized wall loss indications were detected. Among these defects, 173 were classified as *shallow* ($\geq 65\%$ RW), 221 were *medium* (40-64% RW) and 43 were *deep* (20-39% RW). There were four defects that measured $< 20\%$ RW: 3%RW in P0150, 9%RW in P0160 and both 4%RW and 14%RW in P0300. The 4%RW defect in P0300 was verified as a through-hole during AECOM's lining of the watermain.

Note that all defect depth values in this report, and those in the original version (Rev 1.0) sent on February 2, 2016 were re-calibrated based on the aforementioned leak.

Table 1 provides an overview of the RFT findings on 10in Maryland Bridge Watermain River Crossing.

Table 1: Feature Indication Summary	
Inspected Length:	167.03m* <i>Cast Iron (46.33m) Steel (120.70m)</i>
Number of Pipe Sections:	38 <i>*Includes 2 elbows and couplings.</i>
Number of Analyzed Pipe Sections:	34
Average Wall Thickness:	98.2%
Thinnest circumferential pipe wall (Tcircmin):	78.4% (in Pipe 0020)
Number of pipes without localized wall loss indications:	5
Number of pipes with localized wall loss indications:	29
• Number of pipes in 'good' condition ($\geq 75\%$ RW):	6
• Number of pipes in 'fair' condition (50-74% RW):	9
• Number of pipes in 'poor' condition ($< 50\%$ RW):	19
Total number of wall loss indications reported:	441
Number of joint connections (bell & spigot, girth welds):	31
Number of Elbows:	2
Number of Couplings:	2

Figures 1 and 2 illustrate the distribution of localized wall loss along the 250mm (10in) Maryland Bridge Watermain (Assiniboine River Crossing) with respect to remaining wall and circumferential location respectively. Note that there may be some (partially) overlapping data points due to defect proximity.

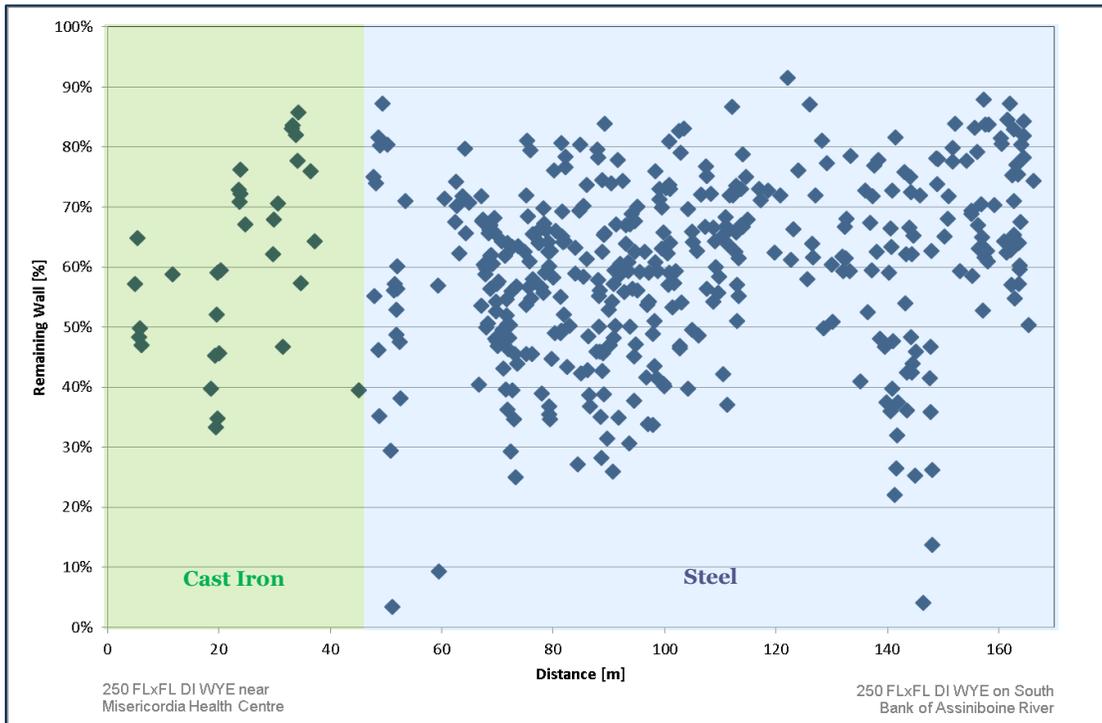


Figure 1: Distribution of wall loss with respect to remaining wall (%NWT) in pitting regions along the inspected section.

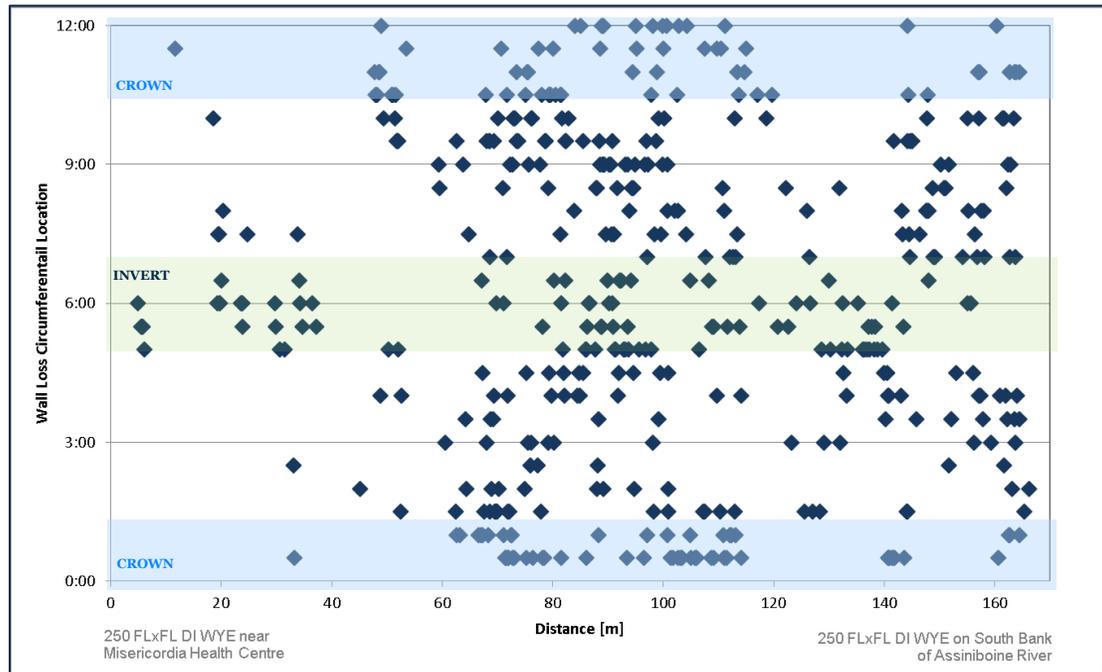


Figure 2: Circumferential distribution of pitting regions along the inspected section.

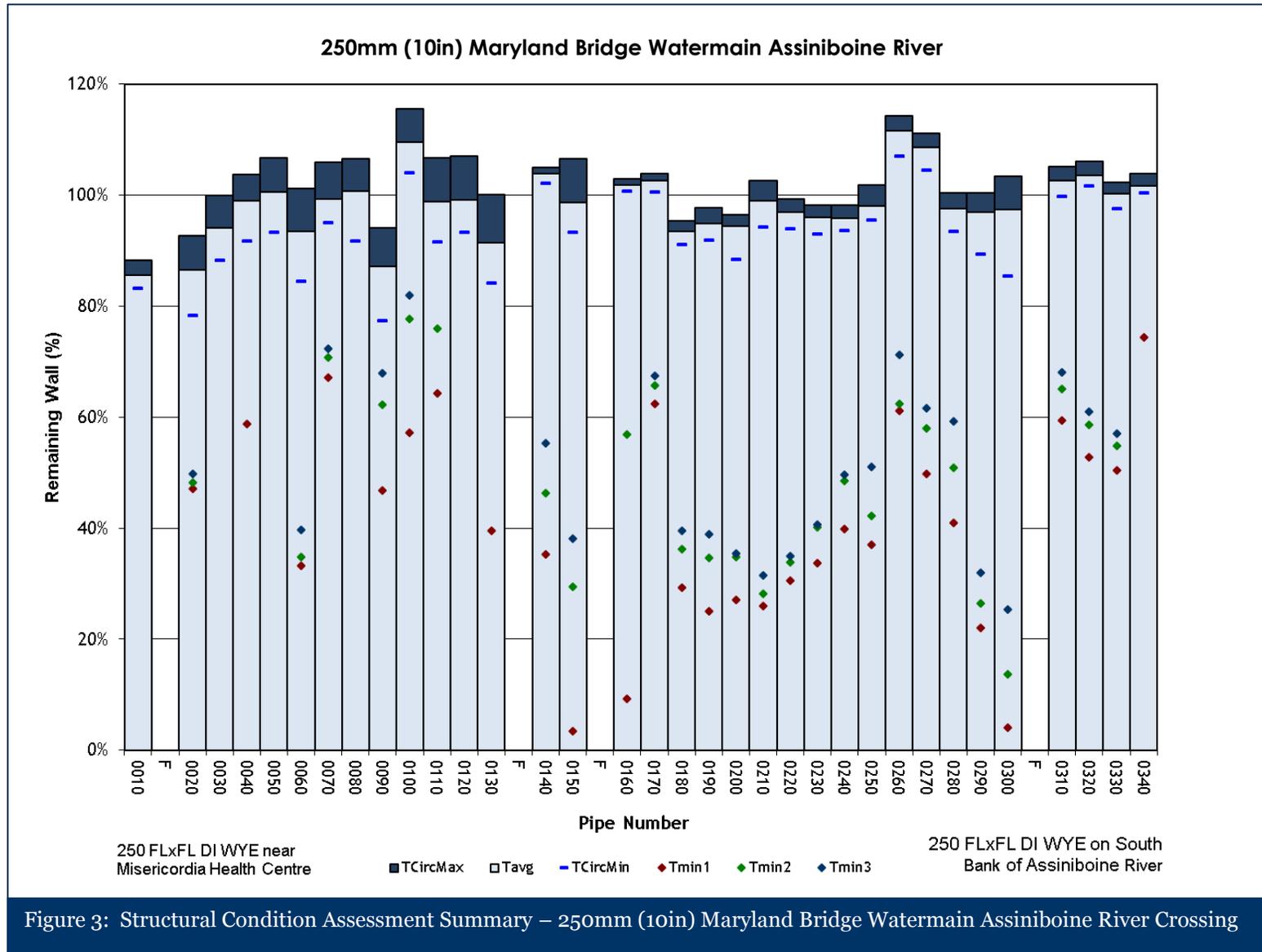


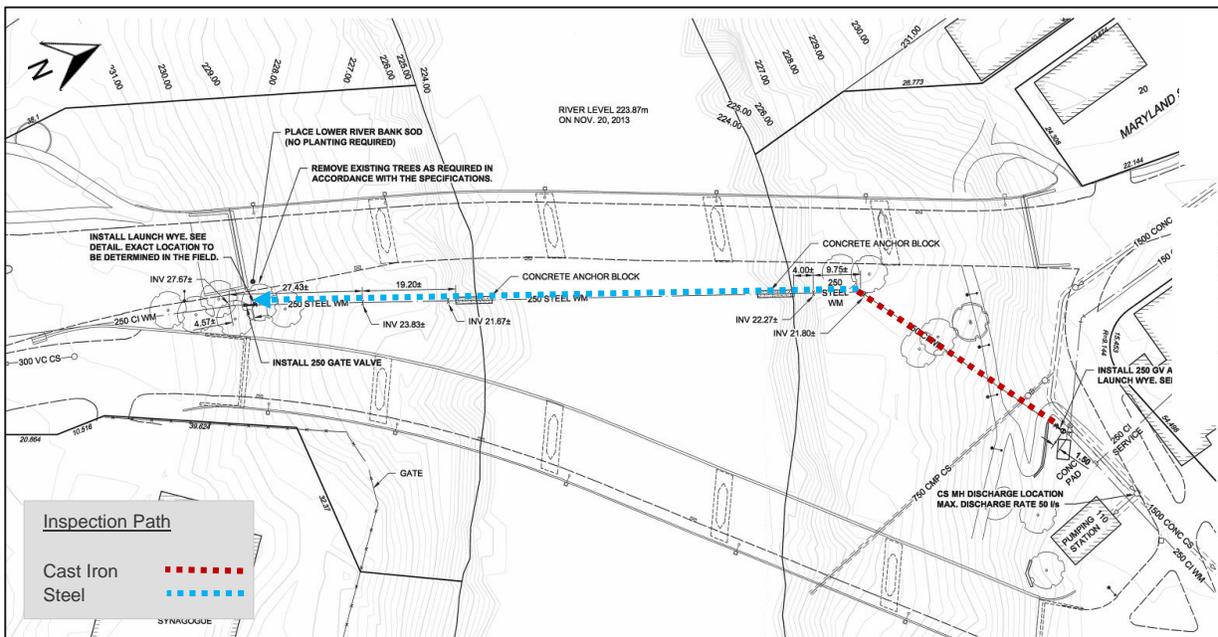
Table 2: Inspection Overview

Client:	City of Winnipeg	Location:	Winnipeg, MB
Line Name:	Maryland Bridge Watermain River Crossing	Pipe Diameter:	250mm (10in)
Material:	Cast Iron (46.33m) Steel (120.70m)	Year Installed:	1970
Break History:	One recent failure post inspection.		
Inspection Date:	October 20, 2015		
Inspected Length:	167.03m		
Lead Technician:	P. Ryhanen	Technician(s):	G. Bouchard, B. Senka, G. Hubbard, D. Barron, D. Burton
Launch Access:	250mm FLxFL DI WYE near Misericordia Health Center		
Retrieve Access:	250mm FLxFL DI WYE near Wellington Cr (South river bank)		

Operational Comments:

October 20, 2015

- 7:45am: Arrived on site and commence dual winch set up.
- 8:30am: Pull winch line to north side.
- 9:00am: Tool on and placed in pipe.
- 9:20am: Tool was launched from near Misericordia Health Centre.
- 10:25am: Tool arrived at the retrieve end.
- 10:45am: Retrieve run starts.
- 11:45am: Tool arrived back at the launch wye near Misericordia Health Centre and tool removed from line.
- 12:05pm: Data downloaded.
- 1:15pm: Site broken down.



Analysis Results

Location Reporting, Pipe Lengths & Features

The total logged distance for the 10in Maryland Bridge Watermain Assiniboine River Crossing was 167.03m. The inspection started on the north side of the Assiniboine River and ended on the south side near Wellington Crescent. During the inspection, the tool was stopped just short of the access wye at the retrieve (south) side. As a result, approximately 3.1m of the line was not inspected (at the south end). Adding this missed 3.1m to the total logged distance of 167.03m results in a total distance of 170.13m, which represents full coverage of the line.

The zero datum point was set at the downstream joint of a 30° bend that is part of the lateral connecting to the launch WYE. PICA confirmed with Tricor that the launch access was modified differently from the specified configuration (refer to Figure 4 below) due to the depth and steep slope associated with the north bank.

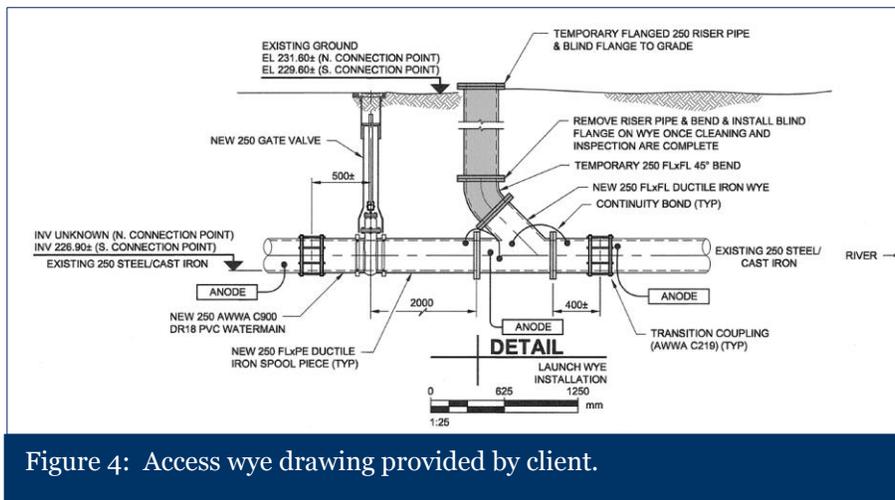


Figure 4: Access wye drawing provided by client.

The inspected line is comprised of two different pipe types: cast iron for the first 46m and steel for the remaining 121m. The average lengths for a standard pipe are 3.57m (11.7ft) and 6.31m (20.7ft) for cast and steel respectively. Shorter pieces were found adjacent to features (i.e. couplers, deflections). A total of four line features (two 45° bends and two couplers) were identified in the RFT data. In addition, nine shallow deflections were identified in the data using on board tool sensors.

General Wall Thickness

Pipe sections longer than 1.0m were analyzed to obtain the average remaining wall thickness (PARW) calculated over the length of the pipe section. The measured average wall thickness for the inspected portion of the 10in Maryland Bridge Watermain River Crossing is 98.2%.

Due to manufacturing tolerances, fluctuations of $\pm 15\%$ in the individual PARW values are common. Variations outside the normal $\pm 15\%$ spread can be an indicator of a different nominal wall thickness (NWT) or point towards a problem like aggregate pitting or general wall loss. All the inspected pipes exhibited PARW values that are within the manufacturing tolerances.

As mentioned above, there were two pipe types (cast iron and steel) that were observed in the data. The respective PARW values were calculated using nominal wall thickness values from each pipe type. Table 3, on pages 12-14, lists the pipe type transition points along the inspected section.

Local Wall Thickness

In general, the RFT analysis found the inspected section to be in “*fair to poor*” condition, with 82% of all pipes measuring less than 74% remaining wall (RW). A total of 441 localized wall loss indications were detected. Among these defects, 173 were classified as *shallow* ($\geq 65\%$ RW), 221 were *medium* (40-64% RW) and 43 were *deep* (20-39% RW). There were four defects that measured $< 20\%$ RW: 3%RW in P0150, 9%RW in P0160 and both 4%RW and 14%RW in P0300. The 4%RW defect in P0300 was verified as a through-hole during AECOM’s lining of this line.

Table 3, on pages 12-14, detail the three worst pitting indications per pipe (T_{min1} , T_{min2} and T_{min3}), as well as the average, minimum and maximum remaining wall values for the inspected section. The same results are shown graphically on page 4.

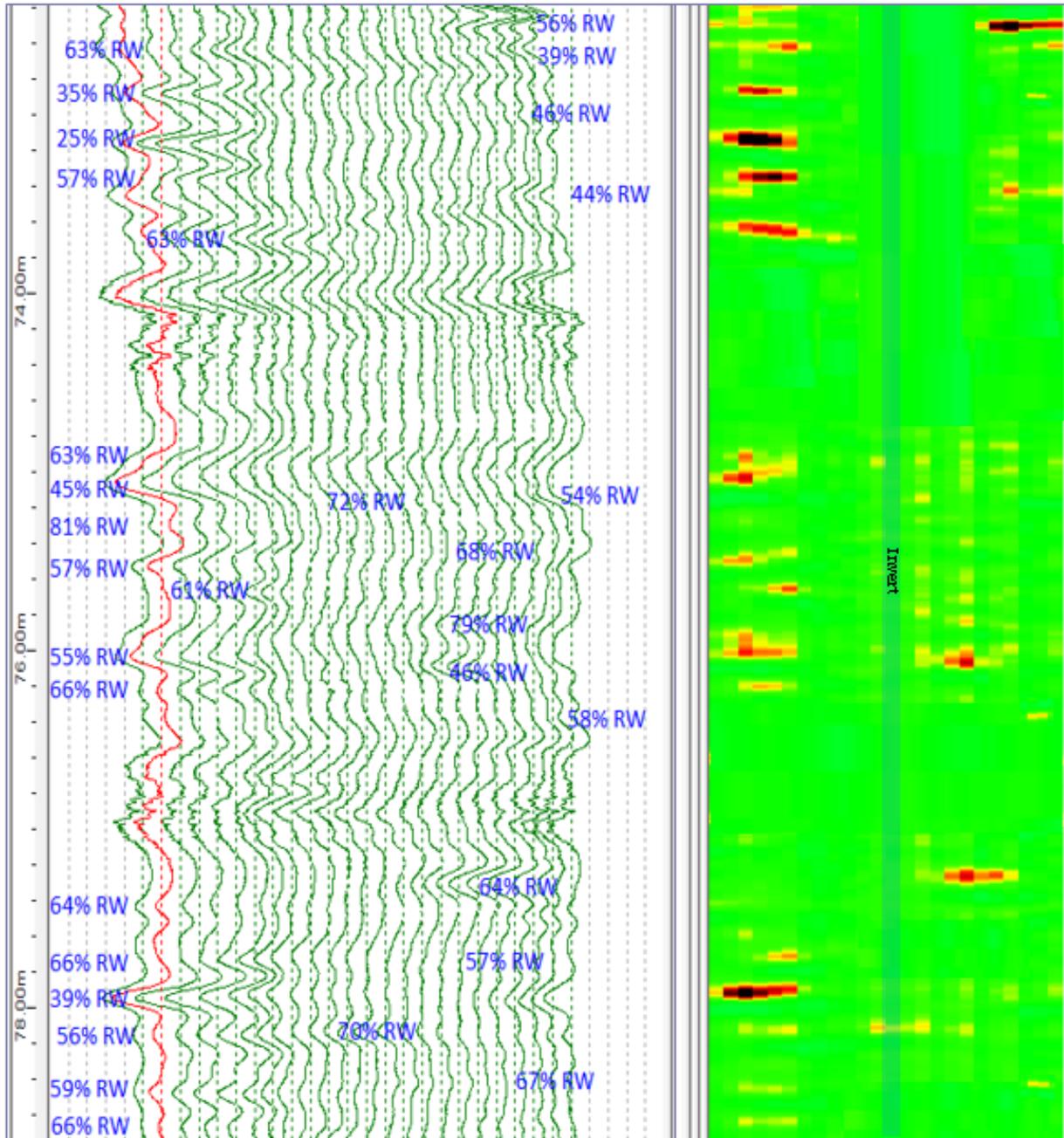
Table 4, on pages 15-55, provides additional detail for each defect with regards to its location relative to upstream and downstream features, axial length, signal quality index and analysis confidence level scoring.

Note that all defect depth values in this report, and those in the original version (Rev 1.0) sent on February 2, 2016 were re-calibrated based on the leak detected on the west bank during the lining operation.

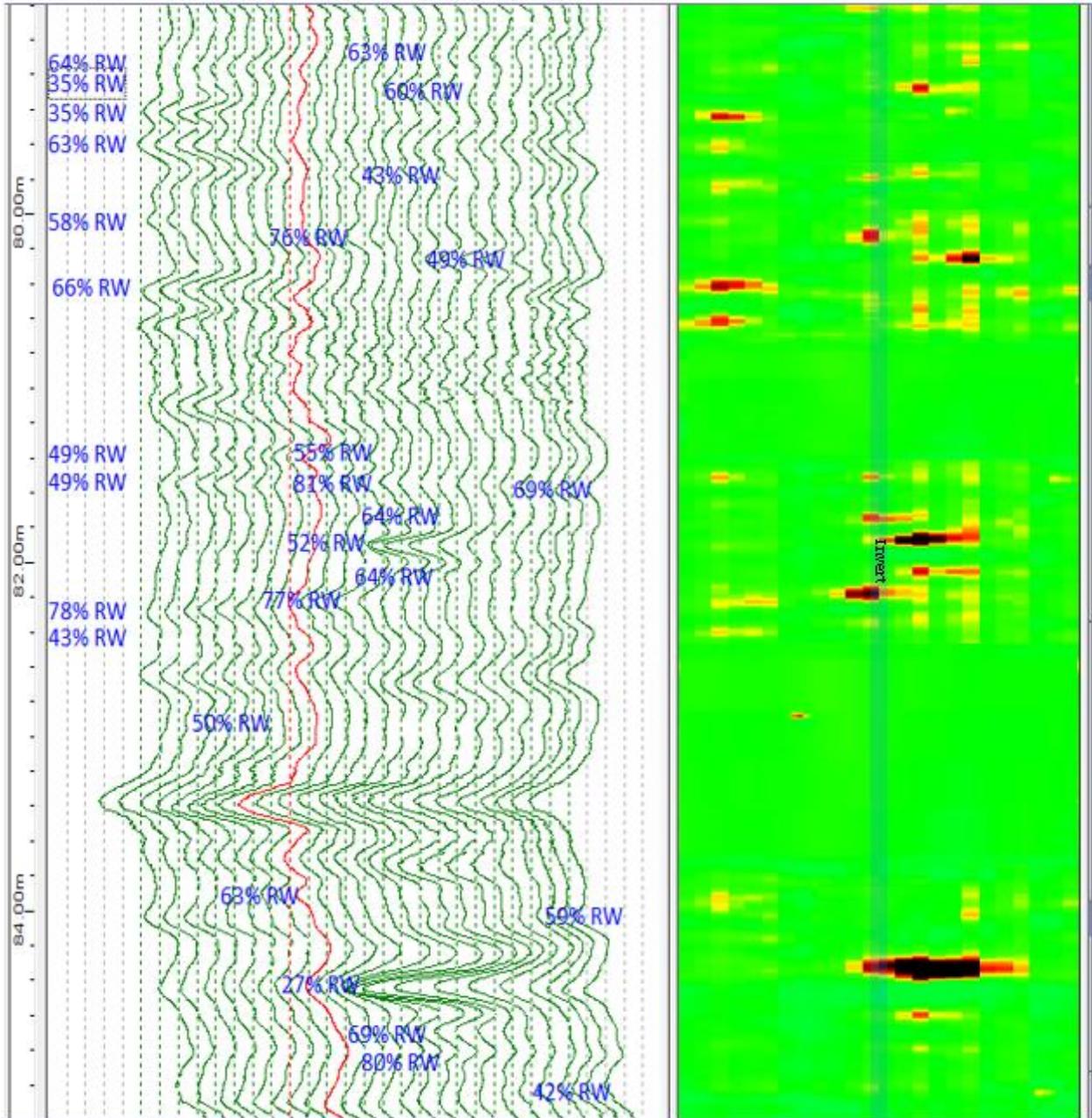
RFT Color Maps - Pipes 0190, 0200, 0210 and 0220

Color maps for four pipe sections with significant corrosion are presented in the next four pages. It is important to note that these color maps are for visual purposes only and should not be used to estimate the remaining wall.

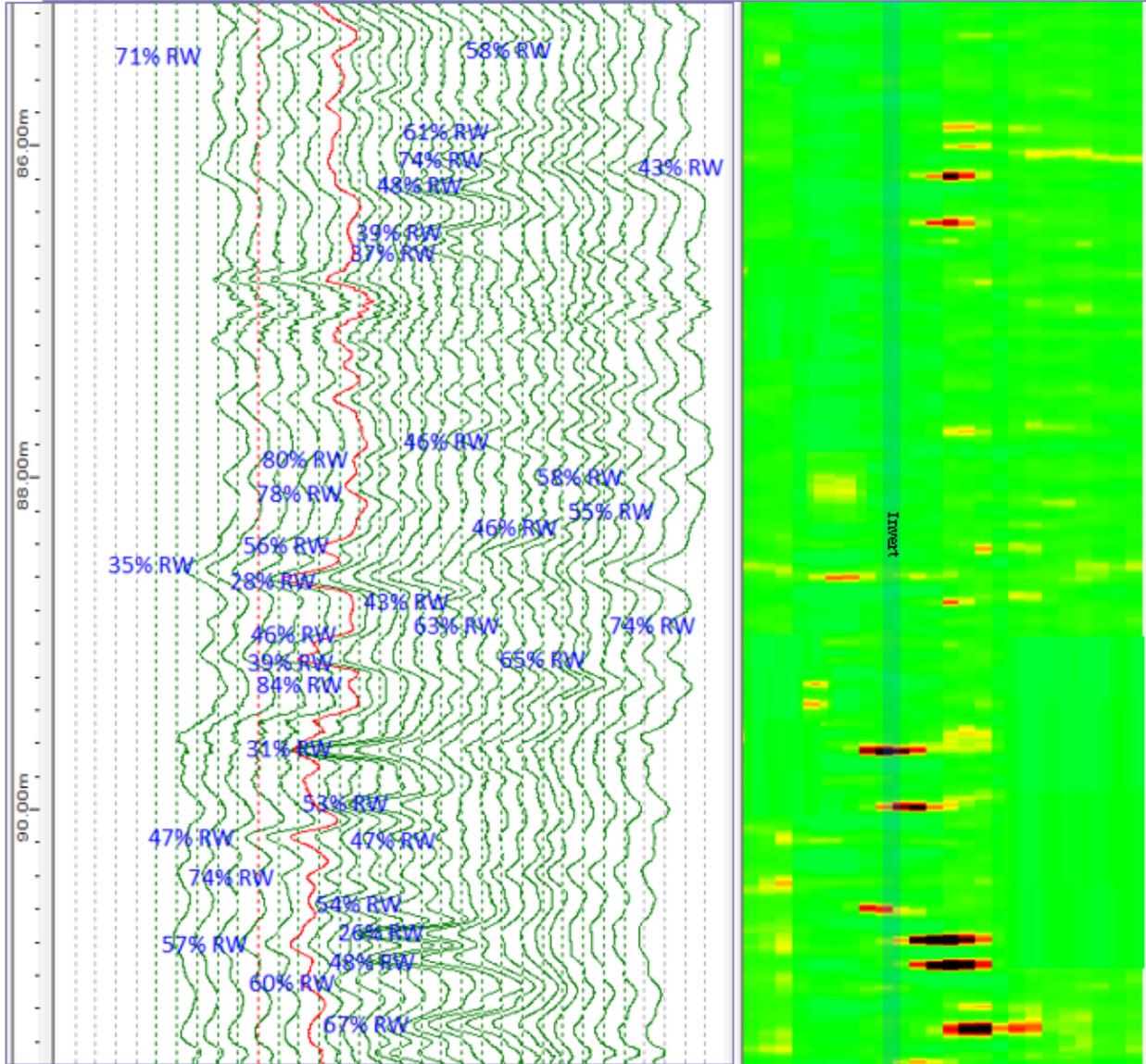
Pipe 0190: 72.44-78.81m (32 defects ranging between 25% and 81% RW)



Pipe 0200: 78.81-85.14m (31 defects ranging between 27% and 81% RW)



Pipe 0210: 85.14-91.52m (38 defects ranging between 26% and 84% RW)



Pipe 0220: 91.52-97.56m (30 defects ranging between 31% and 78% RW)

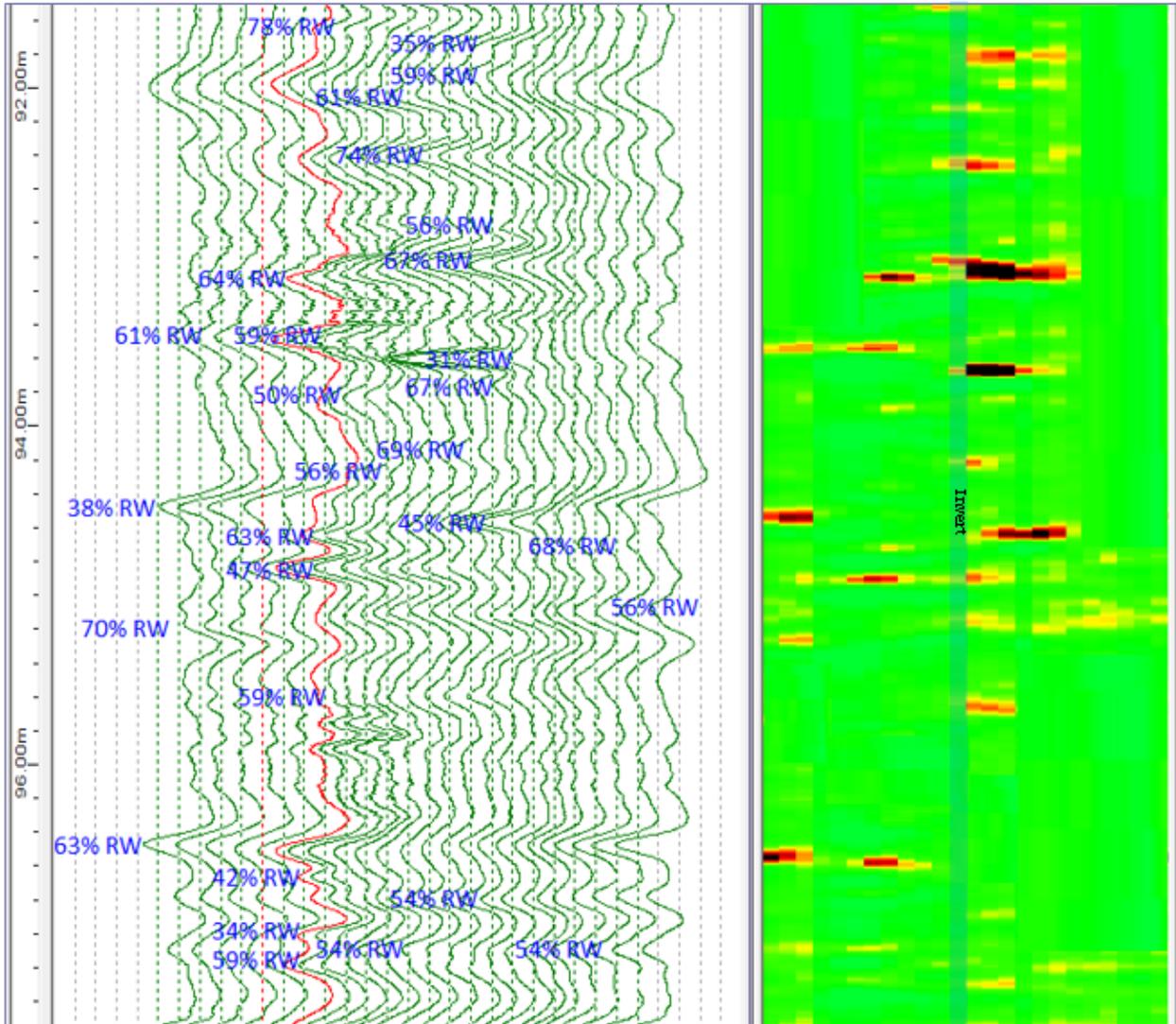


Table 3: Pipe List and Wall Thickness Readings – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

Pipe No.	Pipe Location			Tavg RW (%)	Circumferential Wall Thickness		Local Wall Thickness <i>Clock positions are with a Northeast to Southwest perspective (E.g. 3:00=NW, 9:00=SE). Defects entries in red were measured with <20% RW.</i>									Comment Entries in black are cast iron. Entries in blue are steel pipes.	
	Start (m)	End (m)	Length (m)		Tciremax RW (%)	Tciremin RW (%)	Tmin1			Tmin2			Tmin3				
							RW (%)	Location (m)	Clock Position	RW (%)	Location (m)	Clock Position	RW (%)	Location (m)	Clock Position		
0010	0.00	3.62	3.62	86%	88%	83%											The zero datum point was set at the downstream joint of a 30° bend that is part of the lateral connecting to the launch WYE. Cast iron section.
F	3.62	4.25	0.63														45° vertical bend
0020	4.25	7.76	3.51	86%	93%	78%	47%	6.08	5:00	48%	5.65	5:30	50%	5.75	5:30		This pipe contains two additional defects with 57% and 65% RW.
0030	7.76	11.32	3.56	94%	100%	88%											
0040	11.32	14.84	3.52	99%	104%	92%	59%	11.66	11:30								
0050	14.84	18.40	3.56	101%	107%	93%											
0060	18.40	21.94	3.54	93%	101%	85%	33%	19.49	7:30	35%	19.76	6:00	40%	18.55	10:00		This pipe contains 5 additional defects between 45% and 59% RW.
0070	21.94	25.53	3.58	99%	106%	95%	67%	24.78	7:30	71%	23.69	6:00	72%	23.91	6:00		This pipe contains two additional defects with 73% and 76% RW.
0080	25.53	29.10	3.58	101%	107%	92%											
0090	29.10	32.70	3.60	87%	94%	77%	47%	31.47	5:00	62%	29.66	6:00	68%	29.86	5:30		3° vertical deflection at 29.10m; This pipe contains an additional defect with 71% RW.
0100	32.70	36.28	3.58	110%	116%	104%	57%	34.64	5:30	78%	34.16	6:30	82%	33.78	7:30		This pipe contains 3 additional defects between 83% and 86% RW.
0110	36.28	39.88	3.60	99%	107%	92%	64%	37.14	5:30	76%	36.48	6:00					3° vertical deflection at 36.28m.
0120	39.88	43.47	3.59	99%	107%	93%											3° vertical/5° horizontal deflection at 39.88m.
0130	43.47	46.33	2.85	91%	100%	84%	39%	45.14	2:00								End of cast iron section.
F	46.33	47.62	1.29	102%	104%	100%	72%	47.30	1:30	83%	47.09	8:30					Start of steel pipe section. 45° horizontal bend; 13° vertical deflection at 46.33m

Table 3: Pipe List and Wall Thickness Readings – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

Pipe No.	Pipe Location			Tavg RW (%)	Circumferential Wall Thickness		Local Wall Thickness <i>Clock positions are with a Northeast to Southwest perspective (E.g. 3:00=NW, 9:00=SE). Defects entries in red were measured with <20% RW.</i>									Comment Entries in black are cast iron. Entries in blue are steel pipes.
	Start (m)	End (m)	Length (m)		Tciremax RW (%)	Tciremin RW (%)	Tmin1			Tmin2			Tmin3			
							RW (%)	Location (m)	Clock Position	RW (%)	Location (m)	Clock Position	RW (%)	Location (m)	Clock Position	
0140	47.62	50.39	2.77	104%	105%	102%	35%	48.83	4:00	46%	48.56	11:00	55%	47.94	10:30	This pipe contains 6 additional defects between 74% and 87% RW.
0150	50.39	55.85	5.46	99%	107%	93%	3%	51.19	10:30	29%	50.84	10:30	38%	52.57	4:00	7° vertical deflection at 50.39m; This pipe contains 8 additional defects between 47% and 71% RW.
F	55.85	56.06	0.22													Coupler
0160	56.06	59.78	3.71	102%	103%	101%	9%	59.49	8:30	57%	59.35	9:00				5° vertical deflection at 56.06m.
0170	59.78	66.24	6.46	103%	104%	101%	62%	63.10	1:00	66%	64.38	2:00	67%	62.38	1:30	This pipe contains 6 additional defects between 70% and 80% RW.
0180	66.24	72.44	6.20	93%	95%	91%	29%	72.41	1:00	36%	71.77	1:30	40%	71.57	0:30	This pipe contains 38 additional defects between 40% and 72% RW.
0190	72.44	78.81	6.37	95%	98%	92%	25%	73.21	10:00	35%	72.93	10:00	39%	77.99	10:30	This pipe contains 29 additional defects between 39% and 81% RW.
0200	78.81	85.14	6.33	94%	96%	88%	27%	84.46	4:00	35%	79.42	10:30	35%	79.25	10:30	This pipe contains 28 additional defects between 37% and 81% RW.
0210	85.14	91.52	6.38	99%	103%	94%	26%	90.75	6:00	28%	88.61	9:00	31%	89.64	7:30	This pipe contains 35 additional defects between 35% and 84% RW.
0220	91.52	97.56	6.04	97%	99%	94%	31%	93.59	5:30	34%	97.00	9:30	35%	91.74	4:00	This pipe contains 27 additional defects between 38% and 78% RW.
0230	97.56	103.61	6.05	96%	98%	93%	34%	97.88	5:00	40%	99.98	11:30	41%	99.82	9:00	This pipe contains 29 additional defects between 41% and 83% RW.
0240	103.61	109.87	6.26	96%	98%	94%	40%	104.13	7:30	49%	106.09	0:30	50%	104.93	1:00	5° horizontal deflection at 103.61m; This pipe contains 16 additional defects between 54% and 77% RW.
0250	109.87	116.22	6.35	98%	102%	95%	37%	111.21	12:00	42%	110.47	11:30	51%	113.03	7:00	This pipe contains 22 additional defects between 55% and 87% RW.
0260	116.22	122.88	6.66	112%	114%	107%	61%	122.69	5:30	62%	119.72	10:30	71%	117.27	6:00	This pipe contains 4 additional defects between 72% and 92% RW.
0270	122.88	129.45	6.57	109%	111%	104%	50%	128.59	5:00	58%	125.60	1:30	62%	126.64	6:00	This pipe contains 7 additional defects between 64% and 87% RW.
0280	129.45	135.65	6.20	98%	100%	94%	41%	135.19	6:00	51%	130.22	5:00	59%	133.17	4:00	5° vertical deflection at 132.15m; This pipe contains 7 additional defects between 59% and 79% RW.

Table 3: Pipe List and Wall Thickness Readings – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

Pipe No.	Pipe Location			Tavg RW (%)	Circumferential Wall Thickness		Local Wall Thickness									Comment Entries in black are cast iron. Entries in blue are steel pipes.
	Start (m)	End (m)	Length (m)		Tciremax RW (%)	Tciremin RW (%)	Tmin1			Tmin2			Tmin3			
							RW (%)	Location (m)	Clock Position	RW (%)	Location (m)	Clock Position	RW (%)	Location (m)	Clock Position	
							Clock positions are with a Northeast to Southwest perspective (E.g. 3:00=NW, 9:00=SE). Defects entries in red were measured with <20% RW.									
0290	135.65	142.00	6.35	97%	100%	89%	22%	141.36	0:30	26%	141.59	0:30	32%	141.75	9:30	This pipe contains 20 additional defects between 36% and 82% RW.
0300	142.00	148.27	6.27	97%	103%	85%	4%	146.39	7:30	14%	147.98	8:00	25%	144.91	9:30	This pipe contains 20 additional defects between 26% and 76% RW.
F	148.27	148.40	0.12													Coupler; 5° vertical deflection at 148.27m.
0310	148.40	154.61	6.21	103%	105%	100%	59%	153.00	4:30	65%	150.27	9:00	68%	150.83	8:30	This pipe contains 8 additional defects between 72% and 84% RW.
0320	154.61	159.89	5.28	104%	106%	102%	53%	157.12	10:00	59%	155.20	8:00	61%	158.02	8:00	This pipe contains 15 additional defects between 62% and 88% RW.
0330	159.89	165.88	5.99	100%	102%	98%	50%	165.31	1:30	55%	162.91	9:00	57%	162.26	3:30	This pipe contains 24 additional defects between 57% and 87% RW.
0340	165.88	167.03	1.16	102%	104%	100%	74%	166.26	2:00							End of run is near the newly installed 250mm FLxFL DI WYE on the South side of Maryland Bridge.

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
020	3100	4.95	57%	6:00	48	2	Medium	45° Vertical Bend	0.70	45° Horizontal Bend	42.67
020	3282	5.42	65%	5:30	38	2.5	Med-High	45° Vertical Bend	1.17	45° Horizontal Bend	42.20
020	3374	5.65	48%	5:30	30	3	High	45° Vertical Bend	1.40	45° Horizontal Bend	41.97
020	3415	5.75	50%	5:30	32	3	High	45° Vertical Bend	1.50	45° Horizontal Bend	41.87
020	3545	6.08	47%	5:00	89	3	High	45° Vertical Bend	1.83	45° Horizontal Bend	41.54
040	5742	11.66	59%	11:30	78	3	High	45° Vertical Bend	7.41	45° Horizontal Bend	35.96
060	8451	18.55	40%	10:00	18	1.5	Med-Low	45° Vertical Bend	14.30	45° Horizontal Bend	29.07
060	8764	19.34	45%	6:00	18	1.5	Med-Low	45° Vertical Bend	15.09	45° Horizontal Bend	28.28
060	8823	19.49	33%	7:30	56	3	High	45° Vertical Bend	15.24	45° Horizontal Bend	28.13
060	8866	19.60	52%	7:30	42	3	High	45° Vertical Bend	15.35	45° Horizontal Bend	28.02
060	8908	19.71	59%	6:00	10	1	Low	45° Vertical Bend	15.46	45° Horizontal Bend	27.91
060	8931	19.76	35%	6:00	18	1.5	Med-Low	45° Vertical Bend	15.51	45° Horizontal Bend	27.86
060	9036	20.03	46%	6:30	66	3	High	45° Vertical Bend	15.78	45° Horizontal Bend	27.59
060	9139	20.29	59%	8:00	56	3	High	45° Vertical Bend	16.04	45° Horizontal Bend	27.33
070	10416	23.54	73%	6:00	60	3	High	45° Vertical Bend	19.29	45° Horizontal Bend	24.08
070	10475	23.69	71%	6:00	78	3	High	45° Vertical Bend	19.44	45° Horizontal Bend	23.93
070	10531	23.83	76%	5:30	21	2.5	Med-High	45° Vertical Bend	19.58	45° Horizontal Bend	23.79
070	10562	23.91	72%	6:00	16	2	Medium	45° Vertical Bend	19.66	45° Horizontal Bend	23.71

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
070	10906	24.78	67%	7:30	33	2	Medium	45° Vertical Bend	20.53	45° Horizontal Bend	22.84
090	12825	29.66	62%	6:00	46	1.5	Med-Low	45° Vertical Bend	25.41	45° Horizontal Bend	17.96
090	12905	29.86	68%	5:30	98	3	High	45° Vertical Bend	25.61	45° Horizontal Bend	17.76
090	13214	30.64	71%	5:00	76	3	High	45° Vertical Bend	26.39	45° Horizontal Bend	16.98
090	13540	31.47	47%	5:00	26	3	High	45° Vertical Bend	27.22	45° Horizontal Bend	16.15
100	14147	33.01	83%	2:30	161	3	High	45° Vertical Bend	28.76	45° Horizontal Bend	14.61
100	14225	33.21	84%	0:30	132	3	High	45° Vertical Bend	28.96	45° Horizontal Bend	14.41
100	14449	33.78	82%	7:30	132	3	High	45° Vertical Bend	29.53	45° Horizontal Bend	13.84
100	14598	34.16	78%	6:30	104	2	Medium	45° Vertical Bend	29.91	45° Horizontal Bend	13.46
100	14641	34.27	86%	6:00	112	2.5	Med-High	45° Vertical Bend	30.02	45° Horizontal Bend	13.35
100	14786	34.64	57%	5:30	50	3	High	45° Vertical Bend	30.39	45° Horizontal Bend	12.98
110	15513	36.48	76%	6:00	48	1	Low	45° Vertical Bend	32.23	45° Horizontal Bend	11.14
110	15770	37.14	64%	5:30	86	2.5	Med-High	45° Vertical Bend	32.89	45° Horizontal Bend	10.48
130	18921	45.14	39%	2:00	97	2.5	Med-High	45° Vertical Bend	40.89	45° Horizontal Bend	2.48
140	19930	47.70	75%	11:00	54	2	Medium	45° Horizontal Bend	0.08	End of Run is at end of installed DI WYE at South side of Maryland Bridge	119.33
140	20022	47.94	55%	10:30	88	2.5	Med-High	45° Horizontal Bend	0.32	End of Run is at end of installed DI WYE at South side of Maryland Bridge	119.09

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* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
140	20102	48.14	74%	10:30	100	2.5	Med-High	45° Horizontal Bend	0.52	End of Run is at end of installed DI WYE at South side of Maryland Bridge	118.89
140	20267	48.56	46%	11:00	72	3	High	45° Horizontal Bend	0.94	End of Run is at end of installed DI WYE at South side of Maryland Bridge	118.47
140	20305	48.65	82%	11:00	110	3	High	45° Horizontal Bend	1.03	End of Run is at end of installed DI WYE at South side of Maryland Bridge	118.38
140	20375	48.83	35%	4:00	21	2.5	Med-High	45° Horizontal Bend	1.21	End of Run is at end of installed DI WYE at South side of Maryland Bridge	118.20
140	20406	48.91	80%	12:00	88	1	Low	45° Horizontal Bend	1.29	End of Run is at end of installed DI WYE at South side of Maryland Bridge	118.12
140	20593	49.39	87%	10:00	104	1.5	Med-Low	45° Horizontal Bend	1.77	End of Run is at end of installed DI WYE at South side of Maryland Bridge	117.64
140	20926	50.23	80%	5:00	118	1.5	Med-Low	45° Horizontal Bend	2.61	End of Run is at end of installed DI WYE at South side of Maryland Bridge	116.80
150	21166	50.84	29%	10:30	58	3	High	45° Horizontal Bend	3.22	End of Run is at end of installed DI WYE at South side of Maryland Bridge	116.19
150	21302	51.19	3%	10:30	50	3	High	45° Horizontal Bend	3.57	End of Run is at end of installed DI WYE at South side of Maryland Bridge	115.84
150	21385	51.40	56%	10:00	22	1	Low	45° Horizontal Bend	3.78	End of Run is at end of installed DI WYE at South side of Maryland Bridge	115.63
150	21472	51.62	57%	10:30	70	3	High	45° Horizontal Bend	4.00	End of Run is at end of installed DI WYE	115.41

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
										at South side of Maryland Bridge	
150	21542	51.80	53%	9:30	22	2.5	Med-High	45° Horizontal Bend	4.18	End of Run is at end of installed DI WYE at South side of Maryland Bridge	115.23
150	21589	51.92	49%	9:30	17	2.5	Med-High	45° Horizontal Bend	4.30	End of Run is at end of installed DI WYE at South side of Maryland Bridge	115.11
150	21642	52.05	56%	5:00	29	1	Low	45° Horizontal Bend	4.43	End of Run is at end of installed DI WYE at South side of Maryland Bridge	114.98
150	21642	52.05	60%	9:30	74	2.5	Med-High	45° Horizontal Bend	4.43	End of Run is at end of installed DI WYE at South side of Maryland Bridge	114.98
150	21788	52.42	47%	1:30	60	2.5	Med-High	45° Horizontal Bend	4.80	End of Run is at end of installed DI WYE at South side of Maryland Bridge	114.61
150	21847	52.57	38%	4:00	58	2.5	Med-High	45° Horizontal Bend	4.95	End of Run is at end of installed DI WYE at South side of Maryland Bridge	114.46
150	22208	53.49	71%	11:30	131	2	Medium	45° Horizontal Bend	5.87	End of Run is at end of installed DI WYE at South side of Maryland Bridge	113.54
160	24516	59.35	57%	9:00	22	2	Medium	45° Horizontal Bend	11.73	End of Run is at end of installed DI WYE at South side of Maryland Bridge	107.68
160	24569	59.49	9%	8:30	97	3	High	45° Horizontal Bend	11.87	End of Run is at end of installed DI WYE at South side of Maryland Bridge	107.54
170	24988	60.55	71%	3:00	131	1	Low	45° Horizontal Bend	12.93	End of Run is at end of installed DI WYE at South side of Maryland Bridge	106.48

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
170	25708	62.38	67%	1:30	109	2	Medium	45° Horizontal Bend	14.76	End of Run is at end of installed DI WYE at South side of Maryland Bridge	104.65
170	25778	62.56	74%	1:00	96	1.5	Med-Low	45° Horizontal Bend	14.94	End of Run is at end of installed DI WYE at South side of Maryland Bridge	104.47
170	25812	62.64	70%	9:30	104	2	Medium	45° Horizontal Bend	15.02	End of Run is at end of installed DI WYE at South side of Maryland Bridge	104.39
170	25994	63.10	62%	1:00	78	1	Low	45° Horizontal Bend	15.48	End of Run is at end of installed DI WYE at South side of Maryland Bridge	103.93
170	26247	63.75	72%	9:00	62	1.5	Med-Low	45° Horizontal Bend	16.13	End of Run is at end of installed DI WYE at South side of Maryland Bridge	103.28
170	26401	64.14	80%	3:30	84	1	Low	45° Horizontal Bend	16.52	End of Run is at end of installed DI WYE at South side of Maryland Bridge	102.89
170	26497	64.38	66%	2:00	109	1.5	Med-Low	45° Horizontal Bend	16.76	End of Run is at end of installed DI WYE at South side of Maryland Bridge	102.65
170	26685	64.86	71%	7:30	76	1.5	Med-Low	45° Horizontal Bend	17.24	End of Run is at end of installed DI WYE at South side of Maryland Bridge	102.17
180	27375	66.61	40%	1:00	68	2	Medium	45° Horizontal Bend	18.99	End of Run is at end of installed DI WYE at South side of Maryland Bridge	100.42
180	27542	67.04	54%	1:00	34	3	High	45° Horizontal Bend	19.42	End of Run is at end of installed DI WYE at South side of Maryland Bridge	99.99
180	27557	67.07	72%	6:30	37	1.5	Med-Low	45° Horizontal Bend	19.45	End of Run is at end of installed DI WYE	99.96

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
										at South side of Maryland Bridge	
180	27634	67.27	68%	1:00	92	2	Medium	45° Horizontal Bend	19.65	End of Run is at end of installed DI WYE at South side of Maryland Bridge	99.76
180	27660	67.34	68%	4:30	62	1.5	Med-Low	45° Horizontal Bend	19.72	End of Run is at end of installed DI WYE at South side of Maryland Bridge	99.69
180	27729	67.51	60%	1:30	68	2	Medium	45° Horizontal Bend	19.89	End of Run is at end of installed DI WYE at South side of Maryland Bridge	99.52
180	27867	67.86	59%	10:30	70	2	Medium	45° Horizontal Bend	20.24	End of Run is at end of installed DI WYE at South side of Maryland Bridge	99.17
180	27920	68.00	60%	9:30	27	2.5	Med-High	45° Horizontal Bend	20.38	End of Run is at end of installed DI WYE at South side of Maryland Bridge	99.03
180	27928	68.02	50%	3:00	115	2.5	Med-High	45° Horizontal Bend	20.40	End of Run is at end of installed DI WYE at South side of Maryland Bridge	99.01
180	28018	68.25	51%	1:00	19	2.5	Med-High	45° Horizontal Bend	20.63	End of Run is at end of installed DI WYE at South side of Maryland Bridge	98.78
180	28080	68.40	66%	9:30	94	2	Medium	45° Horizontal Bend	20.78	End of Run is at end of installed DI WYE at South side of Maryland Bridge	98.63
180	28166	68.62	66%	7:00	92	2.5	Med-High	45° Horizontal Bend	21.00	End of Run is at end of installed DI WYE at South side of Maryland Bridge	98.41
180	28174	68.64	62%	1:30	82	2.5	Med-High	45° Horizontal Bend	21.02	End of Run is at end of installed DI WYE at South side of Maryland Bridge	98.39

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
180	28185	68.67	56%	9:30	30	1	Low	45° Horizontal Bend	21.05	End of Run is at end of installed DI WYE at South side of Maryland Bridge	98.36
180	28221	68.76	67%	3:30	23	1.5	Med-Low	45° Horizontal Bend	21.14	End of Run is at end of installed DI WYE at South side of Maryland Bridge	98.27
180	28273	68.89	62%	2:00	52	2.5	Med-High	45° Horizontal Bend	21.27	End of Run is at end of installed DI WYE at South side of Maryland Bridge	98.14
180	28372	69.14	60%	3:30	98	3	High	45° Horizontal Bend	21.52	End of Run is at end of installed DI WYE at South side of Maryland Bridge	97.89
180	28469	69.39	68%	4:00	105	3	High	45° Horizontal Bend	21.77	End of Run is at end of installed DI WYE at South side of Maryland Bridge	97.64
180	28474	69.40	66%	9:30	94	3	High	45° Horizontal Bend	21.78	End of Run is at end of installed DI WYE at South side of Maryland Bridge	97.63
180	28530	69.55	48%	1:30	21	2.5	Med-High	45° Horizontal Bend	21.93	End of Run is at end of installed DI WYE at South side of Maryland Bridge	97.48
180	28592	69.70	53%	1:30	22	2	Medium	45° Horizontal Bend	22.08	End of Run is at end of installed DI WYE at South side of Maryland Bridge	97.33
180	28594	69.71	54%	6:00	42	2	Medium	45° Horizontal Bend	22.09	End of Run is at end of installed DI WYE at South side of Maryland Bridge	97.32
180	28639	69.82	48%	1:30	26	3	High	45° Horizontal Bend	22.20	End of Run is at end of installed DI WYE at South side of Maryland Bridge	97.21
180	28691	69.96	57%	1:30	74	3	High	45° Horizontal Bend	22.34	End of Run is at end of installed DI WYE	97.07

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
										at South side of Maryland Bridge	
180	28710	70.00	47%	10:00	105	3	High	45° Horizontal Bend	22.38	End of Run is at end of installed DI WYE at South side of Maryland Bridge	97.03
180	28786	70.20	58%	2:00	109	2.5	Med-High	45° Horizontal Bend	22.58	End of Run is at end of installed DI WYE at South side of Maryland Bridge	96.83
180	28991	70.72	64%	11:30	60	2.5	Med-High	45° Horizontal Bend	23.10	End of Run is at end of installed DI WYE at South side of Maryland Bridge	96.31
180	29061	70.89	50%	8:30	38	2	Medium	45° Horizontal Bend	23.27	End of Run is at end of installed DI WYE at South side of Maryland Bridge	96.14
180	29121	71.05	43%	6:00	32	3	High	45° Horizontal Bend	23.43	End of Run is at end of installed DI WYE at South side of Maryland Bridge	95.98
180	29141	71.10	47%	1:00	64	2.5	Med-High	45° Horizontal Bend	23.48	End of Run is at end of installed DI WYE at South side of Maryland Bridge	95.93
180	29255	71.39	62%	0:30	80	2.5	Med-High	45° Horizontal Bend	23.77	End of Run is at end of installed DI WYE at South side of Maryland Bridge	95.64
180	29326	71.57	40%	0:30	68	3	High	45° Horizontal Bend	23.95	End of Run is at end of installed DI WYE at South side of Maryland Bridge	95.46
180	29355	71.64	52%	7:00	30	1	Low	45° Horizontal Bend	24.02	End of Run is at end of installed DI WYE at South side of Maryland Bridge	95.39
180	29355	71.64	55%	10:30	76	3	High	45° Horizontal Bend	24.02	End of Run is at end of installed DI WYE at South side of Maryland Bridge	95.39

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
180	29405	71.77	36%	1:30	32	1.5	Med-Low	45° Horizontal Bend	24.15	End of Run is at end of installed DI WYE at South side of Maryland Bridge	95.26
180	29432	71.84	64%	4:00	70	2	Medium	45° Horizontal Bend	24.22	End of Run is at end of installed DI WYE at South side of Maryland Bridge	95.19
180	29440	71.86	50%	0:30	26	2.5	Med-High	45° Horizontal Bend	24.24	End of Run is at end of installed DI WYE at South side of Maryland Bridge	95.17
180	29501	72.01	46%	0:30	42	2.5	Med-High	45° Horizontal Bend	24.39	End of Run is at end of installed DI WYE at South side of Maryland Bridge	95.02
180	29549	72.13	48%	1:30	54	2.5	Med-High	45° Horizontal Bend	24.51	End of Run is at end of installed DI WYE at South side of Maryland Bridge	94.90
180	29603	72.27	50%	9:00	21	1.5	Med-Low	45° Horizontal Bend	24.65	End of Run is at end of installed DI WYE at South side of Maryland Bridge	94.76
180	29658	72.41	29%	1:00	72	3	High	45° Horizontal Bend	24.79	End of Run is at end of installed DI WYE at South side of Maryland Bridge	94.62
190	29717	72.56	56%	1:00	21	3	High	45° Horizontal Bend	24.94	End of Run is at end of installed DI WYE at South side of Maryland Bridge	94.47
190	29760	72.67	39%	0:30	44	3	High	45° Horizontal Bend	25.05	End of Run is at end of installed DI WYE at South side of Maryland Bridge	94.36
190	29765	72.68	63%	9:00	64	3	High	45° Horizontal Bend	25.06	End of Run is at end of installed DI WYE at South side of Maryland Bridge	94.35
190	29864	72.93	35%	10:00	56	3	High	45° Horizontal Bend	25.31	End of Run is at end of installed DI WYE	94.10

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
										at South side of Maryland Bridge	
190	29909	73.05	46%	0:30	24	2	Medium	45° Horizontal Bend	25.43	End of Run is at end of installed DI WYE at South side of Maryland Bridge	93.98
190	29973	73.21	25%	10:00	105	4	Very High	45° Horizontal Bend	25.59	End of Run is at end of installed DI WYE at South side of Maryland Bridge	93.82
190	30054	73.42	57%	9:30	50	3	High	45° Horizontal Bend	25.80	End of Run is at end of installed DI WYE at South side of Maryland Bridge	93.61
190	30082	73.49	44%	11:00	42	2	Medium	45° Horizontal Bend	25.87	End of Run is at end of installed DI WYE at South side of Maryland Bridge	93.54
190	30178	73.73	63%	9:30	84	2.5	Med-High	45° Horizontal Bend	26.11	End of Run is at end of installed DI WYE at South side of Maryland Bridge	93.30
190	30651	74.93	63%	2:00	23	1	Low	45° Horizontal Bend	27.31	End of Run is at end of installed DI WYE at South side of Maryland Bridge	92.10
190	30720	75.11	45%	10:30	105	3	High	45° Horizontal Bend	27.49	End of Run is at end of installed DI WYE at South side of Maryland Bridge	91.92
190	30745	75.17	54%	0:30	30	1.5	Med-Low	45° Horizontal Bend	27.55	End of Run is at end of installed DI WYE at South side of Maryland Bridge	91.86
190	30762	75.22	72%	4:30	21	1	Low	45° Horizontal Bend	27.60	End of Run is at end of installed DI WYE at South side of Maryland Bridge	91.81
190	30818	75.36	81%	11:00	161	1	Low	45° Horizontal Bend	27.74	End of Run is at end of installed DI WYE at South side of Maryland Bridge	91.67

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
190	30861	75.47	68%	3:00	29	1	Low	45° Horizontal Bend	27.85	End of Run is at end of installed DI WYE at South side of Maryland Bridge	91.56
190	30905	75.58	57%	11:00	66	2.5	Med-High	45° Horizontal Bend	27.96	End of Run is at end of installed DI WYE at South side of Maryland Bridge	91.45
190	30962	75.72	61%	9:00	68	3	High	45° Horizontal Bend	28.10	End of Run is at end of installed DI WYE at South side of Maryland Bridge	91.31
190	31039	75.92	79%	2:30	66	2	Medium	45° Horizontal Bend	28.30	End of Run is at end of installed DI WYE at South side of Maryland Bridge	91.11
190	31103	76.08	55%	10:00	64	2.5	Med-High	45° Horizontal Bend	28.46	End of Run is at end of installed DI WYE at South side of Maryland Bridge	90.95
190	31120	76.12	46%	3:00	32	2.5	Med-High	45° Horizontal Bend	28.50	End of Run is at end of installed DI WYE at South side of Maryland Bridge	90.91
190	31183	76.28	66%	10:00	92	2.5	Med-High	45° Horizontal Bend	28.66	End of Run is at end of installed DI WYE at South side of Maryland Bridge	90.75
190	31233	76.41	58%	0:30	48	1	Low	45° Horizontal Bend	28.79	End of Run is at end of installed DI WYE at South side of Maryland Bridge	90.62
190	31597	77.34	64%	2:30	113	2.5	Med-High	45° Horizontal Bend	29.72	End of Run is at end of installed DI WYE at South side of Maryland Bridge	89.69
190	31653	77.48	64%	11:30	44	1.5	Med-Low	45° Horizontal Bend	29.86	End of Run is at end of installed DI WYE at South side of Maryland Bridge	89.55
190	31770	77.78	66%	9:00	54	2.5	Med-High	45° Horizontal Bend	30.16	End of Run is at end of installed DI WYE	89.25

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
										at South side of Maryland Bridge	
190	31783	77.81	57%	1:30	23	1	Low	45° Horizontal Bend	30.19	End of Run is at end of installed DI WYE at South side of Maryland Bridge	89.22
190	31856	77.99	39%	10:30	101	4	Very High	45° Horizontal Bend	30.37	End of Run is at end of installed DI WYE at South side of Maryland Bridge	89.04
190	31928	78.18	70%	5:30	76	2	Medium	45° Horizontal Bend	30.56	End of Run is at end of installed DI WYE at South side of Maryland Bridge	88.85
190	31938	78.20	56%	0:30	74	2.5	Med-High	45° Horizontal Bend	30.58	End of Run is at end of installed DI WYE at South side of Maryland Bridge	88.83
190	32028	78.43	67%	0:30	29	1	Low	45° Horizontal Bend	30.81	End of Run is at end of installed DI WYE at South side of Maryland Bridge	88.60
190	32062	78.52	59%	9:30	66	2.5	Med-High	45° Horizontal Bend	30.90	End of Run is at end of installed DI WYE at South side of Maryland Bridge	88.51
190	32144	78.73	66%	9:30	52	1.5	Med-Low	45° Horizontal Bend	31.11	End of Run is at end of installed DI WYE at South side of Maryland Bridge	88.30
200	32293	79.10	63%	3:00	46	1.5	Med-Low	45° Horizontal Bend	31.48	End of Run is at end of installed DI WYE at South side of Maryland Bridge	87.93
200	32311	79.15	64%	8:30	58	1.5	Med-Low	45° Horizontal Bend	31.53	End of Run is at end of installed DI WYE at South side of Maryland Bridge	87.88
200	32352	79.25	35%	10:30	21	2	Medium	45° Horizontal Bend	31.63	End of Run is at end of installed DI WYE at South side of Maryland Bridge	87.78

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
200	32353	79.26	37%	10:30	26	1.5	Med-Low	45° Horizontal Bend	31.64	End of Run is at end of installed DI WYE at South side of Maryland Bridge	87.77
200	32365	79.29	60%	4:30	76	1.5	Med-Low	45° Horizontal Bend	31.67	End of Run is at end of installed DI WYE at South side of Maryland Bridge	87.74
200	32418	79.42	35%	10:30	46	3	High	45° Horizontal Bend	31.80	End of Run is at end of installed DI WYE at South side of Maryland Bridge	87.61
200	32493	79.61	63%	10:30	117	2.5	Med-High	45° Horizontal Bend	31.99	End of Run is at end of installed DI WYE at South side of Maryland Bridge	87.42
200	32547	79.75	45%	4:00	26	2	Medium	45° Horizontal Bend	32.13	End of Run is at end of installed DI WYE at South side of Maryland Bridge	87.28
200	32659	80.03	58%	11:30	52	1.5	Med-Low	45° Horizontal Bend	32.41	End of Run is at end of installed DI WYE at South side of Maryland Bridge	87.00
200	32699	80.14	76%	6:30	102	1	Low	45° Horizontal Bend	32.52	End of Run is at end of installed DI WYE at South side of Maryland Bridge	86.89
200	32747	80.26	49%	3:00	60	2.5	Med-High	45° Horizontal Bend	32.64	End of Run is at end of installed DI WYE at South side of Maryland Bridge	86.77
200	32813	80.43	66%	10:30	115	2	Medium	45° Horizontal Bend	32.81	End of Run is at end of installed DI WYE at South side of Maryland Bridge	86.60
200	33187	81.37	49%	10:30	23	2	Medium	45° Horizontal Bend	33.75	End of Run is at end of installed DI WYE at South side of Maryland Bridge	85.66
200	33196	81.40	55%	7:30	76	1	Low	45° Horizontal Bend	33.78	End of Run is at end of installed DI WYE	85.63

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
										at South side of Maryland Bridge	
200	33250	81.54	81%	6:00	100	2	Medium	45° Horizontal Bend	33.92	End of Run is at end of installed DI WYE at South side of Maryland Bridge	85.50
200	33254	81.55	49%	10:30	66	1.5	Med-Low	45° Horizontal Bend	33.93	End of Run is at end of installed DI WYE at South side of Maryland Bridge	85.48
200	33273	81.59	69%	0:30	52	1	Low	45° Horizontal Bend	33.97	End of Run is at end of installed DI WYE at South side of Maryland Bridge	85.44
200	33306	81.68	65%	10:00	14	1	Low	45° Horizontal Bend	34.06	End of Run is at end of installed DI WYE at South side of Maryland Bridge	85.35
200	33334	81.75	64%	5:00	18	2	Medium	45° Horizontal Bend	34.13	End of Run is at end of installed DI WYE at South side of Maryland Bridge	85.28
200	33391	81.89	52%	4:30	66	3	High	45° Horizontal Bend	34.27	End of Run is at end of installed DI WYE at South side of Maryland Bridge	85.14
200	33464	82.08	64%	4:00	33	2	Medium	45° Horizontal Bend	34.46	End of Run is at end of installed DI WYE at South side of Maryland Bridge	84.95
200	33521	82.22	77%	6:30	94	2.5	Med-High	45° Horizontal Bend	34.60	End of Run is at end of installed DI WYE at South side of Maryland Bridge	84.81
200	33536	82.26	78%	9:30	70	2	Medium	45° Horizontal Bend	34.64	End of Run is at end of installed DI WYE at South side of Maryland Bridge	84.77
200	33603	82.43	43%	9:30	25	1	Low	45° Horizontal Bend	34.81	End of Run is at end of installed DI WYE at South side of Maryland Bridge	84.60

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
200	33790	82.91	50%	10:00	22	1	Low	45° Horizontal Bend	35.29	End of Run is at end of installed DI WYE at South side of Maryland Bridge	84.12
200	34183	83.90	63%	8:00	62	1.5	Med-Low	45° Horizontal Bend	36.28	End of Run is at end of installed DI WYE at South side of Maryland Bridge	83.13
200	34231	84.03	59%	12:00	60	1	Low	45° Horizontal Bend	36.41	End of Run is at end of installed DI WYE at South side of Maryland Bridge	83.00
200	34401	84.46	27%	4:00	170	4	Very High	45° Horizontal Bend	36.84	End of Run is at end of installed DI WYE at South side of Maryland Bridge	82.57
200	34499	84.71	69%	4:30	127	2	Medium	45° Horizontal Bend	37.09	End of Run is at end of installed DI WYE at South side of Maryland Bridge	82.32
200	34569	84.89	80%	4:00	50	1.5	Med-Low	45° Horizontal Bend	37.27	End of Run is at end of installed DI WYE at South side of Maryland Bridge	82.14
200	34614	85.00	42%	12:00	19	1	Low	45° Horizontal Bend	37.38	End of Run is at end of installed DI WYE at South side of Maryland Bridge	82.03
210	34784	85.43	58%	4:30	52	1.5	Med-Low	45° Horizontal Bend	37.81	End of Run is at end of installed DI WYE at South side of Maryland Bridge	81.60
210	34792	85.45	70%	9:30	86	1.5	Med-Low	45° Horizontal Bend	37.83	End of Run is at end of installed DI WYE at South side of Maryland Bridge	81.58
210	34994	85.96	61%	5:00	38	2.5	Med-High	45° Horizontal Bend	38.34	End of Run is at end of installed DI WYE at South side of Maryland Bridge	81.07
210	35027	86.05	74%	5:00	54	3	High	45° Horizontal Bend	38.43	End of Run is at end of installed DI WYE	80.98

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
										at South side of Maryland Bridge	
210	35060	86.13	43%	0:30	28	3	High	45° Horizontal Bend	38.51	End of Run is at end of installed DI WYE at South side of Maryland Bridge	80.90
210	35105	86.25	48%	5:30	48	3	High	45° Horizontal Bend	38.63	End of Run is at end of installed DI WYE at South side of Maryland Bridge	80.78
210	35214	86.52	39%	6:00	24	3	High	45° Horizontal Bend	38.90	End of Run is at end of installed DI WYE at South side of Maryland Bridge	80.51
210	35250	86.62	37%	6:00	18	3	High	45° Horizontal Bend	39.00	End of Run is at end of installed DI WYE at South side of Maryland Bridge	80.42
210	35702	87.76	46%	5:00	91	2	Medium	45° Horizontal Bend	40.14	End of Run is at end of installed DI WYE at South side of Maryland Bridge	79.27
210	35747	87.88	80%	8:30	54	1.5	Med-Low	45° Horizontal Bend	40.26	End of Run is at end of installed DI WYE at South side of Maryland Bridge	79.15
210	35799	88.01	58%	2:00	38	1	Low	45° Horizontal Bend	40.39	End of Run is at end of installed DI WYE at South side of Maryland Bridge	79.02
210	35817	88.06	78%	8:30	113	1.5	Med-Low	45° Horizontal Bend	40.44	End of Run is at end of installed DI WYE at South side of Maryland Bridge	78.97
210	35867	88.18	55%	2:30	23	1	Low	45° Horizontal Bend	40.56	End of Run is at end of installed DI WYE at South side of Maryland Bridge	78.85
210	35894	88.25	50%	1:00	26	1.5	Med-Low	45° Horizontal Bend	40.63	End of Run is at end of installed DI WYE at South side of Maryland Bridge	78.78

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
210	35914	88.30	46%	3:30	22	2	Medium	45° Horizontal Bend	40.68	End of Run is at end of installed DI WYE at South side of Maryland Bridge	78.73
210	35964	88.43	56%	9:30	50	3	High	45° Horizontal Bend	40.81	End of Run is at end of installed DI WYE at South side of Maryland Bridge	78.60
210	36009	88.54	35%	11:30	117	1	Low	45° Horizontal Bend	40.92	End of Run is at end of installed DI WYE at South side of Maryland Bridge	78.49
210	36036	88.61	28%	9:00	81	3	High	45° Horizontal Bend	40.99	End of Run is at end of installed DI WYE at South side of Maryland Bridge	78.42
210	36092	88.75	43%	5:30	22	3	High	45° Horizontal Bend	41.13	End of Run is at end of installed DI WYE at South side of Maryland Bridge	78.28
210	36135	88.86	63%	5:30	52	1	Low	45° Horizontal Bend	41.24	End of Run is at end of installed DI WYE at South side of Maryland Bridge	78.17
210	36140	88.88	74%	12:00	74	1.5	Med-Low	45° Horizontal Bend	41.26	End of Run is at end of installed DI WYE at South side of Maryland Bridge	78.15
210	36184	88.99	46%	9:00	24	3	High	45° Horizontal Bend	41.37	End of Run is at end of installed DI WYE at South side of Maryland Bridge	78.04
210	36225	89.09	39%	9:00	40	3	High	45° Horizontal Bend	41.47	End of Run is at end of installed DI WYE at South side of Maryland Bridge	77.94
210	36241	89.13	65%	2:00	64	1.5	Med-Low	45° Horizontal Bend	41.51	End of Run is at end of installed DI WYE at South side of Maryland Bridge	77.90
210	36263	89.19	66%	12:00	36	1.5	Med-Low	45° Horizontal Bend	41.57	End of Run is at end of installed DI WYE	77.84

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
										at South side of Maryland Bridge	
210	36288	89.25	84%	9:00	120	1.5	Med-Low	45° Horizontal Bend	41.63	End of Run is at end of installed DI WYE at South side of Maryland Bridge	77.78
210	36440	89.64	31%	7:30	52	3	High	45° Horizontal Bend	42.02	End of Run is at end of installed DI WYE at South side of Maryland Bridge	77.39
210	36567	89.96	53%	6:30	88	3	High	45° Horizontal Bend	42.34	End of Run is at end of installed DI WYE at South side of Maryland Bridge	77.07
210	36644	90.16	47%	6:00	26	3	High	45° Horizontal Bend	42.54	End of Run is at end of installed DI WYE at South side of Maryland Bridge	76.87
210	36646	90.16	47%	9:00	50	3	High	45° Horizontal Bend	42.54	End of Run is at end of installed DI WYE at South side of Maryland Bridge	76.87
210	36745	90.41	74%	9:00	52	2	Medium	45° Horizontal Bend	42.79	End of Run is at end of installed DI WYE at South side of Maryland Bridge	76.62
210	36812	90.58	54%	7:30	76	3	High	45° Horizontal Bend	42.96	End of Run is at end of installed DI WYE at South side of Maryland Bridge	76.45
210	36877	90.75	26%	6:00	48	3	High	45° Horizontal Bend	43.13	End of Run is at end of installed DI WYE at South side of Maryland Bridge	76.28
210	36904	90.82	57%	9:30	82	1.5	Med-Low	45° Horizontal Bend	43.20	End of Run is at end of installed DI WYE at South side of Maryland Bridge	76.21
210	36935	90.89	48%	5:30	70	3	High	45° Horizontal Bend	43.27	End of Run is at end of installed DI WYE at South side of Maryland Bridge	76.14

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
210	36985	91.02	60%	7:30	92	2	Medium	45° Horizontal Bend	43.40	End of Run is at end of installed DI WYE at South side of Maryland Bridge	76.01
210	37034	91.15	50%	5:00	21	2.5	Med-High	45° Horizontal Bend	43.53	End of Run is at end of installed DI WYE at South side of Maryland Bridge	75.88
210	37099	91.31	67%	5:00	90	3	High	45° Horizontal Bend	43.69	End of Run is at end of installed DI WYE at South side of Maryland Bridge	75.72
220	37236	91.66	78%	8:30	41	2	Medium	45° Horizontal Bend	44.04	End of Run is at end of installed DI WYE at South side of Maryland Bridge	75.37
220	37268	91.74	35%	4:00	26	1	Low	45° Horizontal Bend	44.12	End of Run is at end of installed DI WYE at South side of Maryland Bridge	75.29
220	37338	91.92	59%	4:30	60	2.5	Med-High	45° Horizontal Bend	44.30	End of Run is at end of installed DI WYE at South side of Maryland Bridge	75.11
220	37382	92.03	61%	6:30	44	2	Medium	45° Horizontal Bend	44.41	End of Run is at end of installed DI WYE at South side of Maryland Bridge	75.00
220	37527	92.40	74%	6:30	119	3	High	45° Horizontal Bend	44.78	End of Run is at end of installed DI WYE at South side of Maryland Bridge	74.63
220	37690	92.81	56%	5:00	28	2	Medium	45° Horizontal Bend	45.19	End of Run is at end of installed DI WYE at South side of Maryland Bridge	74.22
220	37786	93.06	67%	5:00	123	3	High	45° Horizontal Bend	45.44	End of Run is at end of installed DI WYE at South side of Maryland Bridge	73.97
220	37810	93.12	64%	9:00	46	2	Medium	45° Horizontal Bend	45.50	End of Run is at end of installed DI WYE	73.91

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
										at South side of Maryland Bridge	
220	37953	93.48	61%	0:30	94	2	Medium	45° Horizontal Bend	45.86	End of Run is at end of installed DI WYE at South side of Maryland Bridge	73.55
220	37959	93.50	59%	9:00	62	2.5	Med-High	45° Horizontal Bend	45.88	End of Run is at end of installed DI WYE at South side of Maryland Bridge	73.53
220	37998	93.59	31%	5:30	25	3	High	45° Horizontal Bend	45.97	End of Run is at end of installed DI WYE at South side of Maryland Bridge	73.44
220	38071	93.78	67%	5:00	33	1	Low	45° Horizontal Bend	46.16	End of Run is at end of installed DI WYE at South side of Maryland Bridge	73.25
220	38089	93.83	50%	8:00	64	1.5	Med-Low	45° Horizontal Bend	46.21	End of Run is at end of installed DI WYE at South side of Maryland Bridge	73.20
220	38207	94.13	69%	6:30	86	1	Low	45° Horizontal Bend	46.51	End of Run is at end of installed DI WYE at South side of Maryland Bridge	72.90
220	38269	94.28	56%	8:30	52	2	Medium	45° Horizontal Bend	46.66	End of Run is at end of installed DI WYE at South side of Maryland Bridge	72.75
220	38346	94.48	38%	11:00	127	3	High	45° Horizontal Bend	46.86	End of Run is at end of installed DI WYE at South side of Maryland Bridge	72.55
220	38379	94.56	45%	4:30	62	3	High	45° Horizontal Bend	46.94	End of Run is at end of installed DI WYE at South side of Maryland Bridge	72.47
220	38411	94.64	63%	8:30	66	3	High	45° Horizontal Bend	47.02	End of Run is at end of installed DI WYE at South side of Maryland Bridge	72.39

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
220	38429	94.69	68%	2:00	48	1.5	Med-Low	45° Horizontal Bend	47.07	End of Run is at end of installed DI WYE at South side of Maryland Bridge	72.34
220	38480	94.82	47%	9:00	88	3	High	45° Horizontal Bend	47.20	End of Run is at end of installed DI WYE at South side of Maryland Bridge	72.21
220	38571	95.05	56%	12:00	28	1	Low	45° Horizontal Bend	47.43	End of Run is at end of installed DI WYE at South side of Maryland Bridge	71.98
220	38622	95.18	70%	11:30	86	2	Medium	45° Horizontal Bend	47.56	End of Run is at end of installed DI WYE at South side of Maryland Bridge	71.85
220	38782	95.59	59%	5:00	44	1	Low	45° Horizontal Bend	47.97	End of Run is at end of installed DI WYE at South side of Maryland Bridge	71.44
220	39128	96.47	63%	0:30	121	3	High	45° Horizontal Bend	48.85	End of Run is at end of installed DI WYE at South side of Maryland Bridge	70.56
220	39205	96.66	42%	9:00	23	3	High	45° Horizontal Bend	49.04	End of Run is at end of installed DI WYE at South side of Maryland Bridge	70.37
220	39258	96.80	54%	5:00	64	1	Low	45° Horizontal Bend	49.18	End of Run is at end of installed DI WYE at South side of Maryland Bridge	70.23
220	39340	97.00	34%	9:30	28	2.5	Med-High	45° Horizontal Bend	49.38	End of Run is at end of installed DI WYE at South side of Maryland Bridge	70.03
220	39375	97.09	54%	1:00	54	2	Medium	45° Horizontal Bend	49.47	End of Run is at end of installed DI WYE at South side of Maryland Bridge	69.94
220	39375	97.09	54%	7:00	42	2.5	Med-High	45° Horizontal Bend	49.47	End of Run is at end of installed DI WYE	69.94

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
										at South side of Maryland Bridge	
220	39409	97.18	59%	9:00	78	2.5	Med-High	45° Horizontal Bend	49.56	End of Run is at end of installed DI WYE at South side of Maryland Bridge	69.85
230	39681	97.87	49%	10:30	60	2.5	Med-High	45° Horizontal Bend	50.25	End of Run is at end of installed DI WYE at South side of Maryland Bridge	69.16
230	39685	97.88	34%	5:00	64	3	High	45° Horizontal Bend	50.26	End of Run is at end of installed DI WYE at South side of Maryland Bridge	69.15
230	39799	98.17	51%	12:00	97	3	High	45° Horizontal Bend	50.55	End of Run is at end of installed DI WYE at South side of Maryland Bridge	68.86
230	39802	98.18	44%	3:00	97	3	High	45° Horizontal Bend	50.56	End of Run is at end of installed DI WYE at South side of Maryland Bridge	68.85
230	39858	98.32	76%	1:30	35	1	Low	45° Horizontal Bend	50.70	End of Run is at end of installed DI WYE at South side of Maryland Bridge	68.71
230	39877	98.37	61%	7:30	20	1	Low	45° Horizontal Bend	50.75	End of Run is at end of installed DI WYE at South side of Maryland Bridge	68.66
230	39987	98.65	41%	9:30	58	2.5	Med-High	45° Horizontal Bend	51.03	End of Run is at end of installed DI WYE at South side of Maryland Bridge	68.38
230	40045	98.79	59%	11:00	27	1.5	Med-Low	45° Horizontal Bend	51.17	End of Run is at end of installed DI WYE at South side of Maryland Bridge	68.24
230	40176	99.13	71%	3:30	17	1.5	Med-Low	45° Horizontal Bend	51.51	End of Run is at end of installed DI WYE at South side of Maryland Bridge	67.90

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
230	40181	99.14	73%	10:00	46	1	Low	45° Horizontal Bend	51.52	End of Run is at end of installed DI WYE at South side of Maryland Bridge	67.89
230	40324	99.50	70%	4:30	98	2.5	Med-High	45° Horizontal Bend	51.88	End of Run is at end of installed DI WYE at South side of Maryland Bridge	67.53
230	40350	99.57	63%	7:30	60	3	High	45° Horizontal Bend	51.95	End of Run is at end of installed DI WYE at South side of Maryland Bridge	67.46
230	40447	99.82	41%	9:00	32	2.5	Med-High	45° Horizontal Bend	52.20	End of Run is at end of installed DI WYE at South side of Maryland Bridge	67.21
230	40465	99.86	66%	12:00	36	1	Low	45° Horizontal Bend	52.24	End of Run is at end of installed DI WYE at South side of Maryland Bridge	67.17
230	40512	99.98	40%	11:30	32	2	Medium	45° Horizontal Bend	52.36	End of Run is at end of installed DI WYE at South side of Maryland Bridge	67.05
230	40594	100.19	73%	10:00	70	1	Low	45° Horizontal Bend	52.57	End of Run is at end of installed DI WYE at South side of Maryland Bridge	66.84
230	40752	100.59	62%	12:00	101	2	Medium	45° Horizontal Bend	52.97	End of Run is at end of installed DI WYE at South side of Maryland Bridge	66.44
230	40795	100.70	57%	8:00	22	1.5	Med-Low	45° Horizontal Bend	53.08	End of Run is at end of installed DI WYE at South side of Maryland Bridge	66.33
230	40828	100.78	59%	9:00	48	1	Low	45° Horizontal Bend	53.16	End of Run is at end of installed DI WYE at South side of Maryland Bridge	66.25
230	40838	100.81	81%	1:00	120	2	Medium	45° Horizontal Bend	53.19	End of Run is at end of installed DI WYE	66.22

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
										at South side of Maryland Bridge	
230	40872	100.89	74%	2:00	37	1.5	Med-Low	45° Horizontal Bend	53.27	End of Run is at end of installed DI WYE at South side of Maryland Bridge	66.14
230	40876	100.91	64%	1:30	60	1	Low	45° Horizontal Bend	53.29	End of Run is at end of installed DI WYE at South side of Maryland Bridge	66.12
230	40895	100.95	73%	4:30	76	2	Medium	45° Horizontal Bend	53.33	End of Run is at end of installed DI WYE at South side of Maryland Bridge	66.08
230	41067	101.39	53%	0:30	82	2.5	Med-High	45° Horizontal Bend	53.77	End of Run is at end of installed DI WYE at South side of Maryland Bridge	65.64
230	41193	101.71	57%	0:30	84	3	High	45° Horizontal Bend	54.09	End of Run is at end of installed DI WYE at South side of Maryland Bridge	65.32
230	41314	102.02	59%	8:00	56	2	Medium	45° Horizontal Bend	54.40	End of Run is at end of installed DI WYE at South side of Maryland Bridge	65.01
230	41512	102.52	83%	10:30	118	1	Low	45° Horizontal Bend	54.90	End of Run is at end of installed DI WYE at South side of Maryland Bridge	64.51
230	41584	102.70	46%	8:00	58	3	High	45° Horizontal Bend	55.08	End of Run is at end of installed DI WYE at South side of Maryland Bridge	64.33
230	41586	102.71	47%	0:30	26	1.5	Med-Low	45° Horizontal Bend	55.09	End of Run is at end of installed DI WYE at South side of Maryland Bridge	64.32
230	41628	102.82	79%	12:00	29	1	Low	45° Horizontal Bend	55.20	End of Run is at end of installed DI WYE at South side of Maryland Bridge	64.21

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
230	41732	103.08	54%	0:30	84	2	Medium	45° Horizontal Bend	55.46	End of Run is at end of installed DI WYE at South side of Maryland Bridge	63.95
230	41860	103.40	83%	0:30	118	1	Low	45° Horizontal Bend	55.78	End of Run is at end of installed DI WYE at South side of Maryland Bridge	63.63
240	42146	104.13	40%	7:30	109	2.5	Med-High	45° Horizontal Bend	56.51	End of Run is at end of installed DI WYE at South side of Maryland Bridge	62.90
240	42191	104.25	70%	12:00	104	3	High	45° Horizontal Bend	56.63	End of Run is at end of installed DI WYE at South side of Maryland Bridge	62.78
240	42446	104.89	66%	6:30	92	2.5	Med-High	45° Horizontal Bend	57.27	End of Run is at end of installed DI WYE at South side of Maryland Bridge	62.14
240	42459	104.93	50%	1:00	22	2	Medium	45° Horizontal Bend	57.31	End of Run is at end of installed DI WYE at South side of Maryland Bridge	62.10
240	42514	105.07	64%	0:30	109	2	Medium	45° Horizontal Bend	57.45	End of Run is at end of installed DI WYE at South side of Maryland Bridge	61.96
240	42791	105.77	63%	0:30	125	2.5	Med-High	45° Horizontal Bend	58.15	End of Run is at end of installed DI WYE at South side of Maryland Bridge	61.26
240	42919	106.09	49%	0:30	68	3	High	45° Horizontal Bend	58.47	End of Run is at end of installed DI WYE at South side of Maryland Bridge	60.94
240	43082	106.51	72%	5:00	92	1.5	Med-Low	45° Horizontal Bend	58.89	End of Run is at end of installed DI WYE at South side of Maryland Bridge	60.52
240	43370	107.24	67%	1:30	147	2.5	Med-High	45° Horizontal Bend	59.62	End of Run is at end of installed DI WYE	59.79

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
										at South side of Maryland Bridge	
240	43455	107.46	77%	1:30	41	1	Low	45° Horizontal Bend	59.84	End of Run is at end of installed DI WYE at South side of Maryland Bridge	59.57
240	43489	107.54	56%	11:30	46	2	Medium	45° Horizontal Bend	59.92	End of Run is at end of installed DI WYE at South side of Maryland Bridge	59.49
240	43519	107.62	75%	7:00	50	1	Low	45° Horizontal Bend	60.00	End of Run is at end of installed DI WYE at South side of Maryland Bridge	59.41
240	43777	108.27	72%	6:30	127	3	High	45° Horizontal Bend	60.65	End of Run is at end of installed DI WYE at South side of Maryland Bridge	58.76
240	43955	108.73	54%	0:30	113	1.5	Med-Low	45° Horizontal Bend	61.11	End of Run is at end of installed DI WYE at South side of Maryland Bridge	58.30
240	43981	108.79	67%	5:30	60	3	High	45° Horizontal Bend	61.17	End of Run is at end of installed DI WYE at South side of Maryland Bridge	58.24
240	44082	109.05	64%	5:30	21	2	Medium	45° Horizontal Bend	61.43	End of Run is at end of installed DI WYE at South side of Maryland Bridge	57.98
240	44126	109.16	60%	0:30	52	1	Low	45° Horizontal Bend	61.54	End of Run is at end of installed DI WYE at South side of Maryland Bridge	57.87
240	44328	109.67	56%	11:30	84	1	Low	45° Horizontal Bend	62.05	End of Run is at end of installed DI WYE at South side of Maryland Bridge	57.36
240	44359	109.75	58%	4:00	44	1	Low	45° Horizontal Bend	62.13	End of Run is at end of installed DI WYE at South side of Maryland Bridge	57.28

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
250	44553	110.24	65%	1:30	84	2	Medium	45° Horizontal Bend	62.62	End of Run is at end of installed DI WYE at South side of Maryland Bridge	56.79
250	44643	110.47	42%	11:30	115	1.5	Med-Low	45° Horizontal Bend	62.85	End of Run is at end of installed DI WYE at South side of Maryland Bridge	56.56
250	44736	110.71	65%	8:30	101	3	High	45° Horizontal Bend	63.09	End of Run is at end of installed DI WYE at South side of Maryland Bridge	56.32
250	44801	110.87	68%	1:00	60	2	Medium	45° Horizontal Bend	63.25	End of Run is at end of installed DI WYE at South side of Maryland Bridge	56.16
250	44852	111.00	67%	8:00	46	2	Medium	45° Horizontal Bend	63.38	End of Run is at end of installed DI WYE at South side of Maryland Bridge	56.03
250	44863	111.03	67%	0:30	101	2	Medium	45° Horizontal Bend	63.41	End of Run is at end of installed DI WYE at South side of Maryland Bridge	56.00
250	44935	111.21	37%	12:00	72	3	High	45° Horizontal Bend	63.59	End of Run is at end of installed DI WYE at South side of Maryland Bridge	55.82
250	45018	111.43	64%	0:30	135	3	High	45° Horizontal Bend	63.81	End of Run is at end of installed DI WYE at South side of Maryland Bridge	55.60
250	45096	111.62	72%	5:30	19	1.5	Med-Low	45° Horizontal Bend	64.00	End of Run is at end of installed DI WYE at South side of Maryland Bridge	55.41
250	45271	112.07	72%	7:00	121	2.5	Med-High	45° Horizontal Bend	64.45	End of Run is at end of installed DI WYE at South side of Maryland Bridge	54.96
250	45280	112.09	87%	1:00	152	2.5	Med-High	45° Horizontal Bend	64.47	End of Run is at end of installed DI WYE	54.94

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
										at South side of Maryland Bridge	
250	45361	112.30	72%	1:00	78	2	Medium	45° Horizontal Bend	64.68	End of Run is at end of installed DI WYE at South side of Maryland Bridge	54.73
250	45518	112.70	63%	7:00	121	3	High	45° Horizontal Bend	65.08	End of Run is at end of installed DI WYE at South side of Maryland Bridge	54.33
250	45585	112.87	66%	1:30	37	1.5	Med-Low	45° Horizontal Bend	65.25	End of Run is at end of installed DI WYE at South side of Maryland Bridge	54.16
250	45585	112.87	74%	10:00	27	1.5	Med-Low	45° Horizontal Bend	65.25	End of Run is at end of installed DI WYE at South side of Maryland Bridge	54.16
250	45648	113.03	51%	7:00	68	3	High	45° Horizontal Bend	65.41	End of Run is at end of installed DI WYE at South side of Maryland Bridge	54.00
250	45666	113.07	57%	1:00	105	3	High	45° Horizontal Bend	65.45	End of Run is at end of installed DI WYE at South side of Maryland Bridge	53.96
250	45758	113.31	62%	7:30	54	1	Low	45° Horizontal Bend	65.69	End of Run is at end of installed DI WYE at South side of Maryland Bridge	53.72
250	45779	113.36	55%	11:00	74	2.5	Med-High	45° Horizontal Bend	65.74	End of Run is at end of installed DI WYE at South side of Maryland Bridge	53.67
250	45926	113.73	67%	10:30	96	2.5	Med-High	45° Horizontal Bend	66.11	End of Run is at end of installed DI WYE at South side of Maryland Bridge	53.30
250	45945	113.78	73%	5:30	62	2.5	Med-High	45° Horizontal Bend	66.16	End of Run is at end of installed DI WYE at South side of Maryland Bridge	53.25

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
250	46052	114.05	79%	0:30	90	2.5	Med-High	45° Horizontal Bend	66.43	End of Run is at end of installed DI WYE at South side of Maryland Bridge	52.98
250	46069	114.10	67%	4:00	36	1.5	Med-Low	45° Horizontal Bend	66.48	End of Run is at end of installed DI WYE at South side of Maryland Bridge	52.93
250	46300	114.68	75%	11:00	78	2	Medium	45° Horizontal Bend	67.06	End of Run is at end of installed DI WYE at South side of Maryland Bridge	52.35
250	46414	114.97	68%	11:30	92	2.5	Med-High	45° Horizontal Bend	67.35	End of Run is at end of installed DI WYE at South side of Maryland Bridge	52.06
260	47222	117.02	73%	10:30	102	2.5	Med-High	45° Horizontal Bend	69.40	End of Run is at end of installed DI WYE at South side of Maryland Bridge	50.01
260	47319	117.27	71%	6:00	100	3	High	45° Horizontal Bend	69.65	End of Run is at end of installed DI WYE at South side of Maryland Bridge	49.76
260	47851	118.62	73%	10:00	80	3	High	45° Horizontal Bend	71.00	End of Run is at end of installed DI WYE at South side of Maryland Bridge	48.41
260	48285	119.72	62%	10:30	107	2	Medium	45° Horizontal Bend	72.10	End of Run is at end of installed DI WYE at South side of Maryland Bridge	47.31
260	48690	120.75	72%	5:30	107	2.5	Med-High	45° Horizontal Bend	73.13	End of Run is at end of installed DI WYE at South side of Maryland Bridge	46.28
260	49237	122.14	92%	8:30	128	1	Low	45° Horizontal Bend	74.52	End of Run is at end of installed DI WYE at South side of Maryland Bridge	44.89
260	49451	122.69	61%	5:30	84	3	High	45° Horizontal Bend	75.07	End of Run is at end of installed DI WYE	44.34

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
										at South side of Maryland Bridge	
270	49643	123.17	66%	3:00	27	1	Low	45° Horizontal Bend	75.55	End of Run is at end of installed DI WYE at South side of Maryland Bridge	43.86
270	49988	124.05	76%	6:00	151	2.5	Med-High	45° Horizontal Bend	76.43	End of Run is at end of installed DI WYE at South side of Maryland Bridge	42.98
270	50600	125.60	58%	1:30	123	2.5	Med-High	45° Horizontal Bend	77.98	End of Run is at end of installed DI WYE at South side of Maryland Bridge	41.43
270	50767	126.03	87%	8:00	136	2	Medium	45° Horizontal Bend	78.41	End of Run is at end of installed DI WYE at South side of Maryland Bridge	41.00
270	50923	126.42	64%	7:00	36	1	Low	45° Horizontal Bend	78.80	End of Run is at end of installed DI WYE at South side of Maryland Bridge	40.61
270	51008	126.64	62%	6:00	44	2	Medium	45° Horizontal Bend	79.02	End of Run is at end of installed DI WYE at South side of Maryland Bridge	40.39
270	51168	127.05	72%	1:30	54	1	Low	45° Horizontal Bend	79.43	End of Run is at end of installed DI WYE at South side of Maryland Bridge	39.98
270	51658	128.29	81%	1:30	179	2.5	Med-High	45° Horizontal Bend	80.67	End of Run is at end of installed DI WYE at South side of Maryland Bridge	38.74
270	51774	128.59	50%	5:00	72	2.5	Med-High	45° Horizontal Bend	80.97	End of Run is at end of installed DI WYE at South side of Maryland Bridge	38.44
270	51993	129.14	77%	3:00	62	2	Medium	45° Horizontal Bend	81.52	End of Run is at end of installed DI WYE at South side of Maryland Bridge	37.89

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
280	52324	129.98	60%	6:30	31	2.5	Med-High	45° Horizontal Bend	82.36	End of Run is at end of installed DI WYE at South side of Maryland Bridge	37.05
280	52419	130.22	51%	5:00	58	2.5	Med-High	45° Horizontal Bend	82.60	End of Run is at end of installed DI WYE at South side of Maryland Bridge	36.81
280	53088	131.92	62%	8:30	50	2.5	Med-High	45° Horizontal Bend	84.30	End of Run is at end of installed DI WYE at South side of Maryland Bridge	35.11
280	53149	132.08	59%	3:00	52	2	Medium	45° Horizontal Bend	84.46	End of Run is at end of installed DI WYE at South side of Maryland Bridge	34.95
280	53253	132.34	67%	5:00	27	1.5	Med-Low	45° Horizontal Bend	84.72	End of Run is at end of installed DI WYE at South side of Maryland Bridge	34.69
280	53316	132.50	61%	6:00	18	1.5	Med-Low	45° Horizontal Bend	84.88	End of Run is at end of installed DI WYE at South side of Maryland Bridge	34.53
280	53376	132.66	68%	4:30	101	3	High	45° Horizontal Bend	85.04	End of Run is at end of installed DI WYE at South side of Maryland Bridge	34.37
280	53578	133.17	59%	4:00	123	3	High	45° Horizontal Bend	85.55	End of Run is at end of installed DI WYE at South side of Maryland Bridge	33.86
280	53671	133.40	79%	5:00	37	2	Medium	45° Horizontal Bend	85.78	End of Run is at end of installed DI WYE at South side of Maryland Bridge	33.63
280	54373	135.19	41%	6:00	72	2	Medium	45° Horizontal Bend	87.57	End of Run is at end of installed DI WYE at South side of Maryland Bridge	31.84
290	54724	136.08	73%	5:00	119	3	High	45° Horizontal Bend	88.46	End of Run is at end of installed DI WYE	30.95

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
										at South side of Maryland Bridge	
290	54861	136.43	52%	5:00	82	3	High	45° Horizontal Bend	88.81	End of Run is at end of installed DI WYE at South side of Maryland Bridge	30.60
290	55044	136.89	67%	5:00	52	1.5	Med-Low	45° Horizontal Bend	89.27	End of Run is at end of installed DI WYE at South side of Maryland Bridge	30.14
290	55143	137.14	59%	5:30	139	3	High	45° Horizontal Bend	89.52	End of Run is at end of installed DI WYE at South side of Maryland Bridge	29.89
290	55221	137.34	72%	5:00	165	3	High	45° Horizontal Bend	89.72	End of Run is at end of installed DI WYE at South side of Maryland Bridge	29.69
290	55353	137.68	77%	5:30	68	3	High	45° Horizontal Bend	90.06	End of Run is at end of installed DI WYE at South side of Maryland Bridge	29.35
290	55526	138.12	63%	5:00	62	2.5	Med-High	45° Horizontal Bend	90.50	End of Run is at end of installed DI WYE at South side of Maryland Bridge	28.91
290	55639	138.40	78%	5:30	96	2.5	Med-High	45° Horizontal Bend	90.78	End of Run is at end of installed DI WYE at South side of Maryland Bridge	28.63
290	55758	138.71	48%	5:00	70	2	Medium	45° Horizontal Bend	91.09	End of Run is at end of installed DI WYE at South side of Maryland Bridge	28.32
290	56112	139.60	47%	5:00	115	2.5	Med-High	45° Horizontal Bend	91.98	End of Run is at end of installed DI WYE at South side of Maryland Bridge	27.43
290	56220	139.88	37%	4:30	25	2.5	Med-High	45° Horizontal Bend	92.26	End of Run is at end of installed DI WYE at South side of Maryland Bridge	27.15

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
290	56386	140.30	59%	3:30	135	2.5	Med-High	45° Horizontal Bend	92.68	End of Run is at end of installed DI WYE at South side of Maryland Bridge	26.73
290	56466	140.50	66%	4:30	56	2	Medium	45° Horizontal Bend	92.88	End of Run is at end of installed DI WYE at South side of Maryland Bridge	26.53
290	56517	140.63	36%	0:30	25	2.5	Med-High	45° Horizontal Bend	93.01	End of Run is at end of installed DI WYE at South side of Maryland Bridge	26.40
290	56533	140.67	63%	4:00	76	2.5	Med-High	45° Horizontal Bend	93.05	End of Run is at end of installed DI WYE at South side of Maryland Bridge	26.36
290	56590	140.82	40%	0:30	72	3	High	45° Horizontal Bend	93.20	End of Run is at end of installed DI WYE at South side of Maryland Bridge	26.21
290	56617	140.89	73%	4:00	94	3	High	45° Horizontal Bend	93.27	End of Run is at end of installed DI WYE at South side of Maryland Bridge	26.14
290	56641	140.95	48%	0:30	156	3	High	45° Horizontal Bend	93.33	End of Run is at end of installed DI WYE at South side of Maryland Bridge	26.08
290	56805	141.36	22%	0:30	160	4	Very High	45° Horizontal Bend	93.74	End of Run is at end of installed DI WYE at South side of Maryland Bridge	25.67
290	56825	141.42	82%	6:00	207	2	Medium	45° Horizontal Bend	93.80	End of Run is at end of installed DI WYE at South side of Maryland Bridge	25.61
290	56895	141.59	26%	0:30	89	3	High	45° Horizontal Bend	93.97	End of Run is at end of installed DI WYE at South side of Maryland Bridge	25.44
290	56955	141.75	32%	9:30	28	1.5	Med-Low	45° Horizontal Bend	94.13	End of Run is at end of installed DI WYE	25.28

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
										at South side of Maryland Bridge	
290	56998	141.85	38%	0:30	144	4	Very High	45° Horizontal Bend	94.23	End of Run is at end of installed DI WYE at South side of Maryland Bridge	25.18
300	57488	143.10	76%	4:00	86	2	Medium	45° Horizontal Bend	95.48	End of Run is at end of installed DI WYE at South side of Maryland Bridge	23.93
300	57535	143.22	54%	8:00	60	1.5	Med-Low	45° Horizontal Bend	95.60	End of Run is at end of installed DI WYE at South side of Maryland Bridge	23.81
300	57596	143.37	62%	7:30	44	1.5	Med-Low	45° Horizontal Bend	95.75	End of Run is at end of installed DI WYE at South side of Maryland Bridge	23.66
300	57624	143.44	36%	5:30	42	1.5	Med-Low	45° Horizontal Bend	95.82	End of Run is at end of installed DI WYE at South side of Maryland Bridge	23.59
300	57673	143.57	42%	0:30	121	3	High	45° Horizontal Bend	95.95	End of Run is at end of installed DI WYE at South side of Maryland Bridge	23.46
300	57841	144.00	67%	1:30	98	3	High	45° Horizontal Bend	96.38	End of Run is at end of installed DI WYE at South side of Maryland Bridge	23.03
300	57899	144.14	75%	1:30	27	1.5	Med-Low	45° Horizontal Bend	96.52	End of Run is at end of installed DI WYE at South side of Maryland Bridge	22.89
300	57919	144.19	72%	9:30	66	3	High	45° Horizontal Bend	96.57	End of Run is at end of installed DI WYE at South side of Maryland Bridge	22.84
300	57947	144.27	48%	12:00	58	3	High	45° Horizontal Bend	96.65	End of Run is at end of installed DI WYE at South side of Maryland Bridge	22.76

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
300	57999	144.40	62%	10:30	133	2.5	Med-High	45° Horizontal Bend	96.78	End of Run is at end of installed DI WYE at South side of Maryland Bridge	22.63
300	58010	144.43	42%	7:30	20	2	Medium	45° Horizontal Bend	96.81	End of Run is at end of installed DI WYE at South side of Maryland Bridge	22.60
300	58075	144.59	44%	9:30	72	3	High	45° Horizontal Bend	96.97	End of Run is at end of installed DI WYE at South side of Maryland Bridge	22.44
300	58099	144.65	65%	7:00	60	2.5	Med-High	45° Horizontal Bend	97.03	End of Run is at end of installed DI WYE at South side of Maryland Bridge	22.38
300	58202	144.91	25%	9:30	81	3	High	45° Horizontal Bend	97.29	End of Run is at end of installed DI WYE at South side of Maryland Bridge	22.12
300	58276	145.10	46%	9:30	42	2.5	Med-High	45° Horizontal Bend	97.48	End of Run is at end of installed DI WYE at South side of Maryland Bridge	21.93
300	58566	145.84	72%	3:30	86	1.5	Med-Low	45° Horizontal Bend	98.22	End of Run is at end of installed DI WYE at South side of Maryland Bridge	21.19
300	58784	146.39	4%	7:30	56	3	High	45° Horizontal Bend	98.77	End of Run is at end of installed DI WYE at South side of Maryland Bridge	20.64
300	59276	147.64	42%	8:00	40	3	High	45° Horizontal Bend	100.02	End of Run is at end of installed DI WYE at South side of Maryland Bridge	19.39
300	59296	147.69	47%	10:00	64	3	High	45° Horizontal Bend	100.07	End of Run is at end of installed DI WYE at South side of Maryland Bridge	19.34
300	59330	147.78	36%	8:00	64	3	High	45° Horizontal Bend	100.16	End of Run is at end of installed DI WYE	19.25

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
										at South side of Maryland Bridge	
300	59353	147.84	63%	10:30	72	3	High	45° Horizontal Bend	100.22	End of Run is at end of installed DI WYE at South side of Maryland Bridge	19.19
300	59409	147.98	14%	8:00	40	3	High	45° Horizontal Bend	100.36	End of Run is at end of installed DI WYE at South side of Maryland Bridge	19.05
300	59442	148.06	26%	6:30	56	3	High	45° Horizontal Bend	100.44	End of Run is at end of installed DI WYE at South side of Maryland Bridge	18.97
310	59706	148.73	78%	8:30	143	1	Low	45° Horizontal Bend	101.11	End of Run is at end of installed DI WYE at South side of Maryland Bridge	18.30
310	59799	148.97	74%	7:00	174	1	Low	45° Horizontal Bend	101.35	End of Run is at end of installed DI WYE at South side of Maryland Bridge	18.06
310	59866	149.14	78%	7:00	125	2	Medium	45° Horizontal Bend	101.52	End of Run is at end of installed DI WYE at South side of Maryland Bridge	17.89
310	60310	150.27	65%	9:00	76	2	Medium	45° Horizontal Bend	102.65	End of Run is at end of installed DI WYE at South side of Maryland Bridge	16.76
310	60531	150.83	68%	8:30	48	1	Low	45° Horizontal Bend	103.21	End of Run is at end of installed DI WYE at South side of Maryland Bridge	16.20
310	60613	151.04	72%	8:30	119	2.5	Med-High	45° Horizontal Bend	103.42	End of Run is at end of installed DI WYE at South side of Maryland Bridge	15.99
310	60850	151.64	80%	2:30	129	2	Medium	45° Horizontal Bend	104.02	End of Run is at end of installed DI WYE at South side of Maryland Bridge	15.39

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
310	60872	151.69	78%	9:00	159	2	Medium	45° Horizontal Bend	104.07	End of Run is at end of installed DI WYE at South side of Maryland Bridge	15.34
310	61051	152.15	84%	3:30	185	1.5	Med-Low	45° Horizontal Bend	104.53	End of Run is at end of installed DI WYE at South side of Maryland Bridge	14.88
310	61386	153.00	59%	4:30	153	2	Medium	45° Horizontal Bend	105.38	End of Run is at end of installed DI WYE at South side of Maryland Bridge	14.03
310	61844	154.16	78%	7:00	62	1	Low	45° Horizontal Bend	106.54	End of Run is at end of installed DI WYE at South side of Maryland Bridge	12.87
320	62190	155.04	69%	10:00	117	2	Medium	45° Horizontal Bend	107.42	End of Run is at end of installed DI WYE at South side of Maryland Bridge	11.99
320	62206	155.08	69%	6:00	54	1	Low	45° Horizontal Bend	107.46	End of Run is at end of installed DI WYE at South side of Maryland Bridge	11.95
320	62252	155.20	59%	8:00	68	1	Low	45° Horizontal Bend	107.58	End of Run is at end of installed DI WYE at South side of Maryland Bridge	11.83
320	62425	155.64	83%	6:00	152	2	Medium	45° Horizontal Bend	108.02	End of Run is at end of installed DI WYE at South side of Maryland Bridge	11.39
320	62592	156.06	79%	4:30	70	2	Medium	45° Horizontal Bend	108.44	End of Run is at end of installed DI WYE at South side of Maryland Bridge	10.97
320	62664	156.25	67%	3:00	36	1	Low	45° Horizontal Bend	108.63	End of Run is at end of installed DI WYE at South side of Maryland Bridge	10.78
320	62748	156.46	63%	7:30	101	3	High	45° Horizontal Bend	108.84	End of Run is at end of installed DI WYE	10.57

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
										at South side of Maryland Bridge	
320	62877	156.79	70%	7:00	94	1.5	Med-Low	45° Horizontal Bend	109.17	End of Run is at end of installed DI WYE at South side of Maryland Bridge	10.24
320	62957	156.99	65%	11:00	92	2	Medium	45° Horizontal Bend	109.37	End of Run is at end of installed DI WYE at South side of Maryland Bridge	10.04
320	63009	157.12	53%	10:00	42	1	Low	45° Horizontal Bend	109.50	End of Run is at end of installed DI WYE at South side of Maryland Bridge	9.91
320	63023	157.16	63%	4:00	109	3	High	45° Horizontal Bend	109.54	End of Run is at end of installed DI WYE at South side of Maryland Bridge	9.87
320	63072	157.28	88%	11:00	193	2.5	Med-High	45° Horizontal Bend	109.66	End of Run is at end of installed DI WYE at South side of Maryland Bridge	9.75
320	63108	157.37	62%	4:00	68	1	Low	45° Horizontal Bend	109.75	End of Run is at end of installed DI WYE at South side of Maryland Bridge	9.66
320	63206	157.62	84%	8:00	120	1	Low	45° Horizontal Bend	110.00	End of Run is at end of installed DI WYE at South side of Maryland Bridge	9.41
320	63277	157.80	63%	3:30	92	2	Medium	45° Horizontal Bend	110.18	End of Run is at end of installed DI WYE at South side of Maryland Bridge	9.23
320	63364	158.02	61%	8:00	78	2.5	Med-High	45° Horizontal Bend	110.40	End of Run is at end of installed DI WYE at South side of Maryland Bridge	9.01
320	63435	158.20	84%	7:00	124	2	Medium	45° Horizontal Bend	110.58	End of Run is at end of installed DI WYE at South side of Maryland Bridge	8.83

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
320	63850	159.26	70%	3:00	147	1.5	Med-Low	45° Horizontal Bend	111.64	End of Run is at end of installed DI WYE at South side of Maryland Bridge	7.77
330	64276	160.34	82%	12:00	187	2	Medium	45° Horizontal Bend	112.72	End of Run is at end of installed DI WYE at South side of Maryland Bridge	6.69
330	64368	160.57	81%	0:30	173	2.5	Med-High	45° Horizontal Bend	112.95	End of Run is at end of installed DI WYE at South side of Maryland Bridge	6.46
330	64513	160.94	64%	4:00	92	2	Medium	45° Horizontal Bend	113.32	End of Run is at end of installed DI WYE at South side of Maryland Bridge	6.09
330	64662	161.32	62%	10:00	107	3	High	45° Horizontal Bend	113.70	End of Run is at end of installed DI WYE at South side of Maryland Bridge	5.71
330	64774	161.61	85%	10:00	203	2	Medium	45° Horizontal Bend	113.99	End of Run is at end of installed DI WYE at South side of Maryland Bridge	5.42
330	64804	161.68	64%	2:30	117	3	High	45° Horizontal Bend	114.06	End of Run is at end of installed DI WYE at South side of Maryland Bridge	5.35
330	64914	161.96	87%	4:00	177	2.5	Med-High	45° Horizontal Bend	114.34	End of Run is at end of installed DI WYE at South side of Maryland Bridge	5.07
330	64956	162.07	63%	8:30	60	2	Medium	45° Horizontal Bend	114.45	End of Run is at end of installed DI WYE at South side of Maryland Bridge	4.96
330	65032	162.26	57%	3:30	42	2	Medium	45° Horizontal Bend	114.64	End of Run is at end of installed DI WYE at South side of Maryland Bridge	4.77
330	65082	162.39	75%	9:00	94	2.5	Med-High	45° Horizontal Bend	114.77	End of Run is at end of installed DI WYE	4.64

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
										at South side of Maryland Bridge	
330	65118	162.48	63%	1:00	27	2	Medium	45° Horizontal Bend	114.86	End of Run is at end of installed DI WYE at South side of Maryland Bridge	4.55
330	65179	162.63	71%	1:00	102	1.5	Med-Low	45° Horizontal Bend	115.01	End of Run is at end of installed DI WYE at South side of Maryland Bridge	4.40
330	65199	162.69	83%	7:00	185	2	Medium	45° Horizontal Bend	115.07	End of Run is at end of installed DI WYE at South side of Maryland Bridge	4.34
330	65205	162.70	65%	11:00	92	1	Low	45° Horizontal Bend	115.08	End of Run is at end of installed DI WYE at South side of Maryland Bridge	4.33
330	65288	162.91	55%	9:00	74	2	Medium	45° Horizontal Bend	115.29	End of Run is at end of installed DI WYE at South side of Maryland Bridge	4.12
330	65406	163.21	77%	2:00	70	1.5	Med-Low	45° Horizontal Bend	115.59	End of Run is at end of installed DI WYE at South side of Maryland Bridge	3.82
330	65498	163.44	75%	10:00	135	2.5	Med-High	45° Horizontal Bend	115.82	End of Run is at end of installed DI WYE at South side of Maryland Bridge	3.59
330	65532	163.53	57%	3:30	107	3	High	45° Horizontal Bend	115.91	End of Run is at end of installed DI WYE at South side of Maryland Bridge	3.50
330	65576	163.64	64%	11:00	68	1	Low	45° Horizontal Bend	116.02	End of Run is at end of installed DI WYE at South side of Maryland Bridge	3.39
330	65620	163.75	60%	7:00	68	2	Medium	45° Horizontal Bend	116.13	End of Run is at end of installed DI WYE at South side of Maryland Bridge	3.28

Table 4: Anomaly Reference Distances – 250mm (10in) Maryland Bridge Watermain Assiniboine River Crossing

* Signal Quality Index are based on signal strength and correlation with calibration defect signatures.
 ** Distances to Nearest Upstream and Downstream Features are measured from the downstream joint of the feature.

In Pipe #	Data Index	Defect Location [m] (measured from launch)	Defect Remaining Wall	Clock Position*	Axial Length (mm)	Signal Quality Index**	Confidence Level	Nearest Upstream Feature	Distance to Nearest U/S Feature (m)	Nearest Downstream Feature	Distance to Nearest D/S Feature (m)
330	65627	163.77	60%	3:00	36	2	Medium	45° Horizontal Bend	116.15	End of Run is at end of installed DI WYE at South side of Maryland Bridge	3.26
330	65668	163.88	68%	11:00	76	1	Low	45° Horizontal Bend	116.26	End of Run is at end of installed DI WYE at South side of Maryland Bridge	3.15
330	65728	164.03	80%	4:00	172	2.5	Med-High	45° Horizontal Bend	116.41	End of Run is at end of installed DI WYE at South side of Maryland Bridge	3.00
330	65728	164.03	80%	4:00	172	2.5	Med-High	45° Horizontal Bend	116.41	End of Run is at end of installed DI WYE at South side of Maryland Bridge	3.00
330	65876	164.41	78%	3:30	31	1	Low	45° Horizontal Bend	116.79	End of Run is at end of installed DI WYE at South side of Maryland Bridge	2.62
330	65893	164.45	82%	11:00	112	3	High	45° Horizontal Bend	116.83	End of Run is at end of installed DI WYE at South side of Maryland Bridge	2.58
330	65909	164.49	84%	1:00	110	2	Medium	45° Horizontal Bend	116.87	End of Run is at end of installed DI WYE at South side of Maryland Bridge	2.54
330	66231	165.31	50%	1:30	42	1	Low	45° Horizontal Bend	117.69	End of Run is at end of installed DI WYE at South side of Maryland Bridge	1.72
340	66605	166.26	74%	2:00	78	1	Low	45° Horizontal Bend	118.64	End of Run is at end of installed DI WYE at South side of Maryland Bridge	0.77

Disclaimer - PICA Corporation

Scope of Services

The agreement of PICA Corp to perform services extends only to those services provided for in writing. Under no circumstances shall such services extend beyond the performance of the requested services. It is expressly understood that all descriptions, comments and expressions of opinion reflect the opinions or observations of PICA Corp based on information and assumptions supplied by the owner/operator and are not intended nor can they be construed as representations or warranties. PICA Corp is not assuming any responsibilities of the owner/operator and the owner/operator retains complete responsibility for the engineering, manufacture, repair and use decisions as a result of the data or other information provided by PICA Corp. Nothing contained in this Agreement shall create a contractual relationship with or cause of action in favor of a third party against either the Line Owner or PICA Corp. In no event shall PICA Corp's liability in respect of the services referred to herein exceed the amount paid for such services.

Standard of Care

In performing the services provided, PICA Corp uses the degree, care, and skill ordinarily exercised under similar circumstances by others performing such services in the same or similar locality. No other warranty, expressed or implied, is made or intended by PICA Corp.

City of Winnipeg 2015: 400mm (16-in) Doucet St - Goulet St Watermain Seine River Crossing

Condition Assessment Report, Standard Analysis



PICA – Pipeline Inspection & Condition Analysis Corporation
(A Subsidiary of Russell NDT Holdings Ltd.)

**RFT ILI Tool
400mm (16-in) Steel**

**Between Doucet Street and Goulet Street
Winnipeg, MB**

PICA Project: Winnipeg 8007

Inspection Date:	October 19-24, 2015
Report Submission:	February 2, 2016
Operators:	P. Ryhanen, G. Bouchard, B. Senka, G. Hubbard, D. Barron, D. Burton
Analyst:	J. Kim
Reviewers:	J. Regala
Report Revision:	1.0

CONFIDENTIAL

City of Winnipeg:

400mm (16in) Doucet-Goulet Watermain River Crossing

Condition Assessment Report, Standard Analysis

Executive Summary

PICA, under contract with AECOM, inspected a number of Water River Crossings for the City of Winnipeg using Remote Field Testing (RFT) Technology between October 19 and 24, 2015. This report documents the results of the RFT inspection of the 400mm (16in) steel Doucet-Goulet Watermain, which crosses the Seine River. The inspected portion spans between two newly installed access wyes at Doucet St and Goulet St (refer to the line map on page 4). Technical problems with the RFT tool were encountered during the first two days, and traced to a glitch in the exciter drive unit. The final inspection was performed on October 24 after repairs and functional testing. The results reported herein document PICA’s findings.

As noted in the supplied pipeline drawings, this watermain is comprised of asbestos cement (AC) and steel pipe. Approximately 87m of the total inspected distance of 174m is comprised of AC pipes. The AC pipes are non-ferrous in nature, and are therefore excluded from this report.

The RFT analysis found the inspected steel section to be in *excellent* condition, as there was no localized wall loss detected above the threshold-of-detection for the RFT Tool.

Table 1 provides an overview of the RFT findings on 16in Doucet-Goulet Watermain River Crossing.

Table 1: Feature Indication Summary	
Inspected Length	173.82m* <small>*includes ~86.60m of asbestos cement pipe</small>
Number of Steel Pipe Sections:	15
Number of Analyzed Steel Pipe Sections:	15
Average Wall Thickness:	100%
Number of Steel pipes without localized wall loss indications:	15
Number of Steel pipes with localized wall loss indications:	0
• Number of Steel pipes in ‘good’ condition (>75% RW):	15
• Number of Steel pipes in ‘fair’ condition (50-74% RW):	0
• Number of Steel pipes in ‘poor’ condition (<50% RW):	0
Total number of wall loss indications reported:	0
Number of Regular Girth Welds:	7
Number of Flange Connections:	8
Number of Elbows:	2

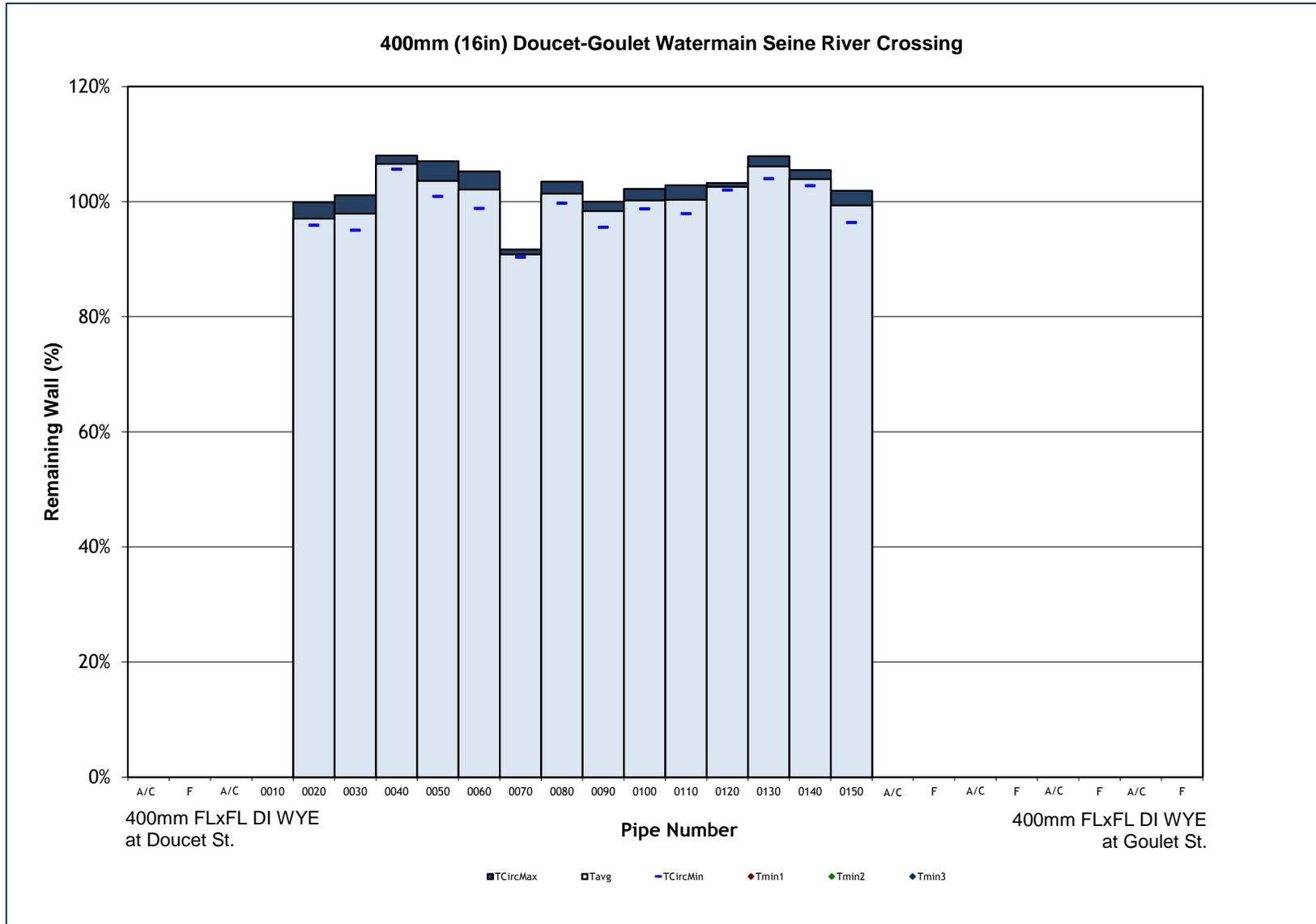


Figure 1: Structural Condition Assessment Summary – 400mm (16in) Goulet-Doucet Water Main

Table 2: Inspection Overview

Client:	City of Winnipeg	Location:	Winnipeg, MB
Line Name:	Doucet-Goulet Watermain River Crossing	Pipe Diameter:	400mm (16in)
Material:	Asbestos Cement (86.60m) Steel (87.22m)	Year Installed:	1953
Nominal Wall Thickness:	Unknown <i>*Based on inspection frequency used, it is believed to be thicker than 0.375in.</i>	Break History:	None provided
Inspection Date (s):	October 19 – 20 & 24, 2015		
Inspected Length:	173.8m		
Lead Technician:	P. Ryhanen	Technician(s):	G. Bouchard, B. Senka, G Hubbard, D. Barron, D. Burton
Launch Access:	400mm FLxFL DI WYE at Doucet St		
Retrieve Access:	400mm FLxFL DI WYE at Goulet St		

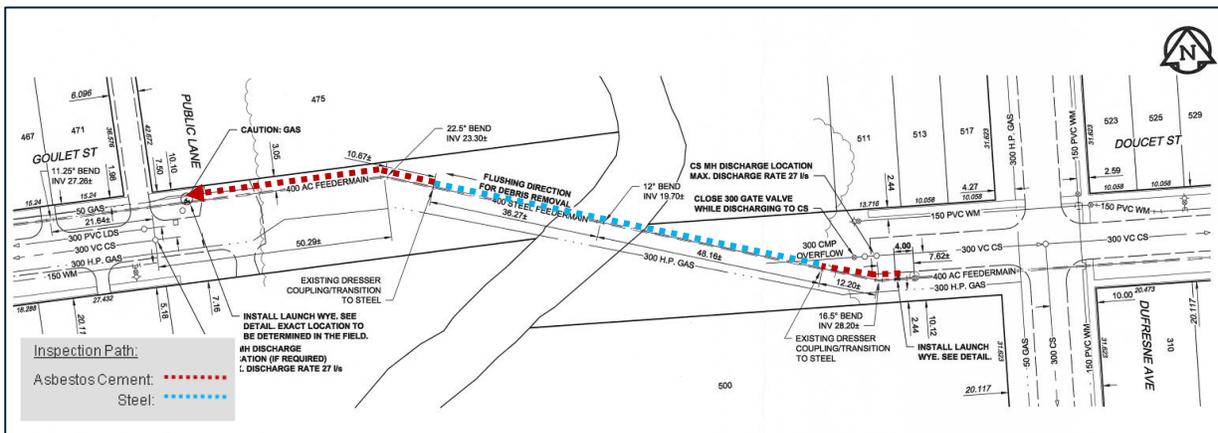
Operational Comments:

October 19 & 20, 2015

Initial inspections were performed but only 15% of the length logged, due to poor data quality. The cause was later traced to the exciter coil drive, which had failed shortly into the first run.

October 24, 2015

- 7:00am: Arrived at Tri-Cor to prepare RFT tool.
- 11:30am: Travel to site and set up to launch from Goulet St site.
- 1:00pm: First run was performed using the primary frequency setting.
Once the tool reached the Doucet St end, retrieve run commenced.
Difficulties were encountered attempting to extract the tool through the Goulet St wye.
Tool was pulled back eastward in order to extract it through the Doucet St wye.
- 2:00pm: A second RFT run was launched from the East side.
- 4:30pm: Second run finished. The data was downloaded and the quality assessed immediately.
- 6:30pm: A third RFT run using a different frequency setting based on the previous two runs was performed, again from the Doucet St side.
- 9:30pm: Tool arrived and site broken down.



Analysis Results

Location Reporting, Pipe Lengths & Features

The total logged distance for the 16in Doucet-Goulet Watermain Seine River Crossing was 173.82m. This distance includes approximately 86.60m of asbestos cement (AC) piping that exists at both ends of the main. More specifically, a total of 16.15m of AC piping is found in the east end near Doucet St and a longer 70.45m AC section in the west end near Goulet St. Asbestos cement piping is non-ferrous in nature and therefore was excluded from the analysis.

The zero datum point was set at the west end flange of the newly installed 400mm FLxFL DI WYE at Doucet St. The inspection started on the east side of Seine River and ended on the west side near Goulet St.

The average length of the common and standard steel pipe is 6.22m (20.41ft) with shorter pieces adjacent to the asbestos cement transition points. The RFT analysis identified two different joint connection types in the steel section: welded and flanged connections. Of all the flanged connections, the flanges located at 6.38m and 66.62m appear differently than the rest. The RFT data also identified three significant deflections in the inspection section: 16° elbow, 12° vertical deflection and 22° elbow. Both elbows and the vertical deflection were confirmed using pipeline documents provided by the client.

General Wall Thickness

Pipe sections longer than 1.5m were analysed to obtain the average remaining wall thickness calculated over the length of the section. The measured average wall thickness for the steel section of the 16in Doucet-Goulet Watermain River Crossing is 100.0%.

Due to manufacturing tolerances, fluctuations of ±10% in the individual PARW values are common. Variations outside the normal ±10% spread can be an indicator of a different nominal wall thickness (NWT) or point towards a problem like aggregate pitting or general wall loss. Largely, the inspected pipes exhibited PARW values that are within the manufacturing tolerances. Pipe 0070 was found with the lowest PARW value of 91%, which is still within the aforementioned tolerance.

Local Wall Thickness

The RFT analysis found the inspected steel section to be in *excellent* condition, as there were no localized wall loss detected above the Threshold-of-Detection for the tool.

Table 3, in the subsequent pages, details the average remaining wall values for the inspected section. The same results are shown graphically on page 3.

Data Quality

The overall quality of the data collected for the 16in Goulet-Doucet line was satisfactory. There were instances of high-frequency noise that occurred throughout the run. The noise was likely caused by the increased tuberculation levels in the line, preventing the tool to travel smoothly during the inspection. In areas where there was significant noise, it is possible that the noise may have masked shallow and small-medium defects.

Table 3: Pipe List and Wall Thickness Readings – 400mm (16in) Doucet-Goulet Watermain Seine River Crossing

Pipe No.	Pipe Location Joint Connection Type			Tavg RW (%)	Local Wall Thickness <i>Grey-highlighted rows are asbestos cement pipes, which were not analyzed.</i>									Comments	
	Start (m)	End (m)	Length (m)		Tmin1			Tmin2			Tmin3				
					RW (%)	Location (m)	Clock Position	RW (%)	Location (m)	Clock Position	RW (%)	Location (m)	Clock Position		
A/C	0.00	5.76	5.75	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	Zero datum point is at the west end flange of the newly installed 400mm FLxFL DI WYE at Doucet St; Asbestos cement pipe.
F	5.76	6.38	0.62	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	16° Elbow (Metallic)
A/C	6.38	16.15	9.77	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	End of asbestos cement section
0010	16.15 Weld	17.58 Flange	1.42	N/A											Start of steel section
0020	17.58 Flange	22.80 Weld	5.22	97%											
0030	22.80 Weld	29.78 Flange	6.98	98%											
0040	29.78 Flange	36.09 Weld	6.31	107%											
0050	36.09 Weld	42.06 Flange	5.97	104%											
0060	42.06 Flange	48.58 Weld	6.52	102%											
0070	48.58 Weld	54.34 Flange	5.76	91%											
0080	54.34 Flange	60.15 Weld	5.81	101%											
0090	60.15 Weld	66.62 Flange	6.47	98%											
0100	66.62 Flange	72.01 Weld	5.39	100%											12° vertical deflection at 66.62m.
0110	72.01 Weld	79.04 Flange	7.03	100%											
0120	79.04 Flange	84.33 Weld	5.29	103%											
0130	84.33 Weld	91.33 Flange	7.00	106%											
0140	91.33 Flange	98.45 Weld	7.12	104%											

Table 3: Pipe List and Wall Thickness Readings – 400mm (16in) Doucet-Goulet Watermain Seine River Crossing

Pipe No.	Pipe Location Joint Connection Type			Tavg RW (%)	Local Wall Thickness <i>Grey-highlighted rows are asbestos cement pipes, which were not analyzed.</i>									Comments	
	Start (m)	End (m)	Length (m)		Tmin1			Tmin2			Tmin3				
					RW (%)	Location (m)	Clock Position	RW (%)	Location (m)	Clock Position	RW (%)	Location (m)	Clock Position		
0150	98.45 Weld	103.38 Flange	4.93	99%											End of steel section
A/C	103.38 (Flange)	109.85	6.47	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	Start of asbestos cement section
F	109.85	110.40	0.55	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	Metallic Piece
A/C	110.40	111.09	0.70	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	
F	111.09	111.60	0.51	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	Metallic Piece
A/C	111.60	111.99	0.38	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	
F	111.99	112.89	0.90	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	22° Elbow (metallic)
A/C	112.89	173.83	60.94	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	End of run is near the newly installed 400mm FLxFL DI WYE at Goulet St.

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Scope of Services

The agreement of PICA Corp to perform services extends only to those services provided for in writing. Under no circumstances shall such services extend beyond the performance of the requested services. It is expressly understood that all descriptions, comments and expressions of opinion reflect the opinions or observations of PICA Corp based on information and assumptions supplied by the owner/operator and are not intended nor can they be construed as representations or warranties. PICA Corp is not assuming any responsibilities of the owner/operator and the owner/operator retains complete responsibility for the engineering, manufacture, repair and use decisions as a result of the data or other information provided by PICA Corp. Nothing contained in this Agreement shall create a contractual relationship with or cause of action in favor of a third party against either the Line Owner or PICA Corp. In no event shall PICA Corp's liability in respect of the services referred to herein exceed the amount paid for such services.

Standard of Care

In performing the services provided, PICA Corp uses the degree, care, and skill ordinarily exercised under similar circumstances by others performing such services in the same or similar locality. No other warranty, expressed or implied, is made or intended by PICA Corp.

City of Winnipeg 2015: 450mm (18-in) St. James Watermain Assiniboine River Crossing

Condition Assessment Report, Standard Analysis



PICA – Pipeline Inspection & Condition Analysis Corporation
(A Subsidiary of Russell NDT Holdings Ltd.)

RFT ILI Tool

450mm (18-in) Cast Iron

Assiniboine River Crossing

Between Wolseley Avenue and Wellington Crescent

Winnipeg, MB

PICA Project: Winnipeg 8007

Inspection Date:	October 21-23, 2015
Report Submission:	January 12, 2016
Operators:	P. Ryhanen, G. Bouchard, B. Senka, G. Hubbard, D. Barron, D. Burton
Analyst:	J. Kim
Reviewers:	J. Regala
Report Revision:	1.0

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City of Winnipeg: 450mm (18in) St. James Watermain River Crossing

Condition Assessment Report, Standard Analysis

Executive Summary

PICA, under contract with AECOM, inspected a number of Water and Wastewater River Crossings for the City of Winnipeg using Remote Field Testing (RFT) Technology between October 19 and 24, 2015. This report documents the results of the RFT inspection of the 450mm (18-in) cast iron St. James Watermain, which crosses the Assiniboine River. The inspected portion spans between two newly installed access wyes at the St. James St/Wolseley Ave intersection and at Wellington Cr (refer to the line map on page 4). The inspection was conducted over the span of three days between October 21 and 23. Technical problems with the RFT tool were encountered during the first two days, and traced to a glitch in the exciter drive unit. The final inspection was performed on October 23 after repairs and functional testing. The results reported herein document PICA’s findings.

As noted in the supplied pipeline drawings, this watermain is comprised of a small section (~7m) of asbestos cement (AC) as well as cast iron (CI) pipe. The AC pipes are non-ferrous in nature, and are therefore excluded from this report.

In general, the RFT analysis found the inspected section to be in “fair to good” condition, with 76% of all pipes measuring at least 50% remaining wall (RW). A total of 165 localized wall loss indications were detected. Of these 165 defects, a total of 152 were measured to be either *shallow* (≥65% RW) or *medium* (>40% RW) in depth. The remaining thirteen defects were classified as *deep* indications with 20%-39% RW. There were no defects that measured less than 20% RW in this main.

Table 1 provides an overview of the RFT findings on 18in St. James Watermain River Crossing.

Table 1: Feature Indication Summary	
Inspected Length:	201.07m* <small>*includes ~7m of asbestos cement pipe</small>
Number of CI Pipe Sections:	59
Number of Analyzed CI Pipe Sections:	55
Average Wall Thickness:	93.7%
Thinnest circumferential pipe wall (Tcircmin):	78.8% (in Pipe 0390)
Number of CI pipes without localized wall loss indications:	19
Number of CI pipes with localized wall loss indications:	36
• Number of CI pipes in ‘good’ condition (>75% RW):	20
• Number of CI pipes in ‘fair’ condition (50-74% RW):	18
• Number of CI pipes in ‘poor’ condition (<50% RW):	12
Total number of wall loss indications reported:	165
Number of Bell and Spigot Joints:	51
Number of Elbows:	3

Figures 1 and 2 illustrate the distribution of localized wall loss along the 450mm (18in) St. James Watermain Assiniboine River Crossing with respect to remaining wall and circumferential location respectively. Note that there may be some (partially) overlapping data points due to defect proximity.

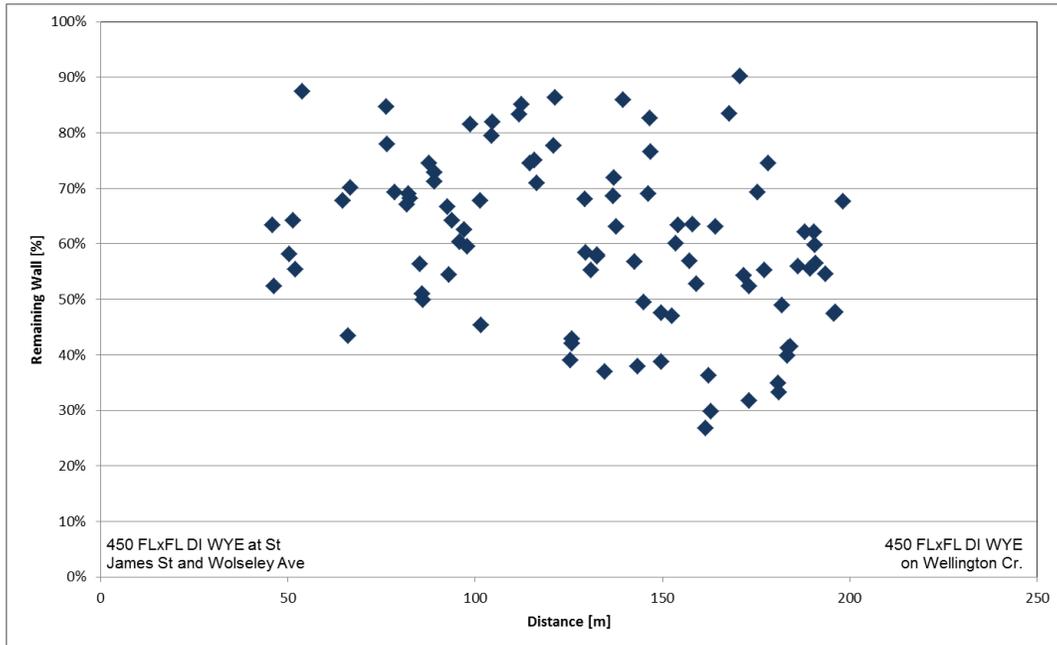


Figure 1: Distribution of wall loss with respect to remaining wall (%NWT) in pitting regions along the inspected section.

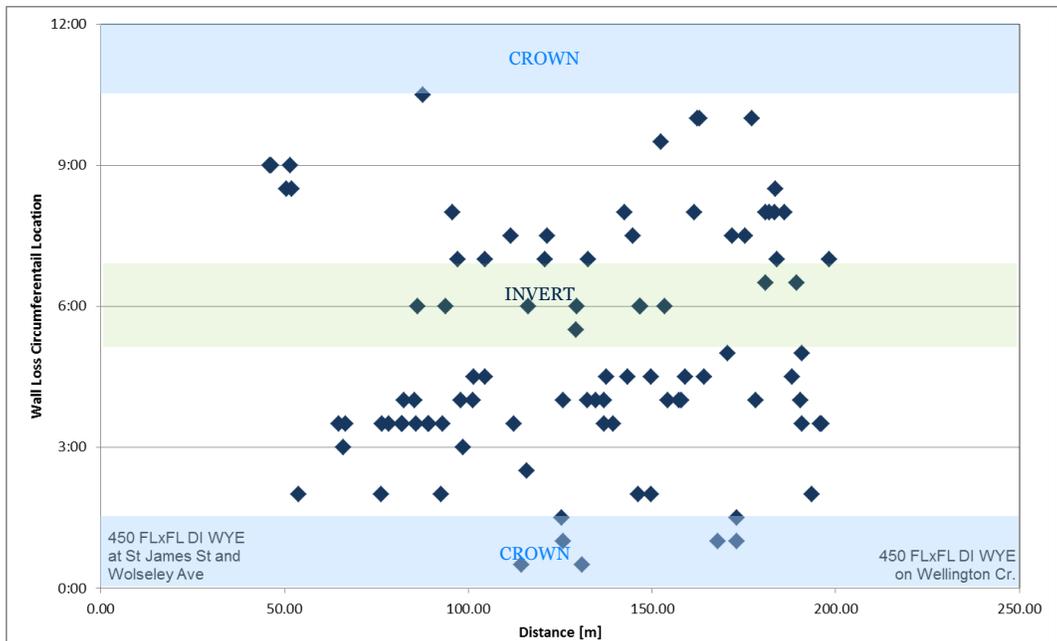


Figure 2: Circumferential distribution of pitting regions along the inspected section.

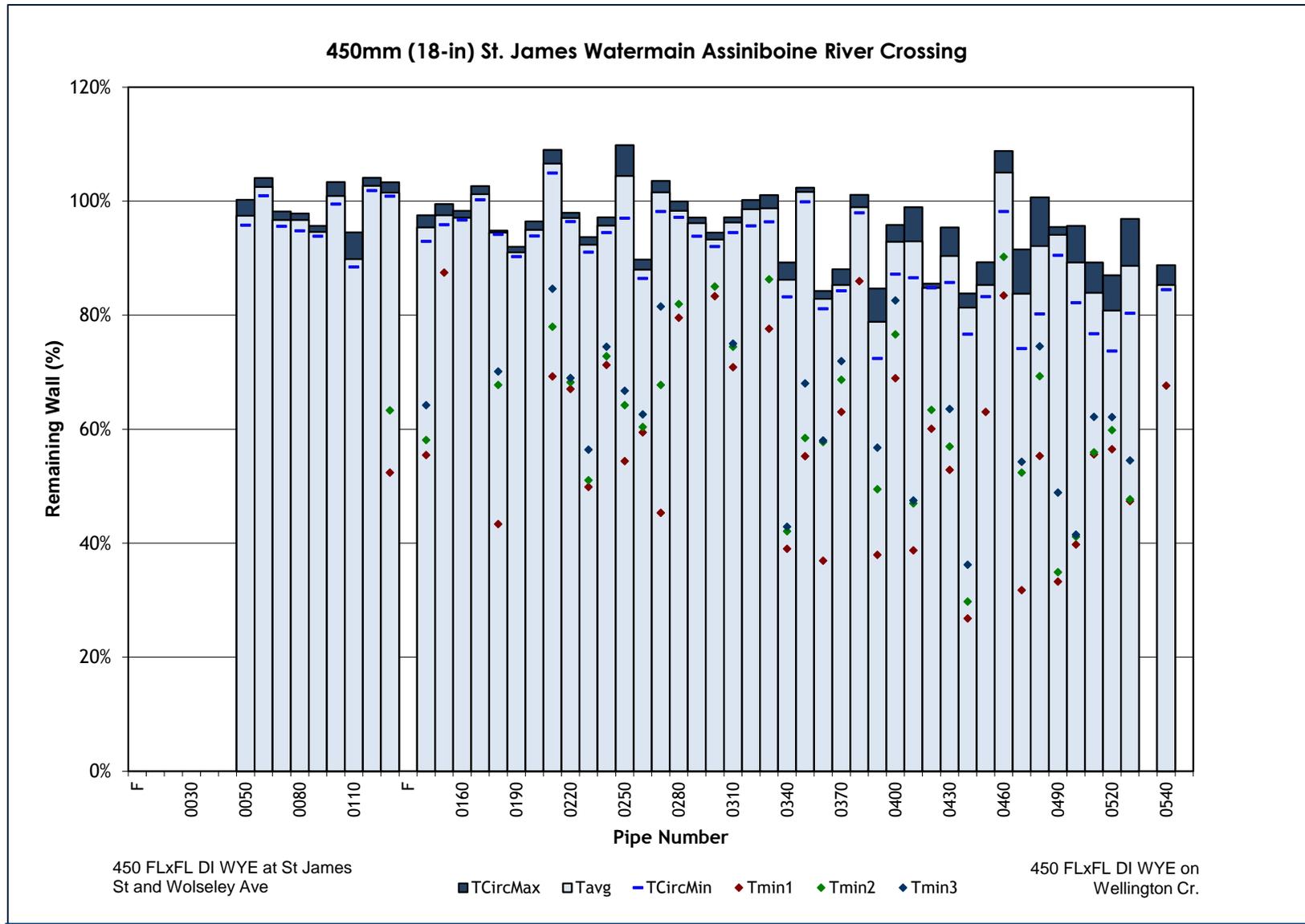


Figure 3: Structural Condition Assessment Summary – 450mm (18-in) St. James Watermain Assiniboine River Crossing

Table 2: Inspection Overview

Client:	City of Winnipeg	Location:	Winnipeg, MB
Line Name:	St. James Watermain River Crossing	Pipe Diameter:	450mm (18in)
Material:	Asbestos Cement (6.80m) Cast Iron (194.27m)	Year Installed:	1946
Nominal Wall Thickness (NWT):	25mm (1-inch)	Break History:	None provided
Inspection Dates:	October 21 - 23, 2015		
Inspected Length:	201.07m		
Lead Technician:	P. Ryhanen	Technician(s):	G. Bouchard, B. Senka, G Hubbard, D. Barron, D. Burton
Launch Access:	450mm FLxFL DI WYE at the intersection of St James St and Wolseley Ave		
Retrieve Access:	450mm FLxFL DI WYE near Wellington Cr		

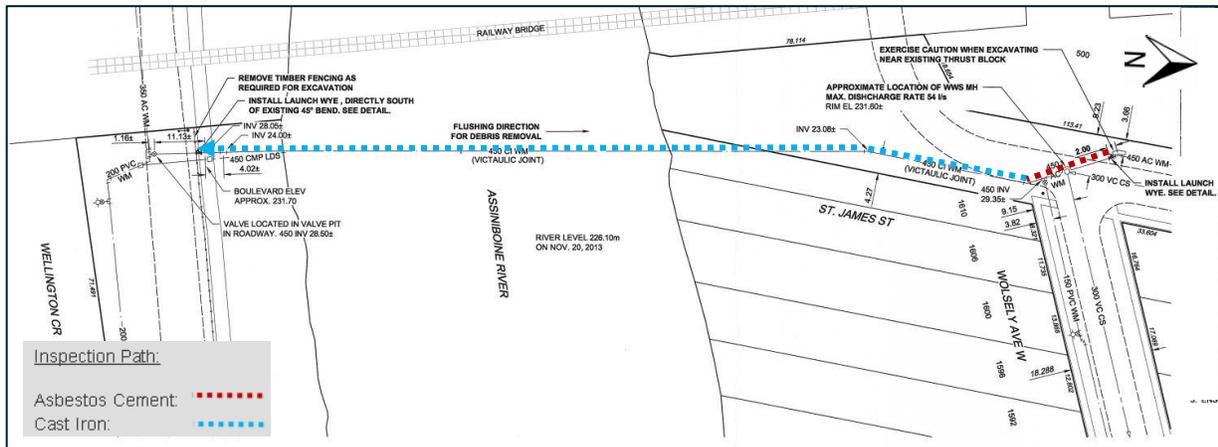
Operational Comments:

October 21 & 22, 2015

Two RFT runs were performed on these dates. Data collected resulted in noisy and unacceptable data quality. RFT tool required servicing after the October 22 run.

October 23, 2015

- 9:00am: Arrived at Tri-Cor to prepare the RFT tool and ensure repair was successful.
- 11:30am: Arrived on site and performed final tool test prior to launching.
- 2:15pm: Site set up for run.
- 2:50pm: Tool was launched from Wolseley Ave and St. James St
- 4:45pm: Tool arrived at the retrieve end.
- 4:50pm: Retrieve run starts. Inspection speed at 2m/min.
- 6:45pm: Tool arrived back at the launch wye.



Calibration

AECOM shipped two short sections of cast iron pipe to PICA in Edmonton. These sections were removed from the line when the access wyes were installed. Normally these sections are too short for calibration purposes, but PICA was able to extend the sections to make them partially suitable for calibration (please see figure 4 below).

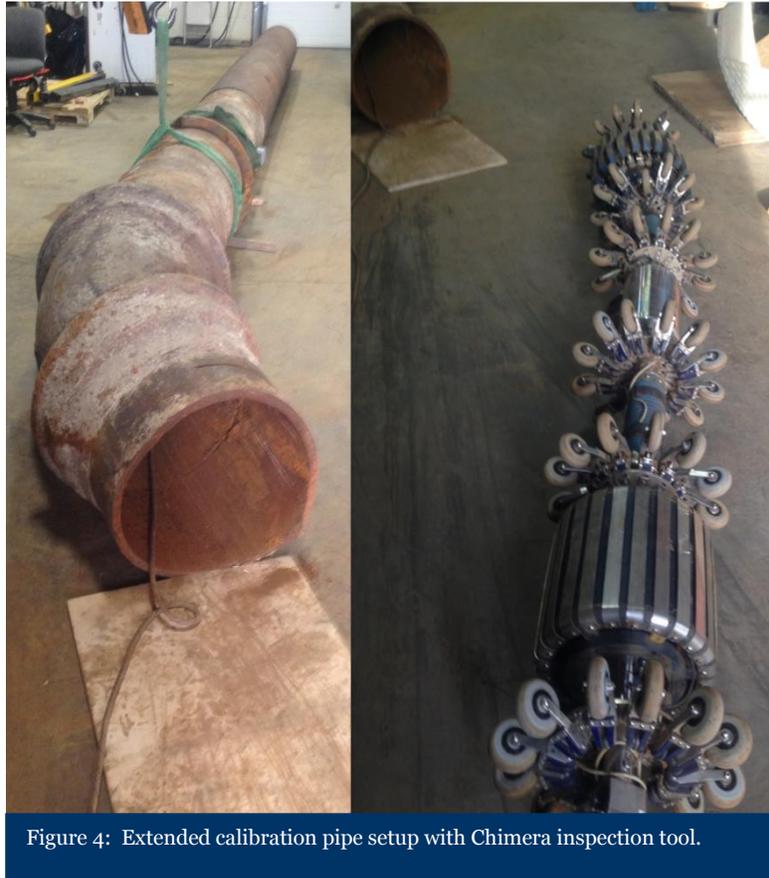


Figure 4: Extended calibration pipe setup with Chimera inspection tool.

The pipe sections appeared to be in good condition: no significant pitting was visible other than minor outer surface corrosion. There was internal tuberculation build-up, which PICA removed prior to running the Chimera tool. Due to the short usable length of the two pipe sections and the difficulty of machining cast iron pipe, a single calibration through hole was machined into the pipe, with dimensions as outlined in Table 3 below. PICA initially attempted to drill a round hole, but this proved difficult and dangerous in the thick cast iron pipe. Instead a rectangular defect was cut into the line. The RFT response to this defect is shown in Figure 5 below.

Table 3: Calibration Defect cut into the 18-inch St James Calibration Pipe

RFT Calibration Through Hole	33 (length) x38mm (circ) (1.3x1.5inch)	100%
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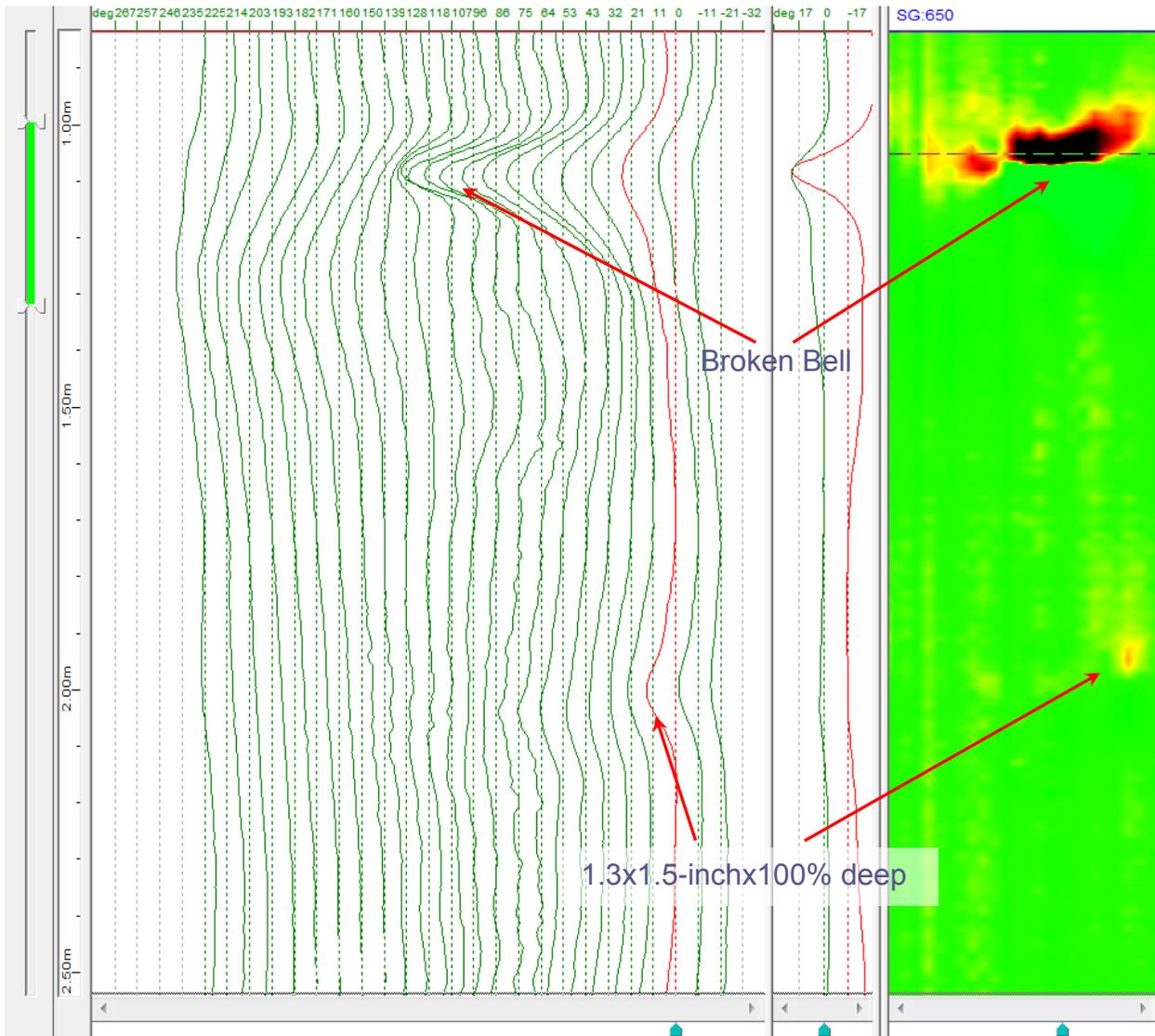


Figure 5: Chimera RFT Defect response to a rectangular through hole cut into the 18-inch St James calibration pipe.

Analysis Results

Location Reporting, Pipe Lengths & Features

The total logged distance for the 18in St. James Watermain Assiniboine River Crossing was 201.07m. This distance includes approximately 6.80m of asbestos cement piping that exists at the north end of the main immediately after the launch site. Asbestos cement piping is non-ferrous in nature and therefore was excluded during the analysis.

The zero datum point was set at the southern flange of the newly installed 450 FLxFL DI WYE at the St James St and Wolseley Ave intersection. The inspection started on the north side of the Assiniboine River and ended on the south side near Wellington Crescent.

The average length of a common and standard pipe is 3.73m (12.24ft) with shorter pieces adjacent to features (ie deflections). A total of eight line features (3 bends and 5 shallow angle deflections) were identified in the RFT data. With the exception of the deflections, the other three bends were confirmed using pipeline documents provided by the client.

General Wall Thickness

Pipe sections longer than 2.3m were analysed to obtain the average remaining wall thickness (PARW) calculated over the length of the section. The measured average wall thickness for the inspected portion of the 18in St. James Watermain River Crossing is 93.7%.

Due to manufacturing tolerances, fluctuations of $\pm 15\%$ in the individual PARW values are common. Variations outside the normal $\pm 15\%$ spread can be an indicator of a different nominal wall thickness (NWT) or point towards a problem like aggregate pitting or general wall loss. Largely, the inspected pipes exhibited PARW values that are within the manufacturing tolerances.

There were two distinct NWT types that were observed in the data. The first NWT Type is found predominantly in the northern 121m, while the second NWT Type is most prevalent in the southern portion. Table 4, on pages 8-11, lists the NWT transition points along the inspected section.

Local Wall Thickness

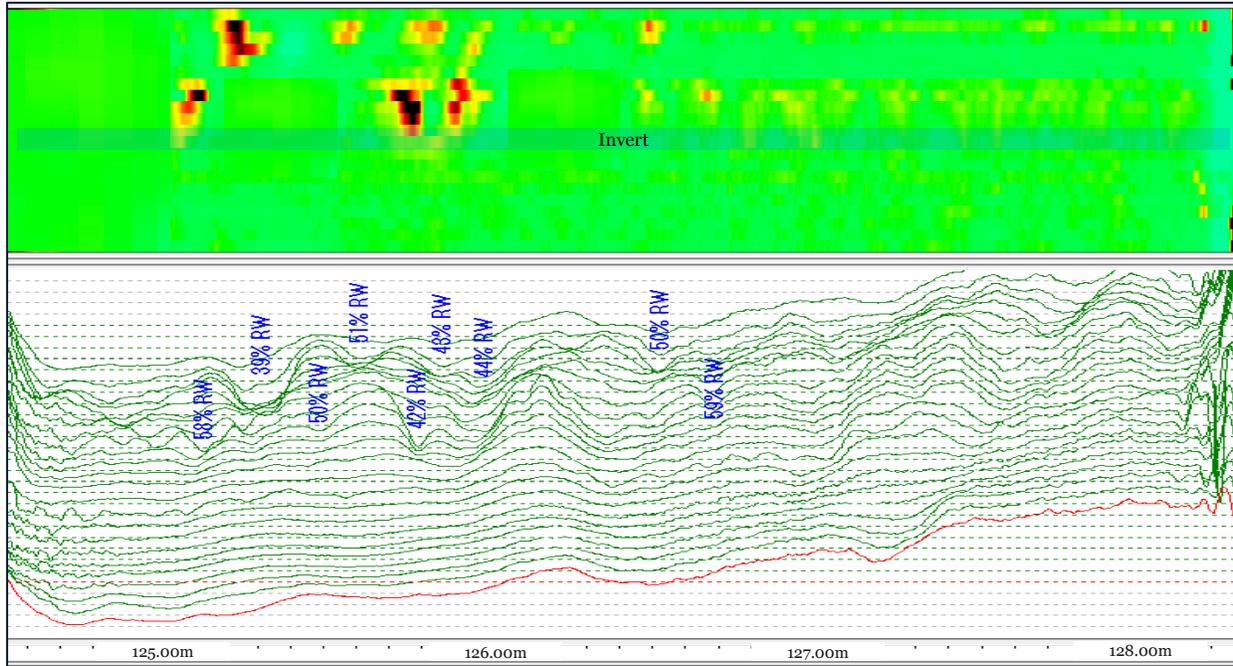
In general, the RFT analysis found the inspected section to be in “*fair to good*” condition, with 76% of all pipes measuring at least 50% remaining wall (RW). A total of 165 localized wall loss indications were detected. Of these 165 defects, a total of 152 defects were measured to be either *shallow* ($\geq 65\%$ RW) or *medium* ($> 40\%$ RW) in depth. The remaining 13 were classified as *deep* indications with 20%-39% RW. There were no defects that measured less than 20% RW in this main.

Table 4, on pages 8-11, details the three worst pitting indications per pipe (T_{min1} , T_{min2} and T_{min3}), as well as the average, minimum and maximum remaining wall values for the inspected section. The same results are shown graphically on page 3.

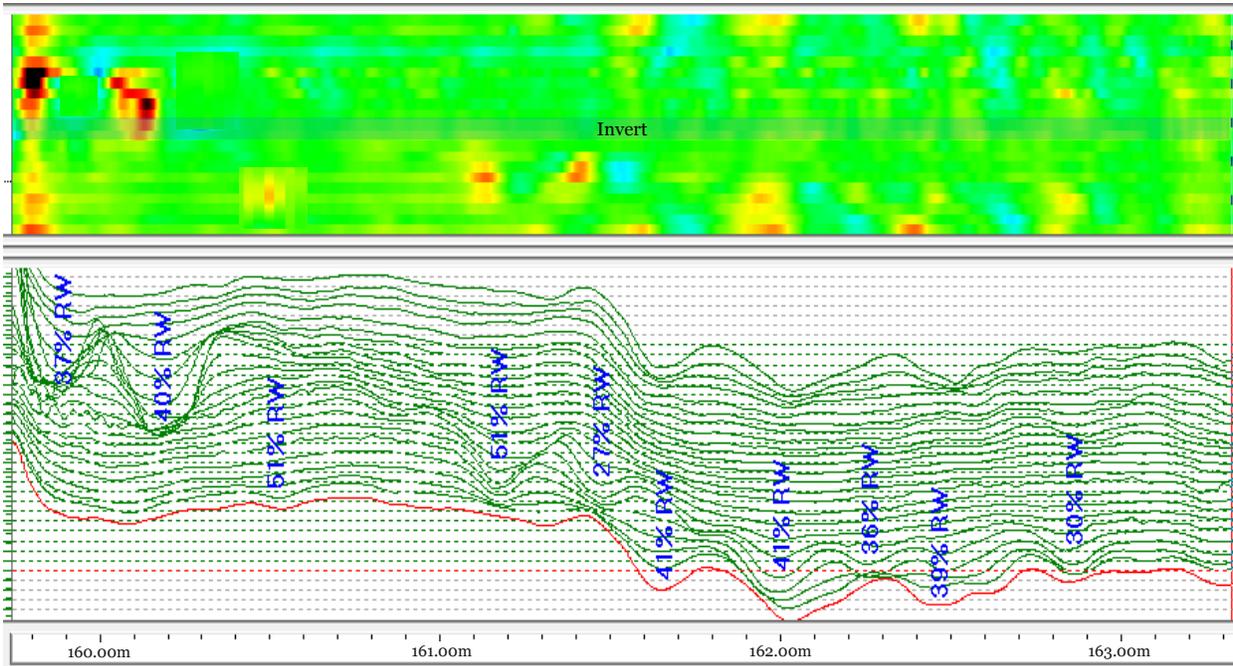
RFT Color Maps - Pipes 0340, 0440, 0490 and 0500

Color maps for four pipe sections with significant corrosion are presented in the next two pages. It is important to note that these color maps are for visual purposes only and should not be used to estimate the remaining wall.

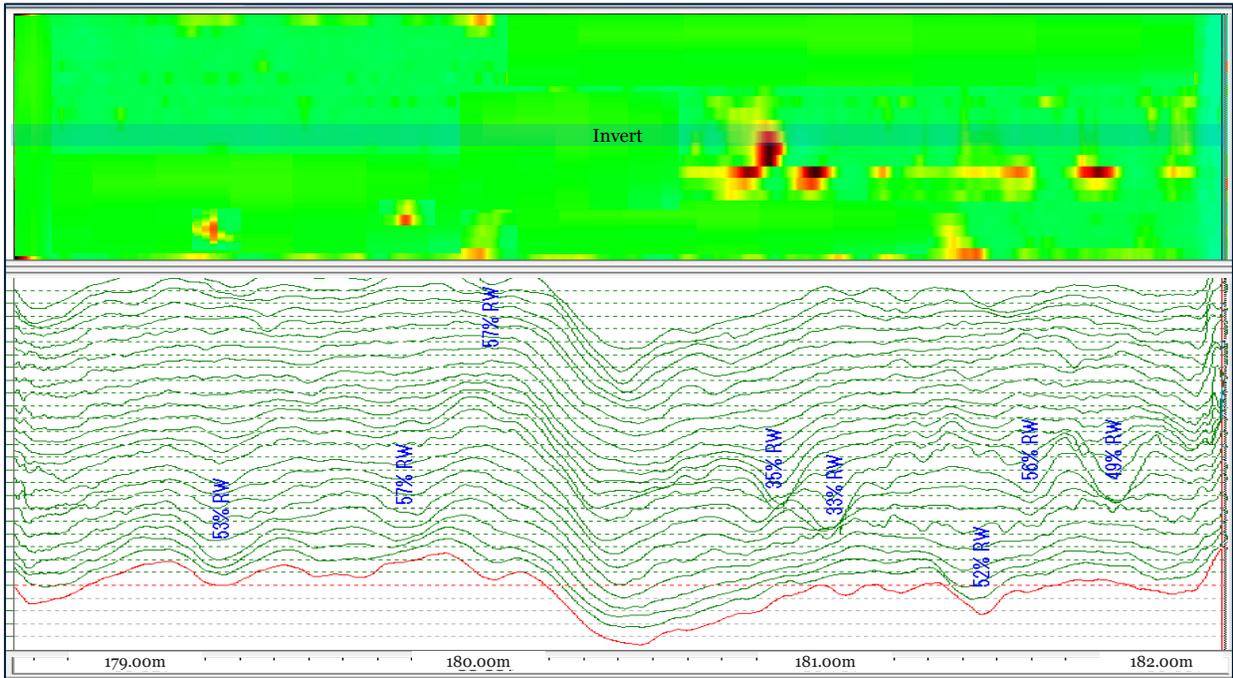
Pipe 0340: 124.43 - 128.25m (9 defects ranging between 39% and 59% RW)



Pipe 0440: 159.68 - 163.44m (10 defects ranging between 27% and 51% RW)



Pipe 0490: 178.53 – 182.16m (6 defects ranging between 40% and 66% RW)



Pipe 0500: 182.16 – 185.84m (6 defects ranging between 40% and 66% RW)

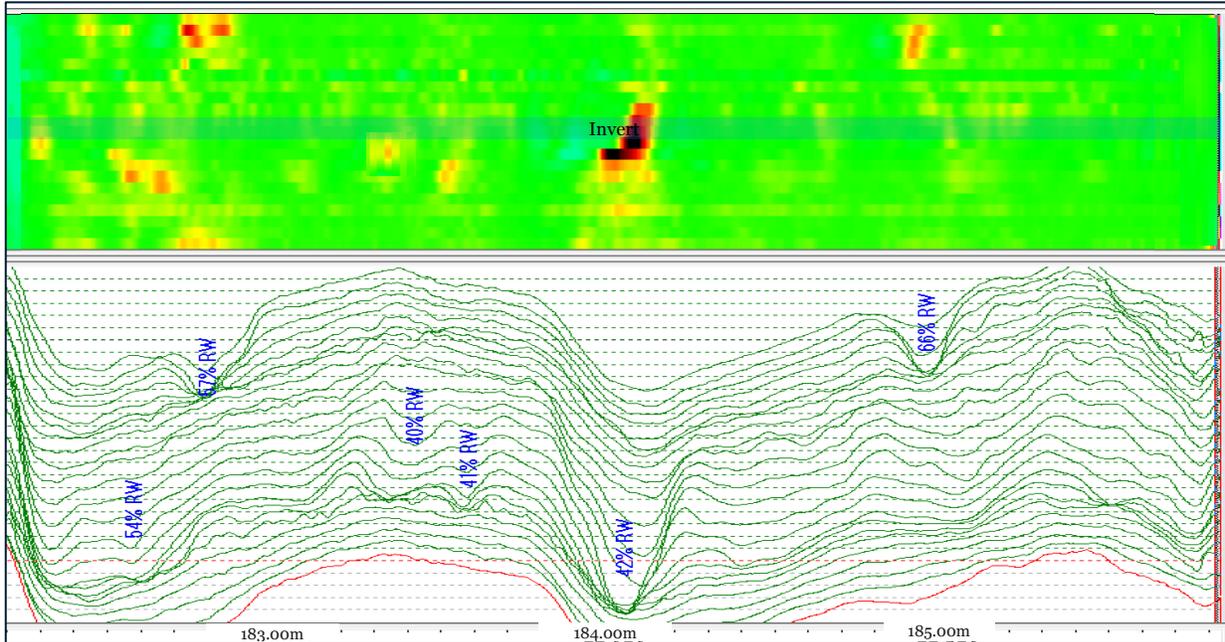


Table 4: Pipe List and Wall Thickness Readings – 450mm (18in) St. James Watermain Assiniboine River Crossing

Pipe No.	Pipe Location			Tavg RW (%)	Circumferential Wall Thickness		Local Wall Thickness <i>Clock positions are with a North to South perspective of (eg. 3:00=West, 9:00=East).</i>									Comments	
	Start (m)	End (m)	Length (m)		Teircmax RW (%)	Teircmin RW (%)	Tmin1			Tmin2			Tmin3				
							RW (%)	Location (m)	Clock Position	RW (%)	Location (m)	Clock Position	RW (%)	Location (m)	Clock Position		
F	0.00	6.80	6.80	N/A	N/A	N/A											Zero datum point is at the south end flange of the newly installed 450mm FLxFL DI WYE at St James St and Wolseley Ave. The first 6.80m is comprised of asbestos cement pipe.
0010	6.80	9.23	2.43	N/A	N/A	N/A											Start of Cast Iron Pipe
0020	9.23	11.14	1.92	N/A	N/A	N/A											
0030	11.14	12.64	1.50	N/A	N/A	N/A											
0040	12.64	14.66	2.02	N/A	N/A	N/A											
F	14.66	15.37	0.71	N/A	N/A	N/A											~29° deflection
0050	15.37	19.14	3.77	97%	100%	96%											NWT Type 1
0060	19.14	22.82	3.68	102%	104%	101%											NWT Type 1
0070	22.82	26.54	3.72	97%	98%	96%											NWT Type 1
0080	26.54	29.86	3.32	97%	98%	95%											NWT Type 1
0090	29.86	33.63	3.77	95%	96%	94%											NWT Type 1; 3° vertical deflection
0100	33.63	37.37	3.74	101%	103%	100%											NWT Type 1; 4° vertical deflection
0110	37.37	41.15	3.78	90%	95%	88%											NWT Type 1; 5° vertical deflection
0120	41.15	44.89	3.74	103%	104%	102%											NWT Type 1; 4° vertical deflection
0130	44.89	48.65	3.75	101%	103%	101%	52%	46.26	9:00	63%	45.89	9:00					NWT Type 1
F	48.65	49.51	0.86	N/A	N/A	N/A											~23° deflection
0140	49.51	53.21	3.70	95%	98%	93%	55%	51.99	8:30	58%	50.44	8:30	64%	51.43	9:00		NWT Type 1
0150	53.21	56.93	3.73	98%	99%	96%	87%	53.86	2:00								NWT Type 1
0160	56.93	60.66	3.73	97%	98%	97%											NWT Type 1
0170	60.66	64.44	3.77	101%	103%	100%											NWT Type 1

Table 4: Pipe List and Wall Thickness Readings – 450mm (18in) St. James Watermain Assiniboine River Crossing

Pipe No.	Pipe Location			Tavg RW (%)	Circumferential Wall Thickness		Local Wall Thickness <i>Clock positions are with a North to South perspective of (eg. 3:00=West, 9:00=East).</i>									Comments
	Start (m)	End (m)	Length (m)		Teircmax RW (%)	Teircmin RW (%)	Tmin1			Tmin2			Tmin3			
							RW (%)	Location (m)	Clock Position	RW (%)	Location (m)	Clock Position	RW (%)	Location (m)	Clock Position	
0180	64.44	67.87	3.44	94%	95%	94%	43%	66.06	3:00	68%	64.69	3:30	70%	66.59	3:30	NWT Type 1; This contains two additional defects with 72% and 80% RW.
0190	67.87	71.66	3.78	91%	92%	90%										NWT Type 1
0200	71.66	75.44	3.78	95%	96%	94%										NWT Type 1
0210	75.44	79.24	3.80	107%	109%	105%	69%	78.45	3:30	78%	76.51	3:30	85%	76.29	2:00	NWT Type 1
0220	79.24	83.01	3.77	97%	98%	96%	67%	81.78	3:30	68%	82.54	4:00	69%	82.12	3:30	NWT Type 1
0230	83.01	86.80	3.79	92%	94%	91%	50%	86.12	6:00	51%	85.87	3:30	56%	85.29	4:00	NWT Type 1; This contains two additional defects with 64% and 69% RW.
0240	86.80	90.57	3.77	96%	97%	94%	71%	89.19	3:30	73%	89.06	3:30	74%	87.71	10:30	NWT Type 1; This contains an additional defect with 85% RW.
0250	90.57	94.44	3.87	104%	110%	97%	54%	92.92	3:30	64%	93.81	6:00	67%	92.65	2:00	NWT Type 2; This contains 3 additional defects between 68% and 83% RW.
0260	94.44	98.11	3.67	88%	90%	86%	59%	97.85	4:00	60%	95.77	8:00	63%	97.07	7:00	NWT Type 2; This contains 3 additional defects between 66% and 79% RW.
0270	98.11	101.83	3.72	102%	104%	98%	45%	101.56	4:30	68%	101.23	4:00	82%	98.58	3:00	NWT Type 2; 4° vertical deflection; This contains one additional defect with 84% RW.
0280	101.83	105.71	3.88	98%	100%	97%	80%	104.49	7:00	82%	104.64	4:30				NWT Type 1
0290	105.71	109.48	3.77	96%	97%	94%										NWT Type 1
0300	109.48	113.26	3.78	93%	94%	92%	83%	111.66	7:30	85%	112.42	3:30				NWT Type 1
0310	113.26	116.92	3.67	96%	97%	94%	71%	116.34	6:00	74%	114.56	0:30	75%	115.89	2:30	NWT Type 1; This contains two additional defects with 75% and 77% RW.
0320	116.92	120.67	3.75	99%	100%	96%										NWT Type 1
0330	120.67	124.43	3.76	99%	101%	96%	78%	120.93	7:00	86%	121.41	7:30				NWT Type 1
0340	124.43	128.25	3.82	86%	89%	83%	39%	125.32	1:30	42%	125.78	4:00	43%	125.85	1:00	NWT Type 2; This contains 6 additional defects between 44% and 59% RW.
0350	128.25	132.05	3.80	102%	102%	100%	55%	131.00	0:30	58%	129.48	6:00	68%	129.22	5:30	NWT Type 2; This contains 6 additional defects between 71% and 77% RW.

Table 4: Pipe List and Wall Thickness Readings – 450mm (18in) St. James Watermain Assiniboine River Crossing

Pipe No.	Pipe Location			Tavg RW (%)	Circumferential Wall Thickness		Local Wall Thickness <i>Clock positions are with a North to South perspective of (eg. 3:00=West, 9:00=East).</i>									Comments
	Start (m)	End (m)	Length (m)		Teircmax RW (%)	Teircmin RW (%)	Tmin1			Tmin2			Tmin3			
							RW (%)	Location (m)	Clock Position	RW (%)	Location (m)	Clock Position	RW (%)	Location (m)	Clock Position	
0360	132.05	135.64	3.59	83%	84%	81%	37%	134.64	4:00	58%	132.49	4:00	58%	132.58	7:00	NWT Type 2; This contains 4 additional defects between 59% and 75% RW.
0370	135.64	138.58	2.94	85%	88%	84%	63%	137.65	4:30	69%	136.88	3:30	72%	137.02	4:00	NWT Type 1
0380	138.58	141.87	3.30	99%	101%	98%	86%	139.39	3:30							NWT Type 1
0390	141.87	145.62	3.75	79%	85%	72%	38%	143.29	4:30	49%	144.90	7:30	57%	142.53	8:00	NWT Type 2; This contains 4 additional defects between 61% and 73% RW.
0400	145.62	149.41	3.79	93%	96%	87%	69%	146.16	2:00	77%	146.90	6:00	83%	146.69	6:00	NWT Type 2
0410	149.41	153.19	3.78	93%	99%	87%	39%	149.74	2:00	47%	152.51	9:30	47%	149.73	4:30	NWT Type 1; This contains 3 additional defects between 59% and 84% RW.
0420	153.19	155.93	2.73	85%	86%	85%	60%	153.51	6:00	63%	154.19	4:00				NWT Type 2
0430	155.93	159.68	3.75	90%	95%	86%	53%	159.04	4:30	57%	157.29	4:00	64%	158.03	4:00	NWT Type 2; This contains 3 additional defects between 64% and 77% RW.
0440	159.68	163.44	3.76	81%	84%	77%	27%	161.46	8:00	30%	162.86	10:00	36%	162.25	10:00	NWT Type 2; This contains 7 additional defects between 37% and 51% RW.
0450	163.44	167.21	3.77	85%	89%	83%	63%	164.13	4:30							NWT Type 1
0460	167.21	170.98	3.77	105%	109%	98%	83%	167.90	1:00	90%	170.64	5:00				NWT Type 1
0470	170.98	174.78	3.80	84%	92%	74%	32%	173.02	1:30	52%	173.15	1:00	54%	171.77	7:30	NWT Type 2; This contains 7 additional defects between 55% and 71% RW.
0480	174.78	178.53	3.75	92%	101%	80%	55%	177.27	10:00	69%	175.29	7:30	75%	178.18	4:00	NWT Type 2
0490	178.53	182.16	3.63	94%	95%	90%	33%	181.00	8:00	35%	180.87	6:30	49%	181.85	8:00	NWT Type 2; This contains 5 additional defects between 52% and 57% RW.
0500	182.16	185.84	3.68	89%	96%	82%	40%	183.35	8:00	41%	183.56	8:30	42%	184.05	7:00	NWT Type 2; This contains 3 additional defects between 54% and 66% RW.
0510	185.84	189.55	3.70	84%	89%	77%	56%	189.41	6:30	56%	186.14	8:00	62%	188.08	4:30	NWT Type 2; This contains 6 additional defects between 65% and 78% RW.
0520	189.55	193.23	3.69	81%	87%	74%	56%	190.89	3:30	60%	190.73	5:00	62%	190.38	4:00	NWT Type 2; This contains an additional defect with 64% RW.

Table 4: Pipe List and Wall Thickness Readings – 450mm (18in) St. James Watermain Assiniboine River Crossing

Pipe No.	Pipe Location			Tavg RW (%)	Circumferential Wall Thickness		Local Wall Thickness <i>Clock positions are with a North to South perspective of (eg. 3:00=West, 9:00=East).</i>									Comments
	Start (m)	End (m)	Length (m)		Teircmax RW (%)	Teircmin RW (%)	Tmin1			Tmin2			Tmin3			
							RW (%)	Location (m)	Clock Position	RW (%)	Location (m)	Clock Position	RW (%)	Location (m)	Clock Position	
0530	193.23	197.02	3.78	89%	97%	80%	47%	195.82	3:30	48%	196.17	3:30	55%	193.50	2:00	NWT Type 2; This contains two additional defects with 61% and 63% RW.
F	197.02	197.86	0.85	N/A	N/A	N/A										45° Vertical Bend
0540	197.86	200.93	3.07	85%	89%	84%	68%	198.22	7:00							NWT Type 1
0550	200.93	201.07	0.13	N/A	N/A	N/A										End of run is near the newly installed 450mm FLxFL DI WYE on Wellington Cr.

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Appendix E
Capital Program

Water Crossings - Capital Program

Site #	Crossing	Nominal Diameter (mm)	2012 Budget Replacement Cost	Estimated Replacement Cost (2016)	Proposed Work	5 year Capital Program	10 Year Capital Program	Beyond 10 Year Capital Program	Reinspection (20 Year Frequency)
9	Assiniboia Feedermain	600	\$1,850,000.00	\$6,000,000.00	Geotechnical (Toe Armoring)	\$25,000.00			
					Rehabilitation (N/A)				
					Inspection (Advanced EM ILI)	\$405,000.00			
10	Goulet-Doucet Watermain	400	\$600,000.00	\$1,500,000.00	Geotechnical (Toe Armoring)	\$25,000.00			
					Rehabilitation (Internal Lining)	\$137,500.00			
					Reinspection (Advanced EM ILI)				\$210,000.00
11	Kildonan-Redwood Feedermain	600	\$2,500,000.00	\$5,800,000.00	Geotechnical (Slope Regrading and Armoring)	\$62,500.00			
					Rehabilitation (N/A)				
					Inspection (Advanced EM ILI)	\$400,000.00			
12	Maryland Bridge Watermain	250	\$675,000.00	\$1,200,000.00	Geotechnical (Slope Regrading and Armoring)	\$125,000.00			
					Rehabilitation TDB (Replacement Cost Presented)	\$1,200,000.00			
					Reinspection (N/A)				
14	St. James Street Watermain	450	\$1,387,500.00	\$3,600,000.00	Geotechnical (Slope Stabilization and Armoring)	\$837,500.00			
					Rehabilitation (N/A)				
					Reinspection (Advanced EM ILI)	\$250,000.00			\$250,000.00
Total:			\$7,012,500.00	\$18,100,000.00		\$3,467,500.00	\$0.00	\$0.00	\$460,000.00