AECOM

The City of Winnipeg

High Risk River Crossings – Phase Two Condition Assessment Report

Prepared by:

AECOM
99 Commerce Drive
Winnipeg, MB, Canada R3P 0Y7
www.aecom.com

204 477 5381 tel
204 284 2040 fax

Project Number: 60549028

April 2020

Statement of Qualifications and Limitations

The attached Report (the “Report”) has been prepared by AECOM Canada Ltd. (“AECOM”) for the benefit of the Client (“Client”) in accordance with the agreement between AECOM and Client, including the scope of work detailed therein (the “Agreement”).

The information, data, recommendations and conclusions contained in the Report (collectively, the “Information”):

- is subject to the scope, schedule, and other constraints and limitations in the Agreement and the qualifications contained in the Report (the “Limitations”);
- represents AECOM’s professional judgement in light of the Limitations and industry standards for the preparation of similar reports;
- may be based on information provided to AECOM which has not been independently verified;
- has not been updated since the date of issuance of the Report and its accuracy is limited to the time period and circumstances in which it was collected, processed, made or issued;
- must be read as a whole and sections thereof should not be read out of such context;
- was prepared for the specific purposes described in the Report and the Agreement; and
- in the case of subsurface, environmental or geotechnical conditions, may be based on limited testing and on the assumption that such conditions are uniform and not variable either geographically or over time.

AECOM shall be entitled to rely upon the accuracy and completeness of information that was provided to it and has no obligation to update such information. AECOM accepts no responsibility for any events or circumstances that may have occurred since the date on which the Report was prepared and, in the case of subsurface, environmental or geotechnical conditions, is not responsible for any variability in such conditions, geographically or over time.

AECOM agrees that the Report represents its professional judgement as described above and that the Information has been prepared for the specific purpose and use described in the Report and the Agreement, but AECOM makes no other representations, or any guarantees or warranties whatsoever, whether express or implied, with respect to the Report, the Information or any part thereof.

Without in any way limiting the generality of the foregoing, any estimates or opinions regarding probable construction costs or construction schedule provided by AECOM represent AECOM’s professional judgement in light of its experience and the knowledge and information available to it at the time of preparation. Since AECOM has no control over market or economic conditions, prices for construction labour, equipment or materials or bidding procedures, AECOM, its directors, officers and employees are not able to, nor do they, make any representations, warranties or guarantees whatsoever, whether express or implied, with respect to such estimates or opinions, or their variance from actual construction costs or schedules, and accept no responsibility for any loss or damage arising therefrom or in any way related thereto. Persons relying on such estimates or opinions do so at their own risk.

Except (1) as agreed to in writing by AECOM and Client; (2) as required by-law; or (3) to the extent used by governmental reviewing agencies for the purpose of obtaining permits or approvals, the Report and the Information may be used and relied upon only by Client.

AECOM accepts no responsibility, and denies any liability whatsoever, to parties other than Client who may obtain access to the Report or the Information for any injury, loss or damage suffered by such parties arising from their use of, reliance upon, or decisions or actions based on the Report or any of the Information (“improper use of the Report”), except to the extent those parties have obtained the prior written consent of AECOM to use and rely upon the Report and the Information. Any injury, loss or damages arising from improper use of the Report shall be borne by the party making such use.

This Statement of Qualifications and Limitations is attached to and forms part of the Report and any use of the Report is subject to the terms hereof.

AECOM: 2015-04-13

AECOM Imagine it. Delivered.

AECOM
99 Commerce Drive 204 477 5381 tel
Winnipeg, MB, Canada R3P 0Y7 204 284 2040 fax
www.aecom.com

April 2, 2020

Mr. Armand Delaurier, C.E.T.
Project Coordinator
Asset Management Branch
The City of Winnipeg
Water and Waste Department
110-1199 Pacific Avenue
Winnipeg, MB R3E 3S8

Dear Mr. Delaurier:

Regarding: High Risk River Crossings – Phase Two Condition Assessment Report

We are pleased to submit the Condition Assessment Report for this project.

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

Sincerely,
AECOM Canada Ltd.

C.C. Macey, P. Eng.
North America Technical Practice Leader
Condition Assessment and Rehabilitation
/gms

cc: C. Wiebe, WWD

Distribution List

# of Hard Copies	PDF Required	Association / Company Name
6	X	Armand Delaurier, City of Winnipeg
	X	Cynthia Wiebe, City of Winnipeg

Revision Log

Revision #	Revised By	Date	Issue / Revision Description
0	A. Braun/M. McDonald/C. Macey	February 2020	Draft
1	A. Braun/M. McDonald/C. Macey	April 2020	Final

AECOM Signatures

Report Prepared By:

Adam Braun, P. Eng.
Municipal Engineer

Report Reviewed By:

Marvin McDonald, C.E.T.
Project Manager

Report Reviewed By:

C.C. Macey, P. Eng.
Americas Technical Practice Leader
Condition Assessment and Rehabilitation

PROVINCE OF MANITOBA
02 April 2020
C. C.
MACEY
REGISTERED ENGINEER

Table of Contents

List of Figures

List of Tables

Appendices

1. Executive Summary

The City of Winnipeg High Risk River Crossing (HRRC) Condition Assessment Program - Phase Two (Phase Two Program) was carried out from 2017 to 2020. The City's Water and Wastewater River Crossing Inventory, operated by the Water and Waste Department (WWD) includes approximately 70 potable water crossings and 41 wastewater sewer crossing locations with 56 separate wastewater pipelines. Initial screening and prioritization for assessment was developed in separate studies for the wastewater[1] and water[2] systems in 2006 and 2011. Phase One of the Program was completed in 2016[3,4] and included detailed assessment of 19 pipelines at 14 separate locations (13 wastewater and 6 potable water).

The river crossing inventory is comprised of very complex pipeline assets to assess, requiring the use of multiple technologies and techniques to ascertain condition in an ever-evolving environment of technological change. The program inherently has considerable operational risk to deploy assessment technologies at many of the sites. Many deployments require system modifications to accommodate inspection tools as well as considerable planning and the implementation of modified operational modes to implement the inspections.

In the Phase Two program, six pipelines were assessed using a variety of inspection technologies to ascertain condition with greater certainty, identify rehabilitation/replacement treatments where warranted and estimate remaining life span and reinspection frequency. As all of the assets cross rivers, the assessment program also included geotechnical investigations to assess slope stability of riverbanks to ascertain whether slope instabilities could engage the pipes over time.

The Phase Two Program included the following assets:

- Site 1 – Kildonan-Redwood Feeder Main Crossing (1955 Steel)
- Site 2 – Charleswood-Assiniboia Feeder Main Crossing (1965 Steel)
- Site 3 – St. Vital Bridge Force Main Crossing (Aerial, 1988 Steel)
- Site 4 – Newton Ave Force Main Crossing (1977 HDPE)
- Site 5 – Heritage Park Force Main Crossing (1989 PVC)
- Site 6 – Fort Garry-St. Vital (FGSV) Feeder Main Crossing (1959 Cast Iron)

The program commenced with a comprehensive planning phase to:

- Select the most appropriate technology or suite of technologies and inspection approach for each site.
- Review the operational constraints associated with implementation at each site, establish system modification requirements, conduct formal risk assessments, and establish operational requirement to facilitate program implementation.
- Develop formal procurement contracts to implement the program.

Geotechnical investigations at each site consisted of a balance of visual site inspections followed up by office stability assessments in instances that warranted more detailed assessment.

---

[1] *UMA-AECOM, "WWS River Crossing Assessment-City File S660", report for WWD, December 22, 2006*
[2] *AECOM, "Water Main Criticality Study – Volume 1 – Overall System Management", report for WWD, July 2011.*
[3] *AECOM, "High Risk River Crossings – Condition Assessment Report – Sewer Crossings", report for WWD, September 2016*
[4] *AECOM, "High Risk River Crossings – Condition Assessment Report – Water Crossings", report for WWD, November 2016*

The technical approach for inspection of each pipeline varied on a site-by-site basis as follows:

- Site 1 and 2 – Direct measurement using continuous internal electromagnetic (EM) inspection techniques to estimate pipe wall loss for steel crossing pipes. Assessment involved structural assessment to ascertain the ramifications of any observed defects.
- Site 3 – Direct measurement using spot external EM techniques to assess wall loss in steel pipe at select locations and then inference to assess overall condition. Assessment was intended to include structural assessment of any observed defects and inference of overall condition.
- Site 4 – The use of continuous SONAR inspection platforms, post-processed to ascertain pipeline geometry, debris and air accumulation as well as opportunistic sampling of material obtained in the Phase One program. Assessment approach involved a balance of structural assessment and material deterioration assessment based on the sampling results.
- Site 5 – Visual external inspection at a select location and sampling of the pipe material. Assessment approach involved a balance of structural assessment and material deterioration assessment based on the sampling results.
- Site 6 – Acoustic leak detection and visual assessment through underwater CCTV. Assessment approach used a balance of inference based on structural assessment, knowledge of the exterior pipe exposure environment and conformation of internal condition and overall hydrostatic integrity through leak detection.

In additional to the acoustic leak detection carried out at Site 6, all other sites were subjected to low pressure testing to assess whether any active leaks were present.

Specialized inspection and system modifications were procured in multiple packages as follows:

1. EM inspection services for Sites 1, 2 and 3 were procured under City RFP 495-2018 – Provision of Non-Destructive Inspection Services for Pipeline River Crossings. This work was awarded to PICA - Pipeline Inspection and Condition Assessment Corporation in September 2018. PICA proposed use of their Chimera Remote Field Technology (RFT) platform for Site 1 and 2, and their external EM bracelet probe for Site 3.
2. SONAR inspection Services for Site 4 were procured under City Bid Opportunity 203-2018 - 2018 Sewer Inspections with Wessuc Inc. The SONAR inspection was completed by their sub contractor, AquaCoustic Remote Technologies Inc.
3. Testing of the PVC pipe sample obtained from Site 5 were completed by PSI Labs, as a disbursement expense under AECOM’s contract.
4. Underwater CCTV and acoustic leakage inspection services for Site 6 were obtained through an existing services contract with Pure Technologies (Bid Opportunity 154-2017), utilizing their Sahara leak detection platform complete with its underwater camera.
5. An inspection support contract was issued under Bid Opportunity 492-2018, to provide support services for the various inspections, including low head leakage tests on all sites. This work was awarded to J-Con Civil Ltd. in August 2018.

Pipeline modifications and inspection work was completed at all sites between October 2018 and March 2019. Raw inspection data results and material reporting were received from the various testing entities and then subjected to the Condition Assessment (CA) reported herein.

1.1 Site by Site Overview

A brief narrative of work carried out at each site and the results of the assessment follows.

1.1.1 Site 1 – Kildonan-Redwood Feeder Main Crossing

Pipe Assessment

The RFT inspection was completed by PICA in March 2019. The pipe barrel in the inspected portion of the crossing appears to be in good condition with some minor pitting corrosion. However, a low head leakage test on the pipe indicated an apparent leak of approximately 400 L/hr. Additional testing confirmed the apparent leak and visual inspection ruled out leakage within the accessible tunnel and tunnel shaft on the west side of the river, confirming the leak was on the east river bank or beneath the river. Additional testing was completed using the City's correlator acoustic leak detection system but was unsuccessful in pinpointing the location. Based on relatively minor detected corrosion pitting from RFT inspection and other investigations it is most likely that the leakage is occurring at buried flanged joints. Corrosion at the flange locations is not visible to the RFT tool. Flanges are at higher risk to preferentially corroding due to potential breaches in field applied coatings, potential use of stainless steel bolts, and increased stress levels in the flange as a result of bolting. Given the magnitude of the apparent leak and the inability to pinpoint a specific leakage, it is likely that multiple leaks are present on the crossing.

Due to the leakage, the City removed the feeder main from service. It was recommended to proceed with rehabilitation of the crossing using Cured in Place Pipe (CIPP) technology prior to putting the main back into service.

Geotechnical Considerations

The calculated Factor of Safety (FS) for the east bank engaging the pipe is estimated between 1.3 and 1.6. However, the overall FS for shallower global movements is between 1.0 and 1.2 and the FS for toe instabilities is less than one. Stability upgrades to protect the lower slope erosion will improve the factors of safety noted above. Additional slope stability analysis including a more detailed geotechnical investigation is recommended and slope stabilization works to protect the slope should be implemented to protect the slope and pipe from future retrogressive slope movements.

1.1.2 Site 2 – Charleswood-Assiniboia Feeder Main Crossing

Pipe Assessment

Inspection of the Charleswood-Assiniboia Feeder Main was completed in March 2019 by PICA. The RFT inspection identified numerous corrosion related defects with Remaining Wall (RW) thicknesses of as low as 16% of the original wall thickness. During the pipeline modifications, prior to inspection, severe corrosion and a through wall defect were discovered. These fittings were repaired by welding and patching, sandblasted, and recoated prior to reinstallation.

AECOM's structural assessment has estimated the remaining lifespan could be as long as 10 years based on extrapolated corrosion rates alone. Based on the limitations of the analysis and assessment it would be prudent to rehabilitate the crossing within the next 5 year capital cycle. The most cost-effective rehabilitation technique that is technically feasible would be a pulled in place flexible reinforced liner, such as Primus Line. Based on a more detailed assessment, CIPP technology may be determined to be technically feasible as well, subject to a more detailed buoyancy assessment of the crossing.

Geotechnical Considerations

There is no visual indication of slope instability, however, toe erosion is evident near the waterline. Toe armoring of the lower river banks is recommended to address erosion issues and minimize future retrogressive failures.

1.1.3 Site 3 – St. Vital Bridge Force Main Crossing (Aerial, Steel)

Pipe Assessment

During preparation for the EM inspection in October 2018, several small active leaks were discovered by the inspection support contractor at the original pipe joints. Prior to commencement of the inspection program, the City had discovered and repaired a similar leak in the summer of 2018. Subsequent inspection by AECOM, using Ultrasonic Testing (UT) spot wall thickness measurements revealed significant pipe wall thinning (<1 mm remaining in places) preferentially along the invert of the pipeline. Based on the discovered leaks, level of deterioration found, and confirmation that the force main had no internal lining it was concluded that the pipeline had reached the end of its useful service life, and further inspection was not warranted. The proposed external EM inspection program was canceled, and the pipeline insulation was temporarily restored.

As a precautionary measure, the City procured and installed a bypass system routed along the St. Vital Bridge over the Red River. The City issued a separate request for proposal to review rehabilitation and replacement options for the force main.

Based on the assessment carried out under this program it was concluded that in-place rehabilitation using CIPP technology was technically feasible should the City desire to keep the force main in its current configuration on the bridge.

Geotechnical Considerations

Generally, the WWD infrastructure is protected from slope instabilities by the bridge at the site. There is evidence of toe instabilities, however, they would affect the bridge before engaging the pipes. Toe armoring of the lower river banks is recommended to address erosion issues. Slope works are not required to protect WWD infrastructure if the force main is replaced at a different location.

1.1.4 Site 4 – Newton Ave Force Main Crossing (HDPE)

Pipe Assessment

Material testing of the force main completed during HRRC - Phase One found the HDPE pipe to have very low resistance to Slow Crack Growth (SCG), which can make the pipe susceptible to brittle failure in response to long-term exposure to either sustained pressure or intermittent short-term over-pressure. This could be inferred to be consistent with other HDPE force mains of this era (i.e. pre-1980’s HDPE).

SONAR imagery completed under this program identified the pipe had localized areas with very high deflection, hinging, and “dents” (possibly related to third party damage). Subsequent CCTV inspection under partially dewatered conditions verified the observations in those areas. The low head leakage test identified an apparent leak of over 800 L/hr. CCTV inspection by the City identified a circumferential split in the HDPE pipe immediately adjacent to the downstream (west) end of the siphon. The leak was repaired by the City using an internal point repair. Additional leakage testing performed under this program suggests the repair was successful.

Based on the condition of the pipe it is recommended to replace the crossing in the very near term (1-3 years) and provide ongoing monitoring in the interim in general conformance to Environment Act License 2684 RRR.

Geotechnical Considerations

There is no visual evidence of global instabilities at this site. Toe armoring of the lower river banks is recommended to address erosion issues.

1.1.5 Site 5 – Heritage Park Force Main Crossing (PVC)

Pipe Assessment

The initial assessment of the Heritage Park Force Main concluded that it originally had a very high FS against internal and external loading. The governing failure mode was hoop stress and the "as-constructed" FS was >3.6. Assessment consisted of material sampling of the pipe to confirm its relevant physical properties as well as to assess the quality of the original extrusion process and a low-head leakage test of the crossing.

The material testing indicated the mechanical properties of the pipe were more than adequate for the applied loading at the site and resulted in very low wall stresses. The quality of the original extrusion, however, was deemed to be poor as the pipe failed a heat reversion test. This was confirmed in a more robust test using the Differential scanning calorimetry (DSC) method. This is an ISO test that provides more definitive data on original extruded quality (ISO 18373-1) throughout the wall section. Based on the DSC results, the original extrusion quality of the pipe is poor; however, the pipe is not deemed to have any active deterioration processes present due to the very low wall stresses that are present in its current operating mode.

The low-head leakage test indicated that there were no issues with hydrostatic integrity. The only recommended action at the site would be reinspection including a low head leakage test in approximately 25 years as per the River Crossing Management Guidelines[5].

Geotechnical Considerations

This site has an adequate safety factor against global stability engaging the utility. Toe armoring of the lower river banks is recommended to address erosion issues.

1.1.6 Site 6 – Fort Garry-St. Vital (FGSV) Feeder Main Crossing (Cast Iron)

Pipe Assessment

This pipeline crossing was installed with the 1650mm Branch II Aqueduct within a tunnel in 1959. The tunnel was constructed in the limestone bedrock stratum under the river. The tunnel annulus is fully encased in

---

[5] *AECOM, "Water Main Criticality Study Technical Memorandum 2.1 – City of Winnipeg Watermain River Crossing Design, Construction and Management Standard" report for WWD, July 2011*

concrete, elevating the pH of the outer pipe wall to a very high value which creates a very reliable, very low-corrosive environment for the cast iron pipe. The inspection program consisted of an inspection with the Sahara inspection platform under the City's existing contract with Pure Technologies. The inspection did not detect any leaks and the visual interior corrosion was considered minor, consistent with other ferrous metal pipelines in the system. Based on the above, the pipeline is considered to be in good condition with no short-term rehabilitation or replacement required.

In accordance with the River Crossing Management Guidelines[6]. it is recommended to reinspect the crossing in approximately 20 years in the same manner (i.e. visually and with acoustic leak detection).

Geotechnical Considerations

This site was found to have an adequate FS against global stability engaging the FGSV feeder main or Branch II Aqueduct, while slope stability doesn't impact the water utilities, the FS for the embankment engaging the adjacent interceptor sewers is estimated between 1.3 and 1.7. Standard protocol in Winnipeg has been to maintain FS for bank stability intercepting critical infrastructure at 1.5 or higher. Armoring of the lower river banks is recommended to address erosion issues affecting the adjacent WWS siphon crossing.

1.2 Recommendations

The intent of the HRRC Program is to assess the condition of these critical pipeline crossings in a more detailed mode than the initial screening process to gain a quantitative understanding of the failure risk. Our analysis included updating the risk profile developed as part of prior desktop studies for each crossing. While the consequences of failure for each site were not modified in this assessment, the probability of failure has been revised based on the results of the CA process.

Three of the six pipelines inspected in this program were determined to be actively leaking; the Kildonan-Redwood Feeder Main, the St. Vital Bridge Force Main and the Newton Avenue Force Main. These mains have definitively failed (i.e. probability of failure = 100%). The St. Vital and Newton Force Mains have had their active failures repaired and were returned to service, however, the former was subsequently removed from service due to ongoing leakage issues. The Kildonan-Redwood Feeder Main remains out of service, pending rehabilitation. However, all three of these assets are at a high risk of failure moving forward and should be rehabilitated in the short term (within the 5-year capital, prioritized as noted herein). Toe armouring of the river banks should be completed at all sites to prevent erosion. Unaddressed toe erosion leads to the development of more complex retrogressive bank instabilities. The east bank of the Kildonan-Redwood Feeder Main requires a more detailed geotechnical assessment to more accurately assess bank stability and potential stabilization works.

Two of the assets inspected; the FGSV Feeder Main and the Heritage Park Force Main were found to be in good condition and no pipeline remediation is required within current planning horizon. They are considered to have a very low probability of failure at the present time and for the foreseeable future. The assets should be monitored as an ongoing process, and reinspection completed in a 20 year to 25 year horizon, respectively. Riverbank toe stabilization should be considered at both sites to prevent retrogressive failures of the slope. The water assets at FGSV site are not affected by the toe instabilities, however, the adjacent interceptor sewer crossings are.

One site, the Charleswood-Assiniboine Feeder Main, was found to be in fair condition, with a few pits exhibiting less than 20% RW thickness. Through wall perforation could occur in as little as 10 years. Toe armouring as well should be completed to prevent retrogressive bank instabilities.

---

[6] *AECOM, "Water Main Criticality Study Technical Memorandum 2.1 – City of Winnipeg Watermain River Crossing Design, Construction and Management Standard" report for WWD, July 2011*

Based on the assessment carried out in the HRRC – Phase 2 a proposed treatment program to mitigate risk to acceptable levels has been recommended for each site. As per the Phase 1 Program, these were broken into the foreseeable 5 year capital, 5-10 year Capital horizons and longer term reinspection requirements. These are summarized in Table 1 below. The cost estimates have been prepared in accordance with AACE 97R-18[7] and are considered a Class 5 Estimate. The estimates include a 15% allowance for engineering and a 30% estimating allowance . Estimates are presented in 2020 dollars.

Table 1: Proposed Treatment Measures by Site

Site	**Crossing**	**Nominal Diameter (mm)**	**Estimated Replacement Cost (2020)**	**Proposed Work**	**5 year Capital Program**	**10 year Capital Program**	**Reinspection (15-25 Year Frequency; dependent on size of crossing)**
1	Kildonan-Redwood Feeder Main	600	$5,400,000.00	Geotechnical: Toe Armouring & Regrading 2 Sites	$105,000.00		
				Rehabilitation: CIPP	$2,015,000.00		
				Reinspection			
2	Charleswood-Assiniboia Feeder Main	600	$5,600,000.00	Geotechnical: Toe Armouring 2 Sites	$60,000.00		
				Rehabilitation: CIPP	$2,095,000.00		
				Reinspection			
3	St. Vital Bridge Force Main	500	$4,200,000.00	Geotechnical: Toe Armouring 2 Sites	$60,000.00		
				Rehabilitation: CIPP	$805,000.00		
				Reinspection			
4	Newton Avenue Force Main	350	$7,700,000.00	Geotechnical: Toe Armouring 2 Sites	$60,000.00		
				Replacement	$7,700,000.00		
				Reinspection			
5	Heritage Park Force Main	250	$1,300,000.00	Geotechnical: Toe Armouring 2 Sites	$60,000.00		
				Rehabilitation - NR			
				Reinspection/Pressure Test			$15,000.00
6	Fort Garry – St. Vital Feeder Main	600	$6,100,000.00	Geotechnical: Toe Armouring 1 Site*	$30,000.00		
				Rehabilitation - NR			
				Reinspection/Video			$360,000.00
		Replacement Total:	**$30,300,000.00**	**Geotechnical Total:**	**$375,000.00**	**$0.00**	**$0.00**
				Pipeline Total:	**$12,615,000.00**	**$0.00**	**$375,000.00**
				Combined Total:	**$12,990,000.00**	**$0.00**	**$375,000.00**
						Total Work Program:	**$13,365,000.00**

* Required for the adjacent interceptor sewer

---

[7] *Cost Estimate Classification System – As Applied in Engineering, Procurement, and Construction for the Pipeline Transportation Infrastructure Industries – AACE 97R-18, AACE, August 2019.*

As noted above, only one of the crossings does not appear to be a candidate for rehabilitation over replacement. Rehabilitation of the crossings noted herein in a timely manner would save the City of Winnipeg over $16,000,000 in capital expenditures and in conjunction with reinspection at the recommend frequency should reduce the City’s exposure to unanticipated failure for these high failure consequence assets.

2. 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 potable water crossings and 41 sewer crossing locations 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.

Due to regulatory changes in the mid 2000's, the City's of Winnipeg has had a requirement to inspect and assess the condition of the pipeline crossings in their inventory. Previous desktop study's, including the 2006 WWS River Crossing Assessment[8] and the Watermain Critical Study[9] identified assets in the river crossing inventory with very high risk ratings in terms of combined failure probability and consequence. In response, the City's WWD initiated a program to assess the condition of the crossings in greater detail via 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.

AECOM completed the first High Risk River Crossing (HRRC Condition Assessment Program – Phase One (Phase One Program) between 2012 and 2016[10,11]. This included detailed assessment of 19 pipelines at 14 separate locations (13 wastewater and 6 potable water). The Phase One Program also conducted desk top studies and advanced inspection planning for a number of pipelines which had their detailed inspections deferred, largely due to system operational conflicts and pipeline access issues. One Phase One Program site was recommended for additional inspection using alternative assessment technologies. The following sites are included in the current program to complete their detailed inspection based on their deferral from the Phase One Program:

- Site 1 – Kildonan-Redwood Feeder Main Crossing (Advanced internal EM inspection)
- Site 2 – Charleswood-Assiniboia Feeder Main Crossing (Advanced internal EM inspection)
- Site 3 – St. Vital Bridge Force Main Crossing (Advanced external EM inspection)
- Site 4 – Newton Ave Force Main Crossing (Sonar inspection)

The current program also includes two additional crossings:

- Site 5 – Heritage Park Force Main Crossing
- Site 6 – Fort Garry-St. Vital (FGSV) Feeder Main Crossing (added after award)

The results of the advanced CA program are intended to provide definitive direction for input into the City's overall asset management program. The results of the inspections and subsequent assessments guided AECOM's recommendations on remedial works where required and "capital" definitive timelines (e.g. immediate, 5 year, 10 year, 10-25 year) and for re-inspection where no remedial works are required.

---

[8] *UMA/AECOM, "WWS River Crossing Risk Assessment", Report for the WWD, December 2006*

[9] *AECOM, "Water Main Criticality Study Technical Memorandum 2.1 – City of Winnipeg Watermain River Crossing Design, Construction and Management Standard" report for WWD, July 2011*

[10] *AECOM, "High Risk River Crossings – Condition Assessment Report – Sewer Crossings", report for WWD, September 2016*

[11] *AECOM, "High Risk River Crossings – Condition Assessment Report – Water Crossings", report for WWD, November 2016*

2.1 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 inhabitable 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 raised significant concerns over the long-term reliability of this infrastructure. Further some of the eras of thermoplastic pipes used have been known to be problematic (e.g. earlier vintages of HDPE that were used prior to the establishment of a known hydrostatic design basis value for the material) and some, such as PVC crossings, that have never had their condition quantified. Many of the river crossings are also operationally significant and their failure could compromise the City's ability to maintain desired service levels in the distribution and collection systems.

Previous studies investigated the viability of continuous monitoring of these pipelines through leak detection or other monitoring techniques to quantify failure probability. These studies determined that the most cost-effective manner to mitigate future failure risk was deemed to carry out systematic CA using risk-based guidance methods to clarify the most appropriate inspection method and timing[12, 13], which could include leak detection but needed to include a broader range of assessment techniques to be able to both anticipate failure as opposed to confirm failure.

Understanding how these pipelines can fail is critical to developing the most coherent CA approach. Failure in the context of this program is more stringent than the majority of the water and wastewater inventory as a simple loss of hydrostatic integrity of the crossing results in an unregulated discharge of fluid directly to a fish inhabitable water course. For both water and wastewater service this is contrary to the current regulatory requirements as it could possibly damage fish habitat.

The most common failure drivers for the inventory remain unchanged from previous studies and include:

- Material degradation of the pipeline, which results in direct release of fluid or structural failure of the pipeline due to its reduced ability to resist applied loads. While material degradation is generally readily assessable in ferrous metal pipelines it is a far more complex phenomena to assess in thermoplastic pipe materials.
- 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 Winnipeg's river crossing inventory as waterway use is generally restricted to recreation purposes and there is no routine maintenance activities such as channel dredging carried out locally.

The primary focus of CA process for the Phase One Program involved:

1. Attempting to fully understand the applied loads on the pipelines (e.g. to understand how sensitive they are to deterioration processes).

---

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

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

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 and/or indirect assessment of the pipe.
5. Understanding the stability of riverbank crossing and the potential for 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, requires gaining access to the pipe and the use of either direct or indirect measurement CA techniques.

The Phase One Program was almost exclusively ferrous metal pipes and the focus of the program was to utilize continuous measurement electromagnetic (EM) technologies to as great a degree as possible. While of considerable value in the correct application; the Phase Two Program has a markedly different inventory than Phase One and there were considerable lessons learned in the Phase One Program. The following factors were considered in addition to points 1-5 above in implementing the condition assessment process for Phase Two and beyond:

- While continuous EM Technologies can provide considerable insight into future failures, they are largely blind to pipe joints, and don’t eliminate the need for single-event leak detection to confirm hydrostatic integrity.
- In some cases, continuous EM technology is not particularly feasible nor required from a technical perspective to meet overall program technical objectives. The FGSV Feeder Main, for example is a cast iron main constructed in 1959. It was installed in an environment where the likelihood for external corrosion was negligible (it was fully encased in concrete in a large shaft) and internal face corrosion, in Winnipeg’s system is known to reach its limit state in large diameter pipe long before full wall penetration occurs. The pipe joints, from the early days of the modern-day rubber gasket configuration, could be a source of leakage. The inspection strategy, therefore, would more logically utilize a balance of leak detection and visual internal classification of condition, as opposed to continuous EM which is deemed un-necessary in this case and blind to the joints.
- Thermoplastic pipelines cannot be inspected by EM technologies or any other continuous direct assessment technique. Therefore, a balance of leak detection, planned sampling, and intelligent imagery techniques (e.g. SONAR if inspected in the wet and CCTV if dewatering is readily achievable) are most appropriate to assess condition.

Item 5 from the overall process involves a combination of field investigation and desktop analysis of river bank slopes in the vicinity of the pipelines in question. The approach was unchanged from the Phase One Program. Sites are initially visually screened for evidence of instabilities that may engage the pipeline prior to conducting detailed CA techniques. For example, if it is determined that the pipeline is at a high risk of failure due to bank instabilities, and requires relocation, then a continuous EM survey would not be warranted.

The second outcome of the geotechnical program is to recommend further analysis or bank rehabilitation to improve the long-term FS of the pipeline if the pipelines have a remaining service life that warrants undertaking the additional analysis and potential slope stabilization work.

2.2 Program Development

While Section 2.1, highlighted the need to focus closely on alternate CA approaches in greater detail, the overall CA process remains largely as outlined in the Phase One reports[14,15]. The Phase Two Program, therefore, includes the planning to initiate field CA activities, collection of CA data, analysis of the collected information, and reporting of results. Tasks to achieve this includes:

- Inspection Planning and Risk Assessment:
  - Site Investigation and Information Reviews.
  - Preliminary Structural Assessment.
  - Hydraulic Assessment.
  - Logistics Assessment.
  - Risk Assessment.
- Geotechnical Assessment.
- Selection of Technology.
- Inspection Preparation.
- Inspection Program.
- Condition Assessment.

The overall program flow is illustrated in Figure 1.

---

[14] *AECOM, "High Risk River Crossings – Condition Assessment Report – Sewer Crossings", report for WWD, September 2016*
[15] *AECOM, "High Risk River Crossings – Condition Assessment Report – Water Crossings", report for WWD, November 2016*

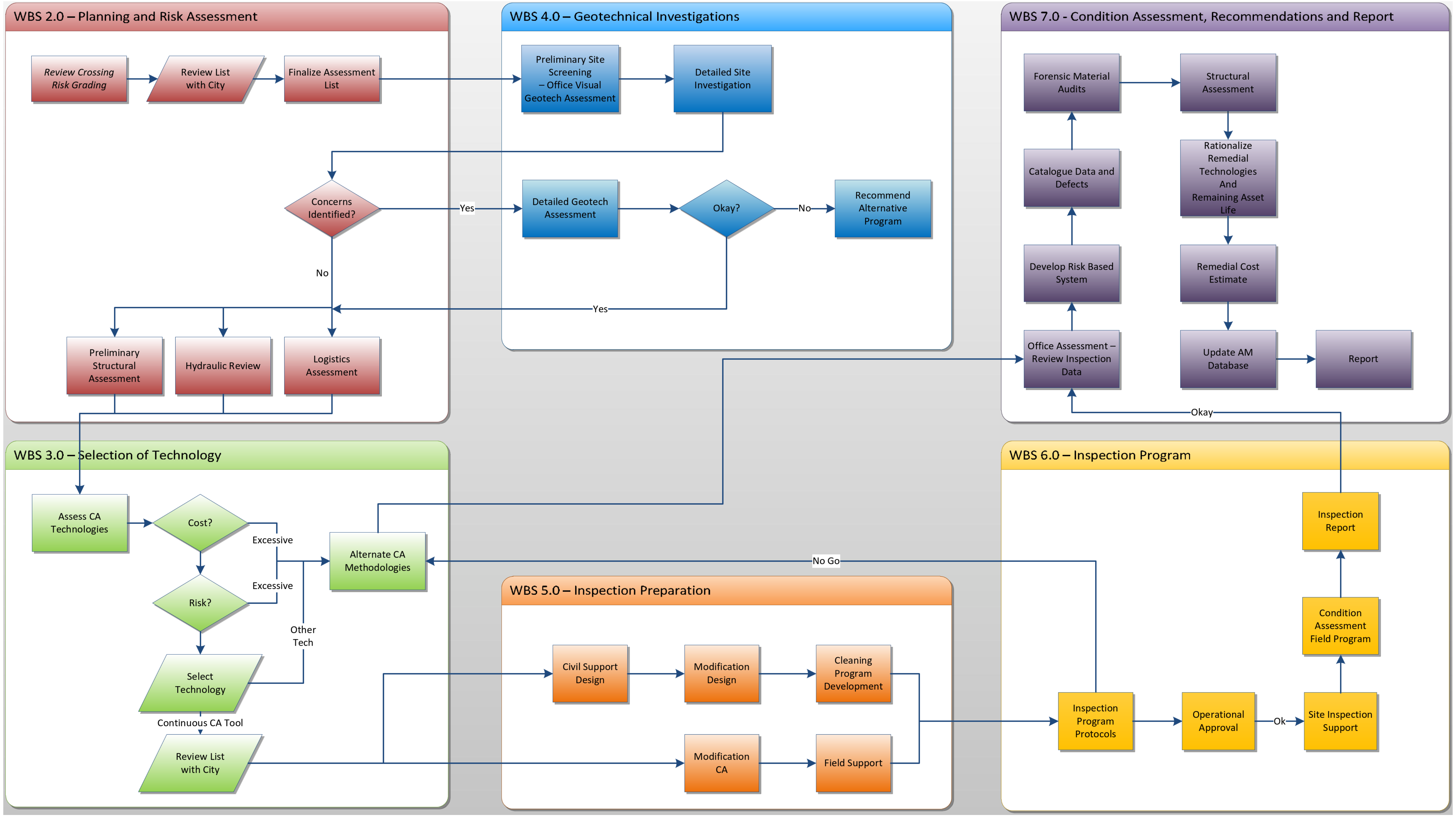


Figure 1: Program Flowchart

3. Inspection Planning and Risk Assessment

3.1 Program Finalization

With the inclusion of the FGSV Feeder Main crossing, the Phase Two Program includes six pipeline crossings at six unique locations as noted in Table 2. All pipelines identified in Table 2 were inspected and/or tested as part of the Program. However, as discussed in the proceeding sections, some of the inspection approaches were modified due to site conditions that were encountered (e.g. the St. Vital Bridge Force Main).

Table 2: Crossings Included in Program

Site	Crossing Location	Nominal Diameter (mm)	Material Type	Installation Year
1	Kildonan-Redwood Feeder Main	600	Steel	1955
2	Charleswood-Assiniboia Feeder Main	600	Steel	1965
3	St. Vital Bridge Force Main	500	Steel	1988
4	Newton Avenue Force Main	350	HDPE	1977
5	Heritage Park Force Main	250	PVC	1989
6	Fort Garry – St. Vital Feeder Main	600	CI	1959

The crossings contained within the Phase Two Program and lessons learned from Phase One Program, resulted in recommendations to proceed with a wider range of inspection approaches than the Phase One Program. This necessitated engaging several different vendors and support services as opposed to a single assessment vendor for all crossings. The following sections outline the inspection planning process from initial information reviews through the risk assessment process.

3.2 Site Investigation and Information Reviews

As the purpose of the HRRC program is to gather detailed information on the condition of these critical assets, there is often a need to sample and deploy advanced inline or external inspection tools (see Section 5). Many of the technologies used were not in existence during the original design of these crossings nor were future CA programs for failure mitigation contemplated until more recent years. Therefore, the systems themselves often require considerable modifications to facilitate the inspection process. The intent of the initial phase of the program is to ascertain the configuration and makeup of the pipeline crossings for the purposes of inspection planning and the design of the necessary pipeline modifications. AECOM completed site investigations for each crossing to assess the following:

- Site access for civil modifications.
- Pipeline access for inspection.
- Valve chamber configurations.
- Restoration requirements.

The investigations included confined space entry into buried valve chambers and limited topographic surveying where required. Confined space entry was supported by the City of Winnipeg.

While technology selection is discussed in greater detail in Section 5, in general, the data review and Site Investigations were initialled focused on undertaking the following inspection work:

- Ferrous metal pipelines crossings (Steel, Cast Iron) – Deployment of inline or external Electromagnetic (EM) inspection platforms.
- Thermal plastic pipeline crossings (HDPE and PVC) – Deployment of internal SONAR inspection platforms and material sampling.

3.2.1 Site 1 - Kildonan-Redwood Feeder Main

The Kildonan-Redwood Feeder Main crosses the Red River immediately north of the Harry Lazarenko Bridge (Redwood Ave / Hespeler Ave) and was constructed in 1955 with a unique combination of tunnelling and open cut construction techniques as shown in Figure 2. Construction of the 600 mm crossing was preceded by failure of an existing 250 mm water main crossing, also located adjacent to the bridge. We understand the original designers opted to tunnel below the west river bank due to known bank instabilities present at this location. The crossing is constructed from steel pipe (see Table 3), utilizing a combination of flanged and Victaulic couplings. Records further indicate the completion of several field welded joints and the installation of a sleeve style coupling near the end of the tunnelled section.

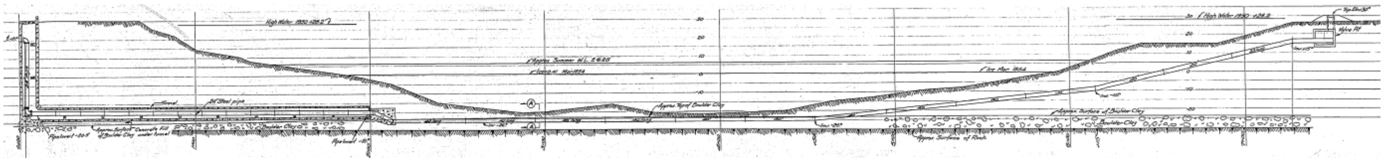


Figure 2: Site 1 - Kildonan-Redwood Feeder Main

Table 3: Site 1 - Pipe Materials

Section	Material	Diameter (OD)	Wall Thickness
Vertical Drop Pipe	Steel	610 mm (24”)	12.70 mm (1/2”)
Horizontal Tunnel Section	Steel	610 mm (24”)	7.94 mm (5/16”)
Buried Section	Steel	610 mm (24”)	7.94 mm (5/16”)

Site 1 was originally included in Phase One of the HRRC Program but removed due to failure of the North Kildonan Feeder Main crossing in 2012 and again in 2014. AECOM completed several investigations between 2012 and 2014 (Phase One Program[16]) and again in 2017 and 2018 under this program to confirm the condition/configuration of the existing chamber piping (see Figure 3), rationalize required piping modifications, and assess construction access within the constrained site.

---

[16] *AECOM, “High Risk River Crossings – Condition Assessment Report – Water Crossings”, report for WWD, November 2016*

Figure 3: Site 1 - Side Outlet 90 Deg Elbow (West Tunnel Shaft)

Site access is limited on both sides of the river as shown in Figure 4. The tunnel shaft and valve chambers on the west side of the river are located within the Redwood Avenue right of way and adjacent easement. The existing valve chamber on the east side of the river is situated within the Hespeler Avenue right of way adjacent to the bridge abutment. Any pipeline modifications and inspections need to be carried out within the confines of the available space.

Figure 4: Site 1 Access

Conclusions of the site investigation and information review included:

- Inspection utilizing an internal EM platform was feasible.
- Configuration of the site crossing is such that pipeline access for the deployment of inline inspection tools would require disassembly of the existing valve chamber piping.
  - West side – Removal of the existing 90 deg elbow at the top of the drop pipe.
  - East side – Disassembly of the valve chamber piping.

- The existing east valve chamber contains piping and valving for operations which are no longer required (see Figure 5).

Through discussions with the WWD, the following works were identified for the east valve chamber:

- Remove the existing hydrant (now redundant) and offtake.
- Remove the existing 300 mm standpipe connection.
- Replace the existing cross with a new 600 x 300 mm tee.
- Remove all valves.
- Install a new valve on the existing 300 mm water main offtake outside of the chamber to facilitate the work and permit system operation.

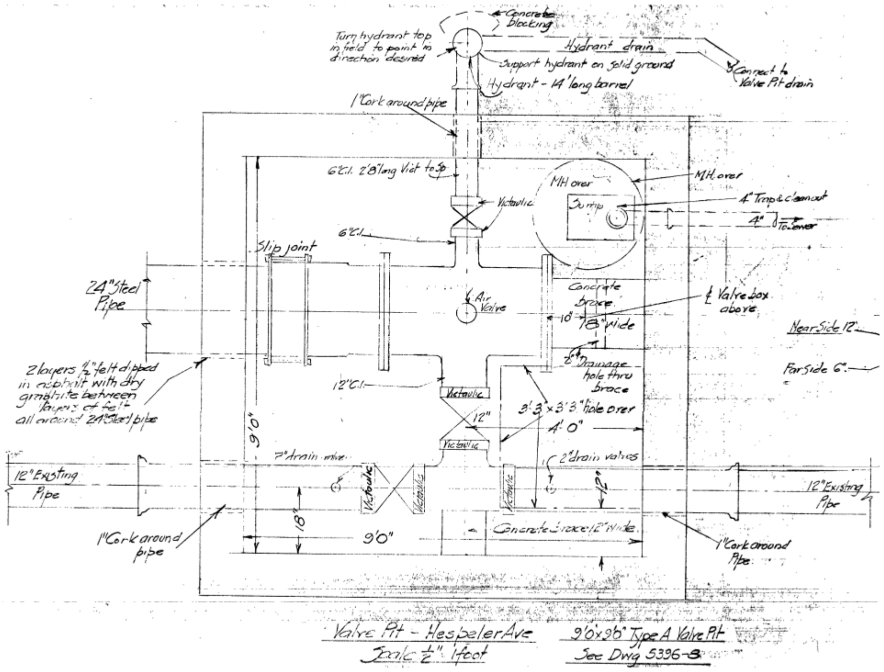


Figure 5: Site 1 - East Valve Chamber

3.2.2 Site 2 - Charleswood-Assiniboia Feeder Main

The Charleswood-Assiniboia Feeder Main crosses the Assiniboine River between Rouge Road (North Side) and Berkley Street (South Side). The feeder main is used to balance pressures within the distribution systems on either side of the river and for redundancy when portions of the system are out of service. The crossing was installed in 1965 via open cut methods as shown in Figure 6.

Records indicate the crossing was constructed from 610 mm (24”), 6.35 mm (1/4”) steel pipe complete with AWWA Class D flanges, coal tar external coating, and AWWA C205 cement mortar lining.

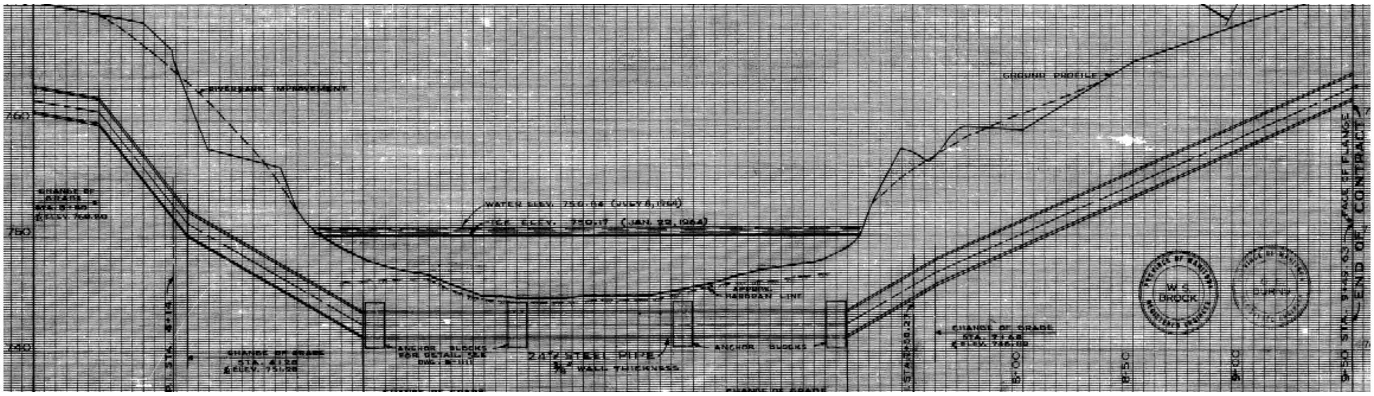


Figure 6: Site 2 - Charleswood-Assiniboia Feeder Main Crossing

The pipeline is situated within existing right of ways on both sides of the river. While the right of way off Rouge Road on the north side of the river is fairly wide open, access on the south side is within a public green space known as "The Passage" for its historical use as a river crossing (Figure 7). Any construction will require protection of existing landscaping and other features.

Figure 7: Site 2 - South Side Access

AECOM originally inspected the chambers at this site in 2011 as part of the City of Winnipeg's Feeder Main Valve Chamber inspection program[17] (Figure 8). Information gathered during that project was augmented by inspections between 2012 and 2014 (Phase One Program) and in 2017 and 2018 as part of this program.

Conclusions of the site investigation and information review included:

---

[17] *AECOM, "Feedermain Valve Chamber Condition Assessment", report for WWD, February 2013*

- Inspection utilizing an internal EM platform was feasible.
- The existing valve chambers cannot be readily disassembled to facilitate the installation of inline tools. Thus, tools must be launched external to the chamber via hard modifications to the pipeline crossing itself.

Figure 8: Site 2 - Rouge Road Valve Chamber

3.2.3 Site 3 - St. Vital Bridge Force Main

The St. Vital Bridge Force Main conveys combined sewage flows from the Baltimore Combined Sewer District via the Baltimore Road Pumping Station and crosses the Red River via an aerial crossing mounted beneath the St. Vital Bridge, see Figure 9. It is also known as the Baltimore Force Main, however, the aerial crossing portion on the bridge, has been referred to as the St. Vital Bridge Force Main. The force main across the bridge was installed in 1988 in conjunction with rehabilitation works on the bridge. The crossing was constructed from 508 mm (20”), 9.525 mm (3/8”) steel pipe installed complete with 50 mm of rigid factory applied polyurethane insulation and spiral wound, 22 gauge galvanized steel cladding. The steel pipe joints were field welded with half shell insulation kits and galvanised steel closures. While the external insultation served as a coating, the main was installed without an internal lining.

The Phase One Program concluded that deployment of an inline EM inspection platform was not practical due to the pipeline modifications required and lack of redundancy. Thus, the site investigation and data review focused on deployment of external non-destructive testing (NDT) platforms, premised on the following:

- The impetus of the program is to assess the condition of the crossings and prevent the inadvertent spill of untreated wastewater (or chlorinated water) to the environment. Since the pipeline is exposed, failure of the pipe and subsequent leakage can be visually monitored.
- Inspection of several locations would be targeted towards locations with a higher probability of deterioration which could be used to infer the condition of the remainder of the pipeline.

Figure 9: Site 3 - St. Vital Bridge Force Main

AECOM completed a preliminary external inspection of the force main on February 8, 2018 with support from the City's under bridge inspection crane truck. The purpose of the inspection was to flag any obvious signs of external pipeline deterioration that would assist in the selection of inspection locations. Beyond some minor surficial corrosion and remnants of previous works (see Figure 10) the pipe appeared to be in relatively good condition.

Figure 10: Site 3 - Preliminary External Inspection

Additional site investigations were undertaken in 2017 and 2018 to ascertain construction access and identify suitable external inspection locations.

The pipe is readily accessible at both ends of the bridge from the river bank off Churchill Drive (North Side) and Kingston Row (South Side) (Figure 11). However, access to the pipeline in locations across the river requires the use of an under bridge crane truck and lane closures on Dunkirk Drive / Osborn Street. Based on past experience north bound lane closures on Osborne are usually limited to after the morning peak period, limiting the working window for the contractor.

Conclusions of the site investigation and information review included:

- Inspection utilizing an internal EM platform is not practical due to the modifications required and lack of redundancy.
- Inspection utilizing an external EM or other NDT platform can reasonably be limited to several discrete locations.
- The preliminary external inspection did not find significant evidence of exterior pipeline deterioration requiring inspection of pipeline above the river (i.e. areas inaccessible from shore). Thus, inspection locations may be limited to those accessible from the shore, significantly simplifying the preparation and inspection process.

Figure 11: Site 3 Access (South Side)

3.2.4 Site 4 - Newton Avenue Force Main

The Newton Avenue Force Main is a twin crossing of the Red River between Fraser's Grove Park and Newton Avenue / Scotia Street. The dual 350 mm force main crossing conveys combined sewage flows from the Linden and Hawthorne Combined Sewer Districts (CSD) via the Linden and Hawthorne Pumping Stations. The crossing was constructed in two stages; the original steel (south) force main was constructed in 1960 and conveyed flows from both force mains. The second HDPE (north) force main was constructed in 1977 and operated in parallel with the steel force main, conveying flows from both pumping stations until 1984 when they were physically separated. In 2014 the two force mains were reconnected and a valve installed between the two upstream valve chambers to facilitate the 2014 Phase One Program inspection works. The operational mode for the system continues to be operation of the force mains separately. Figure 12 and Figure 13 depict the crossing and upstream valve chamber configuration.

Both crossings were originally part of the HRRC Phase One program which resulted in completion of the following inspection work:

- Inline EM inspection of the steel force main. Based on the results of the inspection, the force main is considered to be in good condition and slated for a 20 year reinspection frequency.
- The HDPE force main was sampled and tested for its physical properties. Testing indicated that the pipe is sensitive to SCG under high applied stress loads. AECOM recommended a SONAR inspection to confirm the pipeline's geometry and infer its current stress level.

The HDPE force main was constructed from 350 mm Series 60 (metric) HDPE pipe in accordance with CGSB 41-GP-25M. Series 60 HDPE pipe designates a 60 psi operating pressure. Sclairpipe catalogues from the era of manufacture indicate the pipe would have been manufactured with a 13.97 mm (0.55") wall thickness. Record drawings indicate the pipeline was installed via open cut methods with concrete anchor blocks across the bottom of the river.

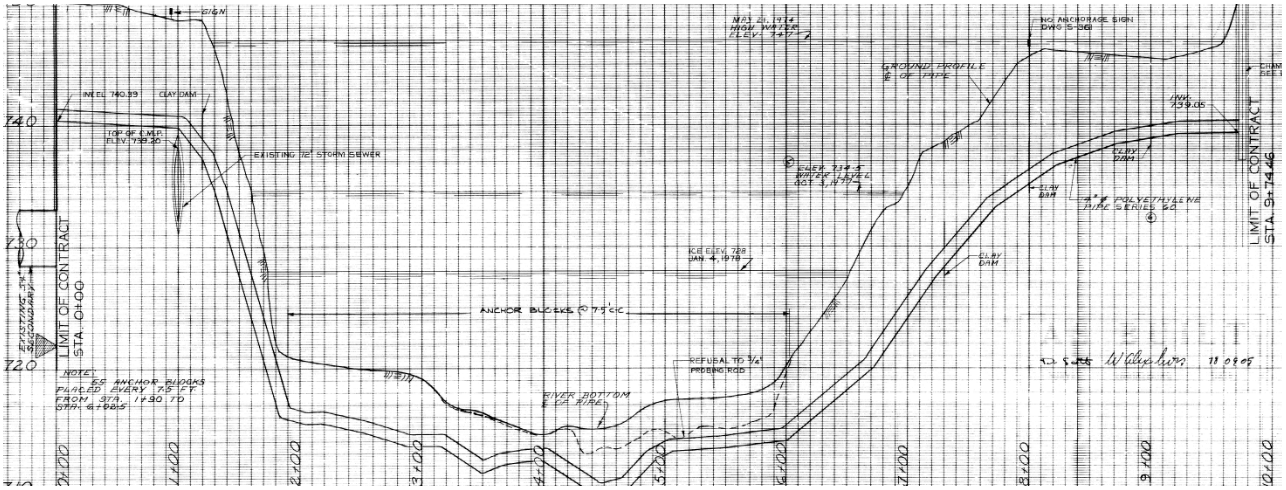


Figure 12: Site 4 - Newton Ave Force Main Crossing

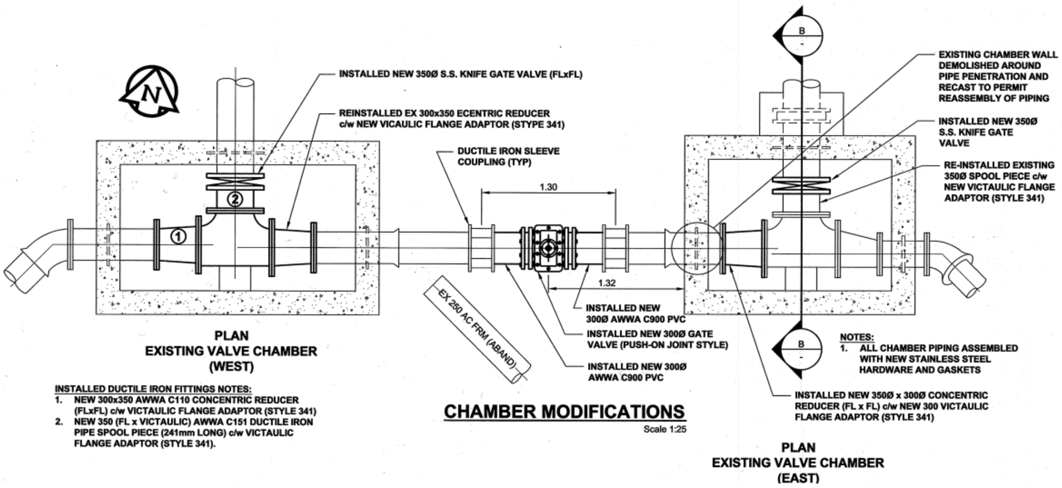


Figure 13: Site 4 - Upstream Valve Chambers

The upstream valve chambers are located within Fraser's Grove Park with access via a multiuse path from Kildonan Dr on both ends of the park. The discharge manhole is adjacent to the Newton Comminutor Station at Scotia Street and Newton Avenue.

AECOM completed several site investigations between 2012 and 2014 (Phase One Program) and again in 2017 and 2018 to ascertain constructability (Figure 14).

Figure 14: Site 4 - North Valve Chamber prior to Phase One Inspections

Conclusions of the site investigation and information review included:

- Inspection utilizing an internal SONAR platform was feasible.
- Civil modifications must include:
  - Disassembly of the upstream north valve chamber allowing access into the force main while maintaining flow through the chamber to the south force main.
  - Temporary removal of the drop pipe within the downstream force main.

3.2.5 Site 5 - Heritage Park Force Main:

The Heritage Park Force Main crosses Sturgeon Creek immediately north of Ness Avenue carrying sewage flows from the Heritage Part Pumping Station. The force main was constructed in 1989 from 250 mm AWWA C905 DR 18 PVC pressure pipe. Portions of the force main were realigned in 2015 to facilitate reconstruction of the Ness Avenue bridge at Sturgeon Creek. Figure 15 and Figure 16 depict the profile and plan of the force main.

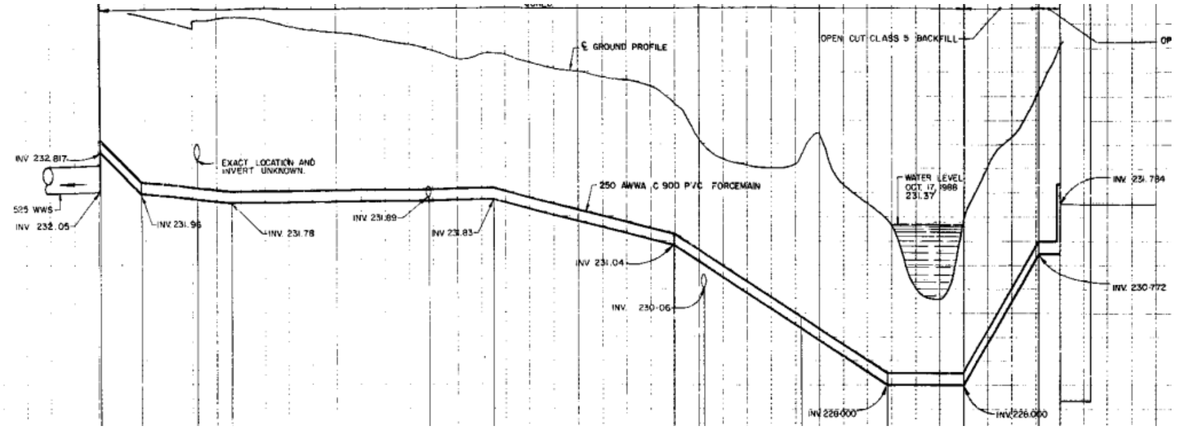


Figure 15: Site 5 - Heritage Park Force Main (Profile)

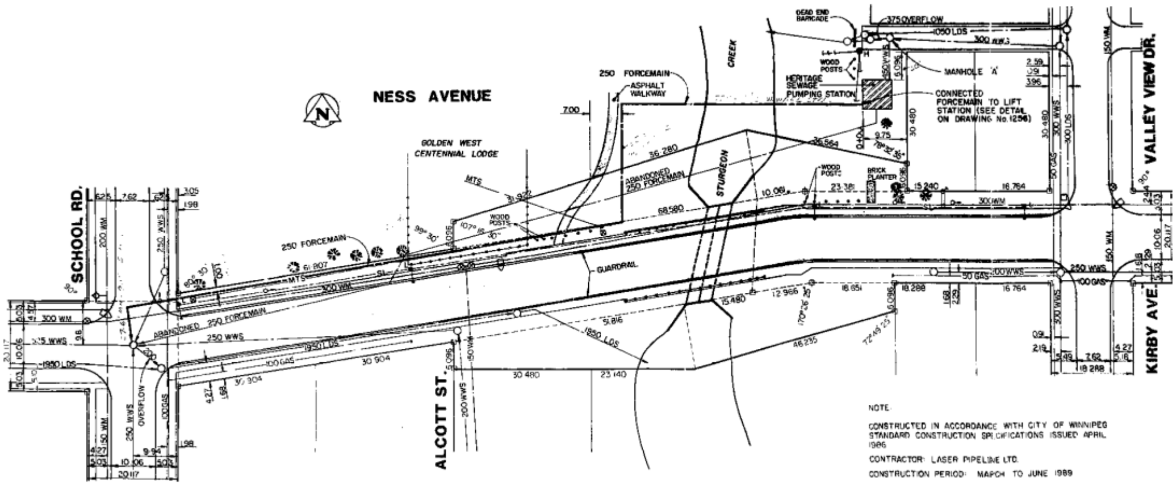


Figure 16: Site 5 - Heritage Park Force Main (Plan)

The Heritage Park Pumping Station is located on the north side of Ness Ave with access from the adjacent alley off Valley View Drive. The force main runs along the north side of Ness Ave before discharging into a manhole in the intersection of Ness Avenue and School Road (Figure 16). Figure 17 depicts both the pump station (left) and force main alignment (right).

AECOM completed several inspections between 2017 and 2018 to ascertain site access and construction feasibility.

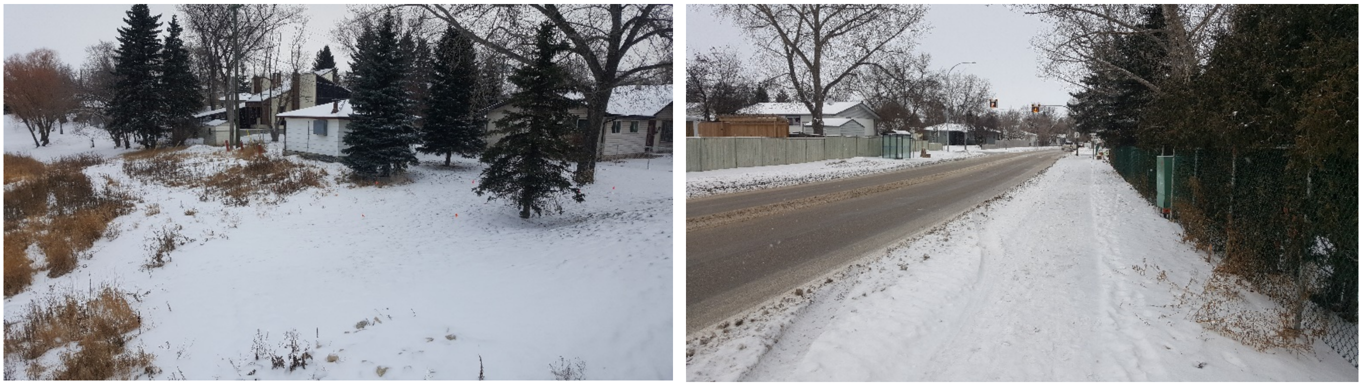


Figure 17: Site 5 Access

Being a thermoplastic pipe, the intent was to deploy an inline SONAR platform to assess the geometry and infer the stress levels that the pipe was subjected to. This, however, would require the following:

- Access into the pipeline from within the pumping station or external to the station through construction of a tool launch assembly.
- Bypass of the force main to facilitate the modifications and inspection.

A review of the force main alignment revealed the presence several 90 deg elbows along the alignment making a complete inspection of the force main impossible due to the inability of most SONAR platforms to traverse sharp bends in smaller diameters. The 90 degree elbows included two immediately outside of the pumping station (see Figure 18), precluding launching an inline tool from within the station. Ultimately the preliminary structural assessment (see Section 3.3) concluded a SONAR inspection wasn't required due to the conservative nature of the original design versus the external loading conditions present.

Conclusions of the site investigation and information review included:

- Inspection utilizing an internal SONAR platform is not practical.
- Sampling and testing to confirm material properties would be sufficient to confirm the condition of the force main:
    - Sampling to occur adjacent to the pump station to avoid other utility conflicts.

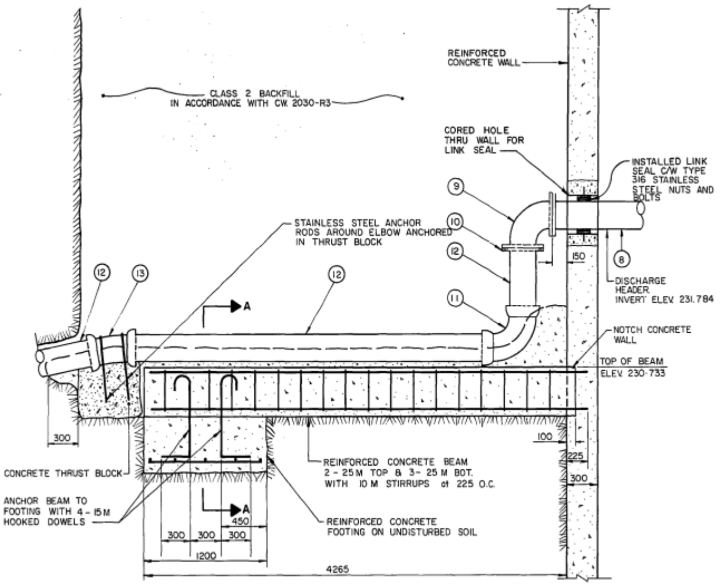


Figure 18: Site 5 - Pumping Station Connection

3.2.6 Site 6 – FGSV Feeder Main

The Fort Garry-St. Vital Feeder Main crosses the Red River between the Fort Garry Bridges (Bishop Grandin Boulevard). The 600 mm feeder main was installed in conjunction with the Branch II Aqueduct construction in 1959 which was completed via tunnelling beneath the river in the limestone bedrock (Figure 19). The crossing was constructed from 600 mm cast iron pipe installed within the Branch II Aqueduct tunnel shaft which was subsequently filled with concrete (Figure 20).

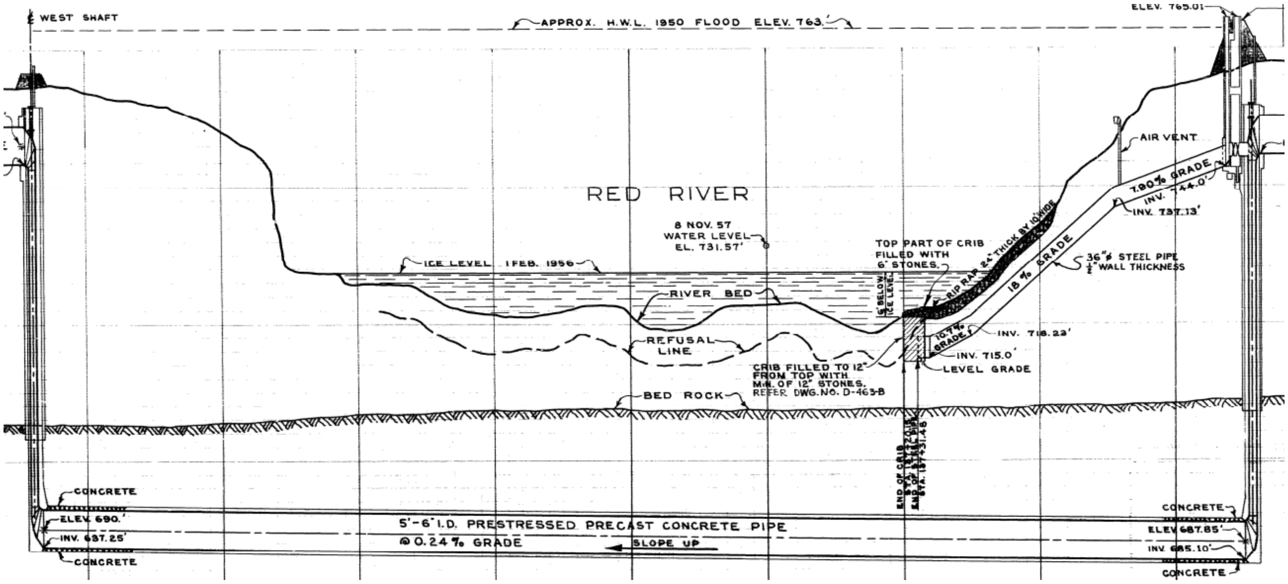


Figure 19: Site 6 - FGSV Feeder Main

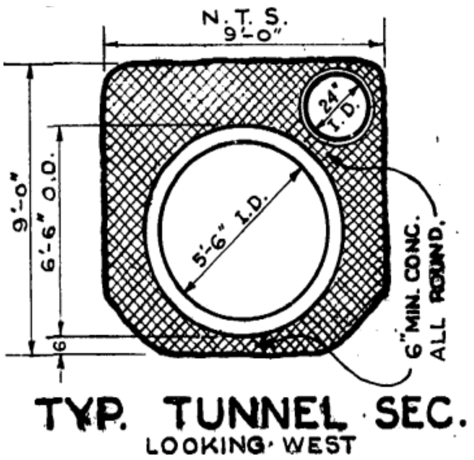


Figure 20: Site 6 - Tunnel Cross Section

To navigate the tunnel shaft and tunnel, the original designers included a series of 90 deg elbows (Figure 21). At the top of the tunnel shaft the feeder main runs north on both sides of the river to adjacent valve/drain chambers. As discussed in Section 5, there are platforms that can traverse 90 degree bends, however, it does increase the risk associated with inspection, especially when there are multiple bends on a single deployment. In addition to 90 degree bends, the adjacent valve chambers contain butterfly valves, which greatly increase the risk of deployment in inline EM Platforms.

Based on the configuration of the crossing, modifications to deploy inline EM platforms could be as extensive as:

- Removal of the top of the Branch II Aqueduct shaft to expose the top 90 degree elbow to facilitate pipe access.
- Reconstruction of the existing valve chambers to facilitate pipe access.

Installation within the Branch II Aqueduct tunnel and encasing the two pipelines in concrete creates an elevated pH environment on the external of the pipe wall resulting in a very reliable, very low-corrosive environment for the cast iron pipe. Additionally, construction of the tunnel within the limestone bedrock effectively shields the pipelines from external loading further reduces stresses within the pipe wall and creating an ideal external operating condition for the crossing. Internal face corrosion has historically not been a large factor in the deterioration of ferrous metal pipes in Winnipeg. Thus, coupled with the external environmental conditions it was concluded that a corrosion related failure was an unlikely failure mechanism for the crossing and inspection geared towards confirming the hydrostatic integrity of the crossing would be sufficient to meet the project objectives.

AECOM undertook a series of investigations in 2017 and 2018 to confirm piping configurations in the adjacent valve chambers on the feeder main which were anticipated to be used for tool deployment. Access to the existing valve chambers is via the D'arcy Pumping Station in Bishop Grandin median on the west side and via the Bishop Grandin Boulevard and/or multiuse path from River Road on the east side.

Conclusions of the site investigation and information review included:

- Deployment of an inline EM inspection platform was not required based on the operating environment of the pipeline.

- Inspection of the pipeline to confirm the interior condition of the pipeline and confirm hydrostatic integrity of the pipeline meets the project objectives.

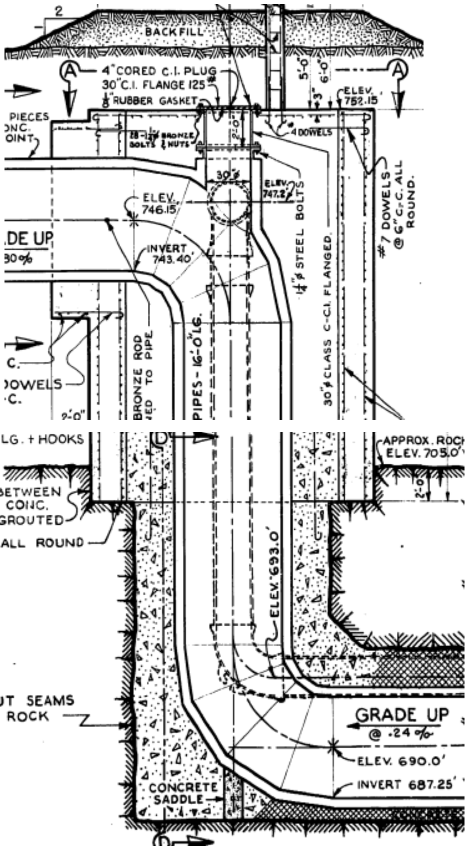


Figure 21: Site 6 - Drop Pipe Configuration

3.3 Preliminary Structural and Operational Assessment

A major input into the program was to conduct an initial structural assessment of each pipeline. The objectives of the preliminary screening included:

- Assess the overall asset FS 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 operational or inspection induced failures. These included the potential for:
  - Inspection induced buoyancy.
  - Inadvertent overpressures.

While a detailed explanation of the structural assessment and calculations undertaken can be found in Section 7, below is a summary of the assessment and the ramifications on program development and development of the final modifications and inspection procurement documents.

Structural and operational checks were undertaken in accordance with current industry design practices, historical design methods utilized in original construction where appropriate, and all available information on the pipeline installation and material properties. As discussed above and in the proceeding sections, preliminary structural reviews provide considerable insight into governing failure modes and allow for a refined inspection program. Detailed assessments for Sites 1, 2, and 4 were completed as part of the Phase One Program and utilized for inspection planning purposes. Preliminary assessments were undertaken for Sites 3, 5, and 6 as part of this program.

The completion of inline inspection works typically requires partially dewatering of the pipelines to facilitate modifications and the inspection work itself. Further, as cleaning of the pipelines is anticipated to include the use of foam pigs, there is the potential to inadvertently dewater the crossing pipes. Thus, confirmation of the FS against flotation is critical to determine during the planning stages to determine if dewatering is permissible. For example, in some cases, dewatering to obtain CCTV inspection is desired. Table 4 lists the FS against floatation for each crossing. Each pipeline was assessed based on available record information as described in Section 7. While all of the crossings have a FS against floatation greater than 1, the calculations are based on record information and actual soil covers over the pipe have not been verified. AECOM recommended limiting dewatering for all crossings.

Table 4: Flotation - Factor of Safety when Dewatered

Site	Crossing	Factor of Safety Against Floatation (Dewatered)
1	Kildonan-Redwood Feeder Main	2.81
2	Charleswood/Assiniboia Feeder Main	2.24
3	St. Vital Bridge Force Main (Aerial)	N/A
4	Newton Ave Force Main (HDPE Siphon Only)	1.65
5	Heritage Park Force Main	7.60
6	Fort Garry-St. Vital Feeder Main (Tunneled)	N/A

3.3.1 Site 1 - Kildonan-Redwood Feeder Main

While the Kildonan-Redwood Feeder Main crossing is constructed from two different thicknesses of steel pipe, the section under the river and exposed to external (soil) loading has a wall thickness of 7.94 mm (5/16”). A detailed structural assessment was completed on the 600 mm steel pipeline as part of the Phase One Program concluded the governing failure mode for the pipe was through wall corrosion. Further, the structural assessment concluded the governing functional/operational limit state to be ring deflection with an FS above the recommended safe FS. The assessment considered an assumed corrosion rate of 0.0797 mm/year, based on data collected during the HRRC Phase One.

The internal pressure capacity of the pipeline was also reviewed consistent with operation of the regional water system with internal operating and transient pressures of 551 kPa (80 psi) and 276 kPa (40 psi), respectively. This resulted in a FS of 2.41 using the ASME B31G[18] method of assessing the effects of corrosion pitting. Again, a pitting rate of 0.0797 mm/year was assumed.

Based on the preliminary structural assessment, the governing failure modes for the pipeline are corrosion related and thus the use of an EM inspection platform to quantity corrosion related defects is suitable.

---

[18] *ASME, “Manual for Determining the Remaining Strength of Corroded Pipelines -B31G–2012, Supplement to the ASME B31 Code for Pressure Piping”, ASME, 2012*

3.3.2 Site 2 - Charleswood-Assiniboia Feeder Main

The detailed structural assessment completed as part of the HRRC Phase One program was completed on the 600 mm steel pipeline and concluded the governing failure mode for the pipe was through wall corrosion. Further, the structural assessment concluded the governing structural consideration to be wall crushing with FS above the recommended safe FS. The assessment considered an assumed corrosion rate of 0.0797 mm/year, based on data collected during the HRRC Phase One.

The internal pressure capacity of the pipeline was also reviewed consistent with operation of the regional water system with internal operating and transient pressures of 551 kPa (80 psi) and 276 kPa (40 psi), respectively. This resulted in a FS of 3.73 using the ASME B31G method of assessing the effects of corrosion pitting. Again, a pitting rate of 0.0797 mm/year was assumed.

Based on the preliminary structural assessment, the governing failure modes for the pipeline are corrosion related and thus the use of EM inspection platforms to quantity corrosion related defects is suitable.

3.3.3 Site 3 - St. Vital Bridge Force Main

As the St. Vital Bridge Force Main is located beneath the bridge, loading on the pipe is limited to longitudinal bending between the pipe supports and internal pressure.

Preliminary structural checks considered a corrosion rate of 0.1166 mm/year based on the average corrosion measured during the HRRC Phase One program on the sewer crossings. This resulted in a FS of 4.61 above an acceptable design for internal pressure using the ASME B31G method of assessing the effects of corrosion pitting.

Longitudinal bending was also reviewed in conjunction with the applicable internal pressures. Based on the original wall thickness of 9.53 mm (3/8”), the pipe currently has a FS against longitudinal bending of 16.38 above an acceptable design FS.

Based on the preliminary structural assessment, the governing failure modes for the pipeline are corrosion and thus the use of EM inspection platforms to quantity corrosion related defects is suitable.

3.3.4 Site 4 - Newton Avenue Force Main

A preliminary structural assessment completed on the 350 mm HDPE pipe and concluded the pipe is extremely sensitive to buckling related failures, specifically with the application of transient vacuums. Consistent with flexible pipe theory, the structural capacity of the pipe is reliant on a suitable pipe/soil structure. While conservative assumptions with respect to soil support were used when analysing the river crossing inventory, the highly variable nature of river crossings and their associated construction methodologies result in a degree of uncertainty with respect to the external operating conditions of the pipe. Further, material testing concluded the pipe was susceptible to SCG failure modes under sustained application of high wall stress (Section 7.2.1). These include localized areas of poor soil support or other external factors causing excessive deflection of the pipe ring.

Given the somewhat fragile nature of the existing crossing pipe, internal pressures during the proposed low head leakage test should be kept to a level consistent with the normal operating pressure. Extreme care should be taken to ensure over pressurization of the pipeline does not occur.

As the pipe is sensitive to external operating conditions and SCG, the use of an inline SONAR inspection platform is suitable to meet the project objectives.

3.3.5 Site 5 - Heritage Park Force Main:

The Heritage Park Force Main is constructed from AWWA C905, DR18 PVC and the preliminary structural assessment concluded the pipe was operating with very low wall stresses relative to the applied loads that would be present. The governing design consideration was buckling with a FS of 3.57. External installation conditions are unlikely to result in failure of the pipe and inspection utilizing SONAR to quantify the shape of the pipeline due to the external operating conditions is a low priority given the extensive modifications and costs associated with obtaining an inspection as discussed in Section 3.2.5.

Therefore, the use of material sampling to confirm material properties for the PVC pipe material is suitable to meet the project objectives.

3.3.6 Site 6 – FGSV Feeder Main:

As discussed in Section 3.2.6, the crossing is installed within the Branch II Aqueduct tunnel within the limestone bedrock beneath the Red River and a concrete filled annulus. This results in a condition where external loading on the pipe is negligible and the potential for external corrosion is very low.

Therefore, the use of an internal CCTV and leak detection platform to confirm the internal condition of the pipeline and hydrostatic integrity of the crossing is suitable to meet the project objectives.

3.4 Hydraulic and Logistical Assessments

As depicted in Figure 1, planning an advanced CA program requires a broad range of considerations to ensure the final program meets the identified objectives without unnecessary impact to the system and elevated costs. Considerable hydraulic and logistical assessments were required to plan overall implementation of the field inspection program.

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. A summary of the operational restrictions and requirements to complete the identified inspections are highlighted within this section.

Hydraulic assessments were undertaken by AECOM for the WWS crossings to assess the incoming flows, determine bypass requirements, and pump station shutdown requirements in order to complete the proposed work. A summary of each site’s unique hydraulic aspects is contained below. Further information on the hydraulic modeling undertaken can be found within the technical memorandums prepared for the following four sites, all of which have been attached in Appendix A.

3.4.1 Water Crossings

The following system operational restrictions were identified by the City of Winnipeg and included in the inspection support contract. These were based on a balance of maintaining desired levels of service and risk mitigation:

- The FGSV Feeder Main crossing cannot be taken out of service concurrently with the Kildonan-Redwood Feeder Main crossing.
- The Charleswood-Assiniboia Feeder Main crossing and Kildonan-Redwood Feeder Main crossings could not concurrently be in state where they could not be put back into service in a matter of hours. Logistically this meant all major modifications were to be completed on the Charleswood-Assiniboia Feeder Main (e.g. all launch wye's needed to be installed) prior to disassembly of the Kildonan-Redwood Feeder Main.

Review of the FGSV Feeder Main Crossing configuration (Section 3.2.6) and the structural assessment (Section 3.3.6) concluded that leak detection and an underwater CCTV inspection would be the most appropriate inspection approach for this crossing. The City of Winnipeg had an existing contract with Pure Technologies, and their Sahara tethered acoustic leak detection and CCTV platform was selected for this site (see Section 5 for a discussion on tool selection). Discussions with Pure identified that a minimum velocity of 1 m/s was required to propel their Sahara inspection tool through the crossing and associated bends. The City's Water Planning group identified that normal operating velocities through the crossing were 0.24 m/s and 0.57 m/s under a modified regional pump station operation regime. Thus, AECOM reviewed the potential to induce flushing velocities through the crossing via the existing 200 mm feeder main drains found in the valve chambers on each side of the crossing. The conclusion of the joint review was the following operational plan:

- Isolate the crossing prior to physical pipe modifications.
- Initiate flushing from west to east by opening the 200 mm drain valve within the east valve chamber and the west feeder main valve. Flushing water would be provided by the Hurst Pumping Station through the FGSV Feeder Main which terminates at the pumping station. Flushing velocities of 1.0 m/s or greater were anticipated through the proposed configuration.
- The 200 mm feeder main drain is connected to the Branch II Aqueduct drain chamber. Flushing water would be dechlorinated from within the drain chamber before discharge to the river.
- Propel the Sahara tool from west to east with the induced flow.
- Cease flushing once the tool is across and inspect by pulling the tool back through the crossing.

The proposed plan limited system impacts east of the river to those related to isolation of the crossing and by completing the flushing operations at night, discoloured water complaints were minimised.

3.4.2 Sewer Crossings

3.4.2.1 Site 3 - St. Vital Bridge Force Main

It was concluded during the Phase One Program that the deployment of inline tools within the St. Vital Bridge Force Main was not practical due to short construction windows (lift station shutdown periods) and high risks associated with completing the work (Section 3.2.3). Thus, as the inspections would be completed externally, no hydraulic assessment was undertaken as part of this program.

3.4.2.2 Site 4 - Newton Avenue Force Main

The Newton Avenue force main crossing consists of a 350 mm steel and 350 mm HDPE force main which convey flows from the Linden CSD to the south and the Hawthorne CSD to the north. The Linden and Hawthorne CSDs represent approximately 400 ha of commercial/residential area (Figure 22).

Figure 22: Newton Ave Force Main - Collection Area

Originally both the Linden and Hawthorne pump stations were serviced by the single 350 mm steel force main. In 1977, a second HDPE force main was added. While originally connected, the two were physically separated in 1984 resulting in each force main conveying flows solely from their respective pumping station. During the Phase One Program, the two chambers were reconnected and a valve installed (Figure 13).

Hydraulic modeling using InfoWorks CS identified that both pump stations could be served by a single force main, as per the original configuration. This was confirmed in 2014 during the Phase One Program where all flows were diverted to the north (HDPE) force main during inspection of the south (steel) force main. To facilitate inspection of the north (HDPE) force main in this program, all flows need to be diverted to the south (steel) force main. This was accomplished by removing the knife gate valve, rotating the tee, and installing a blind flange as shown in Figure 23. Upon completion of inspection, the piping was put back into its original configuration and returned to normal operation.

In order to facilitate the proposed piping modifications, a short-term shutdown of the Hawthorne Pumping Station was required. Figure 24 depicts the projected system water levels during a pumping station shutdown and demonstrates an allowable 12 hr shutdown starting at 10 pm prior to reaching the critical elevation. The critical elevation represented an assumed lowest basement elevation within the catchment area plus a 0.5 m free board.

Figure 23: Newton Ave Force Main – Rotated Tee to Facilitate Inspection

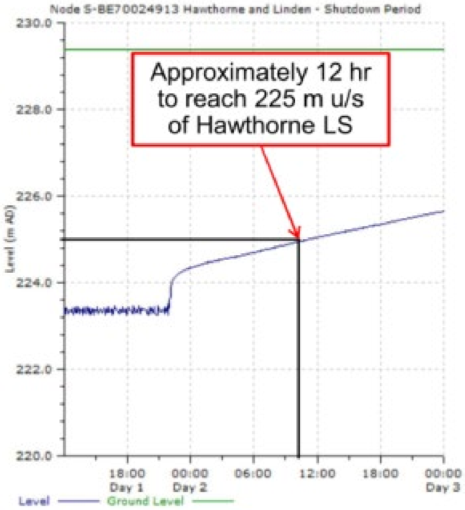


Figure 24: Projected Shutdown Window for the Hawthorne Pumping Station

Changes to the City's operational policies since the Phase One Program in 2014, restricted the use of positive sluice gates from preventing flow storage in the CS piping overflowing to the river. This resulted in the need to install an 1800 mm blocking plug within the trunk sewer at the Hawthorne Pumping Station to facilitate the use of the upstream system for storage during the shutdown. While, the first shutdown was completed in November 2018 without major issue, delays in reinstating the force main to service resulted in completing the second pumping station shutdown in late March 2019 during the spring freshet. To facilitate completion of the pump station during high flow periods, AECOM undertook the following additional efforts:

- Review of incoming flows and atmospheric temperatures to determine an allowable shutdown window which was reduced to approximately 9 hrs.
- Full time monitoring of storage levels during the shutdown to confirm consistency with predicted inflows and remaining shutdown window.

Ultimately the second shutdown was completed successfully within the allowable shutdown window.

3.4.2.3 Site 5 – Heritage Park Force Main

The Heritage Park Pumping Station and Force Main service a 130 ha area of mostly residential development. Figure 25 shows the extent of the Heritage Park Waste Water Sewer District (WWSD). A hydraulic analysis of the Heritage Park WWSD was undertaken to assess the feasibility for short term shutdowns of Heritage Park Pumping Station for the proposes of sampling and leakage testing of the 250 mm force main.

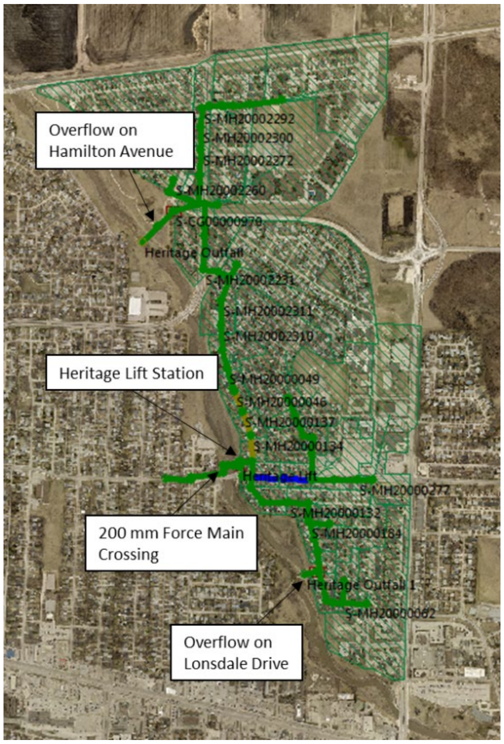


Figure 25: Heritage Park Waste Water District

Review of the system resulted in a recommendation to limit surcharging in the upstream system to the overflow elevation at the Heritage Park Pumping Station, 231.3 m. This resulted in an allowable shutdown window of 5 to 7 hrs when commenced at 12 am (see Figure 26).

Prior to completing the proposed pump station shutdowns, the City completed a trial shutdown which indicated that the tendered 5 hr shutdown window was achievable. Ultimately three pump station shutdowns were successfully completed to facilitate the pipeline sampling and force main leakage test.

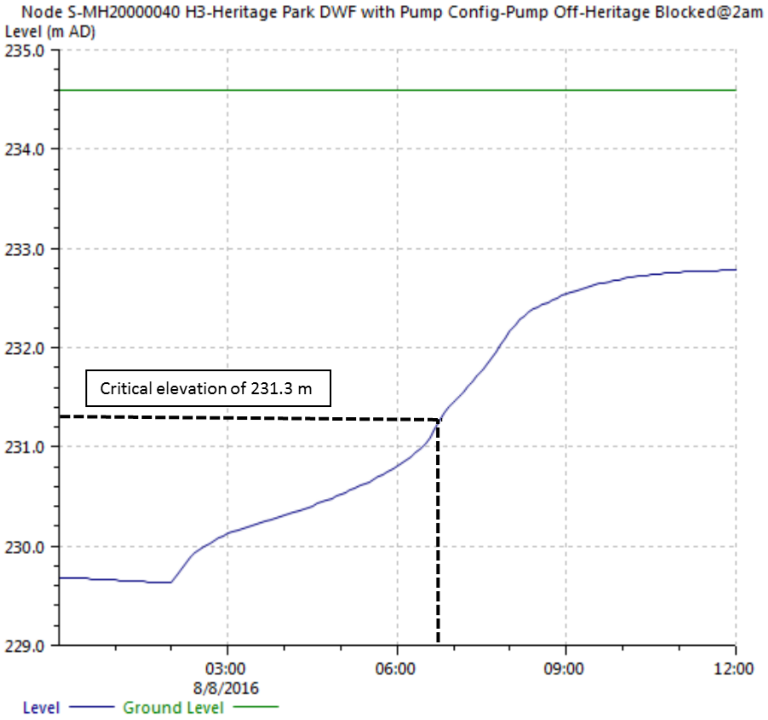


Figure 26: Projected Shutdown Window for the Heritage Park Pumping Station

3.4.3 Disposal of Chlorinated Water

Flushing of feeder mains requires the disposal of chlorinated water in accordance with regulatory requirements and in conformance with AWWA C655[19]. As direct discharge to a fish inhabitable environment is not permittable, the contractor must dechlorinate or discharge to a CS or WWS. AECOM identified the following acceptable options for the discharge of chlorinated water in the Technical Memorandum in Appendix B.

- Discharge to a WWS/CS for treatment at a Pollution Control Center (PCC).
- De-chlorination at source and discharge to river or LDS using Vita-D-Chlor^{TM} Tablets or solution.
- Detention until residuals are within safe limits.

Existing sewers nearby to the feeder main sites were assessed to define acceptable discharge rates based on the capacity of the downstream sewer. These flow rates were identified on the construction drawings issued with the inspection support programs.

For the FGSV Feeder Main crossing, flows of up to 400 L/s were possible under the proposed flushing configuration and the contractor was required to complete dichlorination from within the adjacent Branch II Aqueduct drain chamber though which the flushing water would be directed (Figure 27).

---

[19] *AWWA C655 – Field Dechlorination (C55-18), American Water Works Association, May 1, 2018.*

Figure 27: Site 6 - Flushing Water within the Branch II Aqueduct Drain Chamber

3.5 Risk Assessment

There are numerous factors to be considered in cleaning and inspection programs to mitigate the possibility of inadvertent 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.

AECOM's risk assessment technical memorandum is in Appendix C. A summary of general risks and mitigation factors have been provided below.

3.5.1 Loss of Capacity or Service During Cleaning and Inspection Operations

Low Risk

For sewer force main sites, modifications to gain access to the pipelines are generally more extensive. In some cases they include periods of time where systems are completely taken out of service via invasive construction techniques. Where periodic system disruptions are required, risks will be mitigated by well-planned and conservatively scheduled work plans, such that the system can be returned to service in an appropriate timeframe. Where it is believed that system modifications cannot be completed within the time frames identified for the shutdowns, bypasses will need to be installed. While short term shutdowns are required for the Hawthorne and Heritage Pumping Stations, no major sewer system disruptions are anticipated. A discussion on required pump station shutdowns can be found in Section 3.4.

To mitigate risks associated with pump station shutdowns, tender documents on the Contract included strict requirements for pre planning and procurement of materials for work associated with shutdowns. The Contractor was be responsible for having all parts on site and test fit prior to proceeding with any shutdown.

The largest risks associated with major water crossings can be averted by scheduling of the works away from the high demand water season and with consideration of other regional water operations to ensure that an adequate level of service is maintained. Proposed feeder main crossing shutdowns as noted herein, were not deemed to pose any significant risk or loss of service issues to the regional distribution system, provided shutdowns were scheduled during low demand conditions.

A breakdown of the limitations for removing feeder main crossing from service on this project can be found in Section 3.4.1.

3.5.2 Potential to Aggravate Existing Defects in Deteriorated Pipelines

Low Risk

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 (e.g. the pipeline's in a state of incipient failure) then discovering these defects in a controlled and monitored manner will in and of itself alleviate risk of unattended, unmonitored failures. Cleaning and inspection will result in removal of debris within the pipelines but there is a low risk of increasing pipe wall loss beyond what currently may exist as long as cleaning is done in a staged, controlled manner. The technologies for inspection were specifically selected to navigate the pipelines configuration and do not require aggressive cleaning to a bare pipe wall.

In addition to the deployment of inline inspection tools and physical sampling, the use of low head leakage tests was proposed on all crossing pipelines to confirm hydrostatic integrity. Low head pressure tests (leakage tests) were undertaken at pressures near to the normal operating pressure of the main. This allows for confirmation of current hydrostatic integrity of the crossing without the risk of damaging the pipeline by rupturing a joint or aggravating existing corrosion related defects.

3.5.3 Obstruction of Pipelines Due to Equipment Getting Stuck During the Cleaning and Inspection Process

Low to Medium Risk

The risk of losing cleaning and inspection equipment within the crossings can be mitigated through proper planning and completion of the work by qualified contractors. While cleaning and inspection tasks result in deployment of full diameter cleaning pigs, pipeline pigging is completed in a progressive manner, starting with soft, undersized pigs and progressing to firmer, full sized products as required to achieve the desired level of cleaning. More aggressive pigs are not deployed until previous pigging attempts are proven successful.

Through experience and lessons learned on previous contracts, the pipeline pigging performance specifications required the following:

- Certified and tensile rated tow cables.
- Reinforced cleaning pigs with rated toe cables (Figure 28 depicts a modified pig used in the Phase One Program).
- Controls limiting towing capacity to that of the tow cables and pigs.

Figure 28: Modified Foam Cleaning Pig

In addition to pipeline cleaning, prior to deploying inspection tools, gauge pigs are typically deployed to confirm the pipe bore as depicted in Figure 29.

Figure 29: Gauge Pig with Foam Swab

3.5.4 Buoyancy of Existing Pipelines

Low Risk

Four of the six pipelines to be inspected under this program are dredged into the channel at shallow depths, the remaining two are either encased within a tunnel (Site 6) or aerial (Site 3). Flotation risk for each of the buried pipelines has been assessed for a dewatered state as presented in Section 3.3. Overall, the pipelines exhibit factors of safety above unity (1) and are thus are at a low risk of floatation during inspection, should they become dewatered. However, a requirement to maintain the pipeline in a full, non-dewatered state was included in the cleaning and inspection contracts as an additional means of reducing floatation potential. It is important to ensure that sufficient flow is present behind the cleaning and inspection tools ensuring air entrapment does not occur. This needs to be closely monitored during both submission reviews and the work itself.

3.5.5 Site by Site Summary

3.5.5.1 Site 1 – Kildonan-Redwood Feeder Main

Inspection of the Kildonan-Redwood Feeder Main crossing included the following major work items:

- Isolation of the crossing.
- Disassembly of the existing valve chamber piping to facilitate cleaning and tool deployment.
- Pipeline cleaning.
- Inspection using an inline EM inspection platform.
- Reassembly of the chamber piping.

Beyond the risks discussed above, site specific risks for this location are generally limited to the chamber modifications required to access the pipeline, including:

- Special design fittings: Some of the fittings that are being removed, specifically the 600 mm side outlet 90 degree bend within the west tunnel shaft are unique (see Figure 3) and would have extensive lead times for procurement should they be damaged.
- Reassembly of existing chamber components:
  - Pipe alignment and lay lengths. Challenges can be encountered if the original assembly was poorly executed without recommended joint gaps or misaligned pipes.
  - Procuring new Victaulic components compatible with existing chamber piping can pose issues related to identifying joint styles prior to disassembly and procurement lead times.
- Presence of existing Victaulic jointed valves with non-standard lay lengths.

To mitigate these risks AECOM proposed the following:

- Inspection of the existing chamber to identify Victaulic components (completed during Phase One Program).
- Inclusion of specification requirements for the inspection of necessary Victaulic components by a qualified Victaulic representative and Contractor prior to procurement of components.
- Design of piping modifications to eliminate or account for potential misalignment of piping and reduce risks associated with obscure components.
- Ensure all components are on site and pre-fit prior to cutting into the pipeline.

3.5.5.2 Site 2 – Charleswood-Assiniboia Feeder Main

The site specific risk mitigation were similar to that described for the Kildonan-Redwood Feeder Main crossing, including: Design to reduce construction risk, requirement to confirm dimensions, and pre-fitting of components prior to construction. However, unlike the Kildonan-Redwood Feeder Main, tool launch assemblies were to be installed outside of the existing valve chambers eliminating risks associated with the existing chamber piping.

3.5.5.3 Site 3 – St. Vital Bridge Force Main

The proposed external inspection significantly reduced the risk associated with inadvertent damage to the pipe and loss of service. However, as evidenced during the inspection preparation process, through-wall corrosion defects may be uncovered or disturbed during removal of the pipe cladding and insulation, resulting in an active leak. As a contingency, the contractor was required to have repair clamps on site.

From previous work at this site, it was established that the Baltimore Road Pumping Station can be shut down for up to 8 hours in order to facilitate repairs, should they be required.[20]

3.5.5.4 Site 4 – Newton Avenue Force Main

Inspection of the HDPE Newton Ave Force Main crossing should include the following major work items:

- Isolation of the crossing, including a short-term shutdown of the Hawthorne Pumping Station to complete reconfiguration of the existing valve chamber piping to facilitate cleaning and tool deployment.
- Pipeline cleaning.
- Inspection using an inline SONAR inspection platform.
- Reassembly of the chamber piping.

To mitigate the risk associated with pipeline modification the following should occur prior to undertaking pump station shutdowns and disassembling existing piping:

- All parts are to be on site.
- Existing piping and fittings measured.
- All prep work completed.
- Parts preassembled where possible.

It was anticipated that the time required to complete the piping modifications would be approximately 4 hours per shutdown. Once the piping modifications have been completed the system can operate on one crossing pipe indefinitely, reducing risk during the cleaning and inspection work.

Cleaning pigs will be launched from the upstream valve chamber and pulled through to the downstream manhole. Initial cleaning using conventional sewer flushing equipment was be recommended prior to pigging. It should be noted that SONAR inspections are routinely used to assess the level of debris buildup in pipelines and the inspection probe is not sized to a close tolerance of the pipe ID. As such there is less risk of the equipment becoming stuck in the line than comparable electromagnetic inspection equipment.

Key risk mitigation Items include:

---

[20] *UMA Engineering, Trial Program to Monitor Wastewater Sewer Pipeline River Crossings for Leaks in Compliance with Revised Environmental Act License No. 2669E (Draft), April 2007.*

- Available pump station shutdown windows (12 hours estimate) are well in excess of estimated pipeline modification times.
- All fittings and piping on-site and prepped for assembly prior to pump station shutdowns.
- Operating both Linden and Hawthorne pumping stations through a single force main as per the original configuration has been demonstrated to be viable both through modeling and experience in 2014 (Phase One Program) without adversely affecting pump station operation.

3.5.5.5 Site 5 – Heritage Park Force Main

Inspection of the Heritage Park Force Main is limited to obtaining a sample of the force main for testing and completing a low head leakage test.

Key risk mitigation items include:

- No in-line inspection means there is no need for extensive pipeline modifications or potential to cause a blockage in the line during cleaning.
- The durations for both the sampling and leakage test operations were anticipated to be shorter than the 5 hr shutdown window estimated for the Heritage Park Pumping Station.
- Limiting excavations to non-critical areas away from other infrastructure.
- All fittings and piping need to be on-site and prepped for assembly prior to removing pipe sample.
- Operations required to complete the low head leakage test can be readily removed should there be a need to put the pumping station back into operation in an emergency.

3.5.5.6 Site 6 – FGSV Feeder Main

The deployment of an inline CCTV and leak detection platform vs. an inline EM inspection platform is significant risk mitigation measure in itself due to the significantly smaller impact to the pipeline and the City's regional water system. System modifications for deployment are limited to the installation of a tapping sleeve and valve and replacement of an existing drain valve, none of which require invasive pipeline modifications.

Similar to all other sites requiring pipeline modifications the contractor was required to have all components on site and pre-fit prior to cutting into the pipe.

4. Geotechnical Assessment

4.1 Overview

The intent of the geotechnical program is twofold; first, to evaluate the condition of the river bank by site inspection to assess the general stability of the ground in vicinity of the river crossing assets; and second, to conduct more rigorous stability analysis where warranted and make recommendation for repairs if required.

In the first phase, a visual inspection of the riverbanks was performed in order to assess the existing condition of bank slopes at each of the river crossings. This phase also included a review of existing information, including geotechnical mapping, relevant subsurface information and construction drawings of adjacent structures where available. The visual assessment considers existing features such as over steepened banks, erosion, tension cracks, vegetation cover and other evidence of bank activity. Areas of concern were noted for further investigation. For the highest risk scenarios, where pipeline failure is probable regardless of physical condition of the pipes, re-evaluation of whether to proceed with further investigation is considered. Serious bank instabilities requiring extensive stabilization may result in a relocated pipe asset, irrespective of its condition. Where instabilities are economically reparable, CA of the pipe asset would proceed, and remedial works would be considered and incorporated in recommendations.

In the second phase, a slope stability analysis was completed for crossings where potential bank instabilities were identified based on field inspection, to assess the magnitude of slope failure, and the potential impact on underground infrastructure. In this program, four of the six assigned assets were assessed in the Phase One Program report. The results of the prior assessment are considered in this report. Geotechnical reviews were also performed for the two crossings not assessed under the Phase One Program, in order to assess the bank stability and potential for pipe engagement. Table 5 lists current site designation and description, as well as site designation from Phase One Program reports.

Table 5: Site Designation

Site Designation	Crossing Location	Phase One Program Site Designation
1	Kildonan-Redwood Feeder Main	11
2	Charleswood-Assiniboia Feeder Main	9
3	St. Vital Bridge Force Main	8
4	Newton Avenue Force Main	3
5	Heritage Park Force Main	-
6	Fort Garry – St. Vital Feeder Main	-

The following technical memorandums can be found in Appendix D.

- High Risk River Crossings – Phase 2 - Geotechnical Assessment for Site 5 and 6 (AECOM, September 2018).
- Geotechnical Site Inspections - High Risk Water and Wastewater River Crossings (Phase One Program, AECOM, 2012).
- Geotechnical Slope Stability Analysis - High Risk Water and Wastewater River Crossings (Phase One Program, AECOM, 2016).

Please refer to Table 5 for site number cross references for Phase One Program reports.

4.2 Site Inspection

At each site, experienced geotechnical personnel inspected both riverbanks, identifying and documenting slope features that are potentially linked to bank instability. Visual site inspection is a good first step toward assessing the stability and identify whether or not further subsurface investigation, monitoring or analysis is warranted.

A general description of each site is provided below. Figure 30 provides a summary of the bank inspection criteria, in order of site number. A common observation across almost all of the sites was toe erosion near the normal waterline, as a result of lack of erosion protection. Erosion protection, such as loose rip rap, offers a large degree of protection for a relatively low economical cost. Protection against erosion can aide in prevention of retrogressive shallow slope failures which can lead to reduction of global stability.

- **Site 1 - Kildonan-Redwood Feeder Main**: The west bank at this crossing is known to be unstable. The pipe crossing was originally designed well back from the embankment and is tunneled below the instabilities from a vertical shaft. Although failure scarps and lower bank instabilities are present, the pipe crossing is not contained within the unstable riverbank slope.

  The east bank toe is severely eroded, and the lower bank is over-steepened. No upper bank instabilities were recorded. However, due to the severe toe erosion having a potential to result in retrogressive bank failures, this site was flagged for further assessment to assess whether the pipe is at risk. These results will be discussed later in this document.

- **Site 2 - Charleswood-Assiniboia Feeder Main**: No evidence of instability was noted on the north bank. 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. No erosion control measures are in place on either shore.

- **Site 3 - St. Vital Bridge Force Main:** The pipe is mounted on the bridge, and vertical pipe sections are mounted on the bridge pier on both ends where the pipe enters the ground. The potential of any slope instabilities to engage the pipe are very low. The pipe is shielded by the pier (located upslope of the pier) where it enters the ground on both ends.

  The north bank near the bridge exhibits no evidence of instability and the toe is armored with loose rip rap. However, banks outside of the City Right of Way do show some evidence of bank failure. South bank displays subtle evidence of potential creep displacements and may be the reason for the presence of slope inclinometer casings in the upstream portion. Steep banks and progressive erosion are present downstream of the bridge and is due in part to limited rip rap coverage

- **Site 4 - Newton Ave Force Main:** West bank is in good condition with no evidence of instability. Erosion protection is present at the bank toe. The east bank is also generally in good condition with some erosion at the bank toe. The toe is anchored with the root system from large trees and the mid bank is a relatively flat plane. Erosion may progress further upslope if left unchecked and tree anchorage is undermined.

- **Site 5 - Heritage Park Force Main:** The site is located upstream of both the Ness Ave Bridge and a bend in Sturgeon Creek. Neither the eastern nor western banks exhibit evidence of instability but both do have evidence of toe erosion due to a lack of armoring.

- **Site 6 - FGSV Feeder Main:** The crossing is location between the Forty Garry Bridges on Bishop Grandin Boulevard. The eastern river bank exhibited signs of minor erosion above the riprap but not evidence of slope stabilization issues. The west river bank exhibited signs of riprap loss and erosion. Tension cracking was noted near the crest of the slope but is not believed to be global in nature.

SUMMARY OF SITE INSPECTIONS

									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 IS LEAST SEVERE -3 IS MOST SEVERE	COMMENTS
Site Number	LOCATION	PIPE FUNCTION	RIVER	PIPE DIAMETER (mm)	PIPE MATERIAL	INSPECTION LENGTH (m)	SIDE OF RIVER	NEIGHBOURING STREET	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	NOT EXIST		
1	Kildonan-Redwood Feeder Main	FEEDER MAIN	RED	600	Steel	250	WEST	MAIN ST		X			X		X	X			X			X		X		X		X		1	Cracks present in grouted rip rap at brige abutment and at crossing alignment.
							EAST	GLENWOOD CRES	X			X		X			X	X		X		X			X		X	X		3	Severe toe erosion on crossing alignment resulting in oversteepened toe and recessed river edge.
2	Charleswood-Assiniboia Feeder Main	FEEDER MAIN	ASSINIBOINE	600	Steel	185	NORTH	ASSINIBOINE AVE			X		X	X			X		X		X		X				X		X	1	
							SOUTH	SOUTHBOINE DR			X	X			X		X		X	X			X				X		X	1	Scarp and potential slump block south of crossing alignment.
3	St. Vital Bridge Force M	FORCE MAIN	RED	500	Steel	202	NORTH	CHURCHILL DR	X				X	X			X		X		X	X		X			X	X		1	
							SOUTH	KINGSTON ROW	X				X	X		X		X		X		X			X	X		X		3	Steep bank, erosion, and potential instability east (downstream) downslope of multiple manholes (chambers). Cracks present in concrete drain channels on upper bank.
4	Newton Avenue Force Main	FORCE MAIN	RED	350	Steel	297	WEST	SCOTIA ST	X				X		X		X		X		X	X		X			X		X	1	
				350	HDPE	297	EAST	FRASER'S GROVE	X				X		X		X		X	X			X				X		X	1	
5	Heritage Park Force Main	FORCE MAIN	STURGEON	250	100	100	WEST	VALLEY VIEW DRIVE			X	X		X			X		X	X			X		X	X		X		1	
							EAST	NESS AVE			X	X		X			X		X	X			X		X	X		X		1	
6	Fort Garry-St. Vital Feeder Main	FEEDER MAIN	RED	600	CI	85	WEST	BISHOP GRANDIN BLVD			X		X		X		X		X		X	X		X		X		X		1	
		AQUEDUCT		1650	PCCP		EAST	BISHOP GRANDIN BLVD			X	X		X		X		X		X			X		X	X		X		2	Oversteepened lower riverbank slopes (minor tension cracking)
		WWS Siphon	RED	700	HDPE	85	WEST	BISHOP GRANDIN BLVD			X		X		X		X		X		X	X		X		X		X		1	
				800			EAST	BISHOP GRANDIN BLVD			X	X		X		X		X		X			X		X	X		X		2	Oversteepened lower riverbank slopes (minor tension cracking)

Figure 30: Geotechnical Site Investigation Summary

4.3 Stability Analysis

The results of the stability analysis presented in the geotechnical reports are found in Appendix D.

Stability analysis was performed at Sites 1, 5 and 6 to assess the FS against slope instability. The analysis completed utilized available information from previous soils investigations where available, historic water levels, general soils mapping, available drawings, site investigation observations, and available riverbank LIDAR data. No direct soils investigations were conducted at any sites. Analysis completed is intended to provide a general assessment of slope stability, whether instabilities would engage the assets, and whether more thorough assessment should be carried out in the future. 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.

A FS value ranging from 1.3 to 1.5 is generally considered to be stable and acceptable for infrastructure. A FS of 1.5 is often assigned to critical infrastructure that has a very low risk tolerance for being out of service. Within the range of 1.0 to 1.3, the bank will likely exhibit a higher degree of creep displacement. A FS below 1.0 may imply failure.

Table 6 summarizes the FS for a failure plane that engages the pipe.

Table 6: Global Factor of Safety for Crossings where the Potential Failure Surface Intercepts the Pipe Alignment

Site	Location	Bank	River or Creek	Factor of Safety Affecting Pipe
1	Kildonan-Redwood Feeder Main	East	Red River	1.3 to 1.6
5	Heritage Park Force Main	Both	Sturgeon Creek	1.5 to 2.1
6	FGSV Feeder Main	Both	Red River	1.3 to 1.7*
*** Does not intercept feeder main but does intercept 700 and 800 mm WWS Siphons**				

The following describe the conclusions of the slope stability analysis:

- Site 1 – The east embankment has a reasonable FS against global movement engaging the pipe. The computed safety factor ranges from 1.3 to 1.6 which is slightly less than normally accepted for critical infrastructure. Analysis has indicated that shallower seated rotational failures have a lower safety factor of between 1.0 and 1.2, and stability of the toe has a safety factor less than unity, approximately 0.8. If left unaddressed, continued erosion damage will lower the factor or safety for shallower rotational failures, and eventually may lead to engagement of the pipe.
- Site 5 - The 250 mm force main is not currently at risk due to slope instability within the creek bank slopes. Current global slope stability FS values are greater than 1.5 in all cases. There however is some evidence of toe erosion.
- Site 6 - The 600 mm feeder main and Branch II Aqueduct are not currently at risk of slope instability effects. The computed factors of safety are in excess of 1.5 for global stability. However, the adjacent 700 mm and 800 mm HDPE interceptor sewers have a slightly higher risk of being engaged by a failure surface, between 1.40 and 1.45, during periods of low water level (i.e. during the winter months).

4.4 Synopsis of Geotechnical Review

Generally, the sites reviewed in this report are considered to be reasonably stable against global failures engaging the Water and Waste Infrastructure. All sites exhibited evidence of toe erosion. If toe erosion is left unchecked, it can, with time, lead to retrogressive failures that can lower the overall FS of the slope. Monitoring the banks and toe armouring is recommended if erosion continues.

The exception is the east embankment at the Kildonan Redwood Feeder Main. While the FS of a slope failure engaging the pipe is currently reasonable, the FS against continued toe erosion is very low, less than unity, which will very likely result in retrogressive slope failures, leading to global failures. It is recommended that a more thorough slope analysis be completed, including subsurface investigations and monitoring, to better define the type and extents of remediation. At a minimum, toe armouring should be considered in the short term.

5. Technology Selection

The Technology Selection Technical Memorandum can be found in Appendix E. A summary of the recommendations from the Memorandum is provided below.

A key element of the HRRC program was selection of appropriate inspection technologies to facilitate a more detailed understanding of pipeline condition. The Phase One Program generally focused on ferrous metal pipelines and thus utilized advanced electromagnetic inspection technology in the form of Remote Field Technology (RFT). The current program contains several thermoplastic pipelines which require alternative technologies to facilitate CA and estimation of their remaining service life.

The selection of inspection tools needs to focus on obtaining sufficiently accurate information for performance governing defects without inducing excessive risk or compromising the long-term life of the pipeline. A variety of material-specific technologies and broad-use technologies (apply to virtually any pipe material) were reviewed for use inspecting the Winnipeg river crossing pipes, including:

- Electromagnetic (EM) tools which can detect wall loss in ferrous metal pipe.
- Remote camera inspection (CCTV) which can reveal visual defects.
- SONAR which can detect debris accumulation, air pockets, and define the geometric shapes of any pipe material.
- Leak Detection systems which can detect leakage in higher pressure systems.
- Ultrasonic (UT) measurement that can detect wall thickness and defects within the pipe wall.
- Hydrostatic leakage pressure testing which can confirm if asset is actively leaking.
- Opportunistic sampling which can confirm pipe material properties.

Broad-use technologies such as closed-circuit television (CCTV) and SONAR technologies can be used for any pipe material to gather valuable information to supplement the CA process, or to assess tool deployment risk. Their use in CA, however, is limited to “visual” classification only of defects on the interior face of the pipe, and the way the pipe is reacting to the soil stresses around it. In gravity pipes this is often an adequate level of assessment. In pressure pipe flow, however, even when the pressures are low, more quantitative data on residual pipe structure is a desired outcome of the CA process.

Material-specific inspection technologies, such as EM platforms used on ferrous metal pipes, were developed to acquire quantitative physical information on residual pipe structure and/or to detect specific types of defects and defect geometry. Although deployed from only one side of the pipe (internal or external) they can obtain quantitative data of physical condition beyond the visible surface. Unfortunately, EM 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 capabilities of various tools to detect and quantify 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.

Presently there are no proven inspection platforms developed for continuous measurement of pipe wall condition for the non-metallic crossings. Therefore, the technical approach for CA of the non-metallic pipes needs to be a balanced approach of alternative techniques, planned and/or opportunistic sampling and reviewing relevant mechanical properties over time, and a thorough understanding of the applied loads on the pipe to indirectly assess condition and material failure risk. The technical approaches for assessing the Newton Ave (HDPE) and Heritage Park (PVC) Force Mains are discussed separately in this Section.

Pipelines included in this program include four ferrous metal pipelines (three steel and one CI) and two thermoplastic pipelines, (one HDPE and one PVC). Defect and failure mechanisms for the various pipe types and technologies that can be used to detect these include:

Ferrous Metals (Steel, Cast Iron)

- External general corrosion - EM tools
- Pitting corrosion/pinhole corrosion usually the result of spot defects in protective coatings - EM Tools
- Graphitization of metal (in case of Cast Iron) – EM tools
- Splitting as a result of excessive internal pressure – Pressure Testing, EM tools
- Excessive deflection as a result of external loads – SONAR, CCTV, gauge pigging/mandrels
- Buckling as a result of excess external pressures -SONAR, CCTV, gauge pigging/mandrels
- Internal erosion as a result of high velocities -SONAR, CCTV, EM tools

PVC and HDPE

- Excessive deflection resulting from external loads - SONAR, CCTV, gauge pigging/mandrels
- Buckling as a result of excess external pressures -SONAR, CCTV, gauge pigging/mandrels
- SCG in HDPE pipe as a result of long term stress, particularly in non-modern high performance PE resins – Pressure testing, opportunistic sampling
- Cyclic fatigue stress particularly in PVC pipe - Pressure testing, opportunistic sampling
- Hoop tension failures (splitting) as a result of excessive internal pressure -Pressure testing
- Internal erosion as a result of high velocities and debris- SONAR, CCTV

5.1 Electromagnetic Tools for Ferrous Metal Pipelines

Similar to the Phase One Program, the use of continuous EM inspection methods such as Remote Field Eddy Current (RFEC or RFT) or Magnetic Flux Leakage (MFL), where feasible, was preferential to spot inspections. The objective of the program is 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.

The following technologies were reviewed for use within the ferrous metal pipelines:

- Inline Remote Field Technology (RFT) from Pipeline Inspection and Condition Assessment Corporation (PICA)
- EM Technology (formerly Enhanced EM) from Pure Technologies Ltd. (Pure)
- External Bracelet Probe (RFT) from PICA or Broadband Electromagnetic Bracelets (BEM) by others
- Magnetic Flux Leakage (MFL)

Our assessment indicated that Pure’s EM Technology did not have sufficient resolution to identify the types of defects anticipated and MFL tools were likely unable to traverse the geometry of the crossings. Where continuous EM technology was desired, the recommendation was to utilize an RFT platform as offered by PICA.

5.2 Other Inspection Technologies

Other inspection technologies reviewed were SONAR and CCTV/leak detection combination platforms for use in pressurized pipelines.

SONAR technologies have been used in the City of Winnipeg regularly on the Sewer Condition Assessment Program for inspecting siphons and large diameter sewers. SONAR inspections are useful in detecting pipe geometry, air pockets and sedimentation, which can be indicative of the external operating environment for flexible pipelines, but especially useful for thermoplastic pipes where more advanced inline inspection tools are not available. The intention was for any SONAR inspections to be completed in conjunction with the City's annual condition assessment program.

Leak detection platforms are useful for pinpointing leaks and when coupled with CCTV can provide useful insight into the internal condition of pipelines. There are several platforms available in the marketplace, including Pure's Sahara tool and the LDS 1000 and Investigator from Game Trenchless Consultants. The Game tools have reduced allowable inspection lengths from that of the Sahara tool of 1000 m and 100 m for the LDS 1000 and Investigator, respectively, but otherwise provide similar data. As the City held a contract with Pure Technologies at the time of inspection, the Sahara tool was the recommended tool for the inspections.

5.3 Site by Site Summary

Table 7 provides a site by site breakdown of the recommended inspection technologies for this program. The proceeding sections provide additional rationale for the recommendations listed.

Table 7: Recommended Inspection Technologies

Site	Diameter (mm)	Pipe Material	Governing Failure Mode	Recommended Inspection Technology
1	600	Steel	Corrosion pitting	Inline RFT
2	600	Steel	Corrosion pitting	Inline RFT
3	500	Steel	Corrosion pitting	External RFT
4	350	HDPE	Slow crack growth due to long term wall stress	SONAR
5	250	PVC	Original manufacturing quality	Material Testing
6	600	Cast Iron	Internal face corrosion	CCTV/Leak Detection

5.3.1 Site 1 – Kildonan-Redwood Feeder Main

Based on the discussion in Section 3 and above, AECOM recommended the use of an internal RFT EM inspection platform for the Kildonan-Redwood Feeder Main crossing. The RFT inspection platforms reviewed as part of this program can typically navigate the 90 degree bend found within the west tunnel shaft. If there are concerns with navigating the bend, inspection can easily be completed from either end of the pipeline through two insertions.

5.3.2 Site 2 – Charleswood-Assiniboia Feeder Main

Based on the discussion in Section 3 and above, AECOM recommended the use of an internal RFT EM inspection platform for the Charleswood-Assiniboia Feeder Main crossing.

5.3.3 Site 3 – St. Vital Bridge Force Main

Based on the discussion in Section 3 and above, AECOM recommended the use of an external RFT EM inspection bracelet for spot inspections on the St. Vital Bridge Force Main crossing. To facilitate inspection utilizing an external RFT tool the pipeline cladding and insulation would be removed at discrete, select locations. The discrete inspection locations were used to infer the condition of the crossing as a whole.

Through the information and preliminary investigations completed by AECOM (see Section 3.2.3), five discrete locations were recommended, all approximately 2.7 m long (Figure 31).

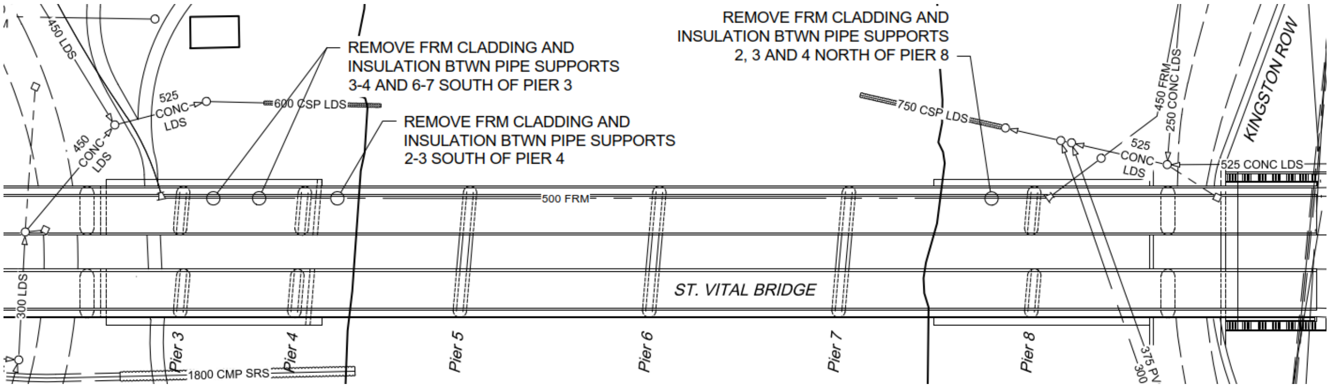


Figure 31: Site 3 - Proposed Inspection Locations

5.3.4 Site 4 – Newton Ave Force Main

Based on the discussion in Section 3 and above, AECOM recommended the use of an internal SONAR inspection platform for the Newton Avenue Force Main crossing. This follows the material sampling and testing completed under the Phase One Program.

5.3.5 Site 5 – Heritage Park Force Main

Based on the discussion in Section 3 and above, AECOM recommended sampling and material testing for the Heritage Park Force Main crossing.

PVC, as with all thermoplastic materials, does not corrode and unless exposed to corrosive waste streams will not normally deteriorate over time due to most environmental exposure conditions. PVC, however, is susceptible to fatigue from cyclic loading, and a variety of wall stress failures due to excessive applied loads. As discussed in Section 3.3, a desktop study determined that the pipe's design was very conservative assuming the pipe was manufactured to the original manufacturing specifications and was not breaking down. Thus, it was recommended that a sample be collected for material testing. The SONAR inspection, while a valuable indicator of stress was deemed to be too complex (and expensive) due to the configuration of the force main and would be of limited value relative to the governing failure mode of the force main.

5.3.6 Site 6 – Fort Garry St. Vital Feeder Main

Based on the discussion in Section 3 and above, AECOM recommended inspection using a combination of CCTV and leak detection for the FGSV Feeder Main crossing.

The cast iron pipe is encased in concrete within the underlying bedrock creating an ideal environment for the crossing as the pipe is generally shielded from all overburden loads and is protected from corrosion due to the high pH environment created by the concrete. Thus, exterior corrosion is not anticipated to be a factor in the deterioration of the cast iron pipeline. Interior corrosion of ferrous metal water mains is typically not a driver in pipeline deterioration in the City of Winnipeg due to both water quality and the nature of the interior corrosion process, which typically results in more uniform corrosion than that of exterior driven corrosion processes. As excessive corrosion is not anticipated for the crossing, an in-line leak detection survey combined with visual inspection was recommended for this site. A visual inspection would allow clarification of a number of key factors:
‘

- Was an interior cementitious coating utilized in the original manufacturing of the pipe?
- Visual assessment of interior corrosion.
- Determine debris levels. Given the nature of the siphon with vertical drop shafts and 90 deg bends, there’s the potential for debris buildup within the siphon. An assessment of debris levels would be a prudent step prior to undertaking cleaning and more advanced inspections, should they have been deemed necessary.

5.4 Procurement

Due to the variation in pipeline materials and corresponding inspection requirements, inspection services were procured through several vendors as described below.

5.4.1 EM Inspection

The Request for Proposal (RFP) 495-2018 was prepared for the purposes of procuring an EM Inspection Contractor. The RFP included both the internal EM inspection of Site 1 and Site 2 as well as the external inspection of Site 3. The RFP closed on June 29, 2018 with two proponents submitting proposals, Pure and PICA proposing the following technologies:

PICA:
- Internal RFT Chimera inspection platform.
- External RFT Bracelet Probe inspection tool.

Pure:
- MFL internal inspection platform.
- External BEM inspection tool.

Ultimately, Pure was unable to demonstrate their ability to traverse the pipelines utilizing their MFL tool and the Contract was awarded to PICA.

5.4.2 SONAR Inspection

The SONAR inspection was procured as part of the City's 2018 Sewer Inspection contract, Bid Opportunity 203-2018 developed and administered by AECOM. The contract was awarded to Wessuc Inc. who utilized AquaCoustic Remote Technologies Inc. as their SONAR sub contractor.

5.4.3 Leak Detection/CCTV

The City held an existing contract with Pure for the provision of non-destructive pipeline inspection under Bid Opportunity 154-2017. The contract included inspection using their Sahara inspection platform.

6. Field Inspection Program

6.1 Overview

Field inspections were completed in the fall of 2018 and spring of 2019. A total of four pipelines were successfully inspected using inline tools, one pipeline was sampled for material testing, and one pipeline inspected externally.

As discussed in the preceding Sections the program included the use of multiple inspection technologies, not all available via a single vendor. Thus, the program included the following contractors:

- Request for Proposal (RFP) 495-2018 (Provision of Non-Destructive Inspection Services for Pipeline River Crossings):
  - Contractor: PICA
  - Services: Internal and external advanced electromagnetic inspection using RFT
  - Pipelines to be inspected:
    - Site 1 – Kildonan-Redwood Feeder Main (inline inspection)
    - Site 2 – Charleswood-Assiniboia Feeder Main (inline inspection)
    - Site 3 – St. Vital Bridge Force Main (external inspection) (Canceled prior to inspection)

- Bid Opportunity 203-2018 (2018 Sewer Inspections):
  - Contractor: Wessuc Inc. (Multisensor Inspection Subcontractor: Aquacoustic Remote Technologies Inc.)
  - Services: Inline SONAR inspection
  - Pipelines to be inspected: Site 4 – Newton Ave Force Main

- Bid Opportunity 154-2107 (Provision of Non-Destructive Condition Assessment of Water & Sewer Pipelines):
  - Contractor: Pure Technologies
  - Services: Acoustic leak detection and CCTV (Sahara platform)
  - Pipelines to be inspected: Site 6 – FGSV Feeder Main

- Bid Opportunity 492-2018 (Provision of Pipeline Access Modifications, Cleaning and Support Services for River Crossing Inspections):
  - Contractor: J-Con Civil Ltd. (J-Con)
  - Services: Pipeline modifications, pipeline cleaning, pipeline sampling, and inspection support
  - Pipelines to be inspected: Support for all inspections

Both Bid Opportunity 492-2018 and RFP 495-2018 were procured under this program, while the remainder of the contractors (Aquacoustic and Pure) had previously been procured or were engaged through other similar work. J-Con was engaged as the general civil support contractor and required to prep for and support all of the various inspections.

Record drawings (draft) for the pipeline modifications can be found in Appendix F.

6.2 Site 1 – Kildonan Redwood Feeder Main

The contractor commenced with the preparation of Site 1 on February 28, 2019. The pipeline modifications included the following:

- Installation of a new 300 mm gate valve external to the east valve chamber for isolation of the distribution system.
- Exposure of chamber and removal of roof slabs from chambers on both sides of the river.
- Disassembly of the east valve chamber. Valve chamber piping reconfigured upon reassembly.
- Removal of the existing side outlet 90 degree side outlet bend in the west drop shaft chamber. Bend reinstalled upon completion of work.
- Miscellaneous chamber repairs to facilitate piping reconfiguration and chamber closure.
- Pipeline cleaning.
- Drawings: D-15158, D-15159

6.2.1 EM Inspection

The EM inspections occurred between March 26th and 28th, 2019 without major issues. Figure 32 depicts insertion of the tool at Site 1, through the existing east valve chamber. PICA's inspection reports can be found in Appendix H. On March 27, 2019, after completion of the inline inspection on the Kildonan-Redwood feeder main, PICA and AECOM completed an inspection of the feeder main piping within the drop shaft and tunnel supported by J-Con. AECOM's inspection report can be found in Appendix G.

In addition to PICA's inspection reports (Appendix H), overlay plots of PICA's inspection results on air photos/GIS plots have been included in Section 6.8. The general findings of the electromagnetic and tunnel inspections are as follows:

- Nine pipes were identified to have localized wall loss with a total of 18 discrete defect locations. The worst corrosion related defect identified had 40% remaining pipe wall, with a majority of the defects having 65% remaining pipe wall. Defects appear to be relatively evenly spread along the length and circumference of the pipe outside of the tunnelled section.
- The tunnel inspection found the existing pipe and coatings were in relatively good condition. No noted corrosion of the pipe itself and only several blemishes in the coating were observed. The tunnel inspection did note that the existing Victaulic couplings and ventilation piping components were severely corroded.

Figure 32: Site 1 - RFT Tool Insertion

Issues encountered during the inline EM inspection and preparation work included:

- Inadequate measurements prior to procuring the new steel pipe components within the east valve chamber resulted in modifications to how the 600 mm welded spool piece was installed. It was ultimately, fit inside a portion of the existing slip coupling and welded externally to the existing coupling.
- The contractor shredded a foam cleaning pig within the line and had to complete additional cleaning efforts to retrieve the foam components of the pig, (Figure 33)

Figure 33: Site 1 - Shredded Foam Cleaning Pig

6.2.2 Low Head Leakage Test

Low head leakage tests were completed as part of the CA process. As described in Section 3.5.2 the leakage tests were completed at pressures consistent with normal or worst case normal operating conditions. The water crossings were tested at 34 kPa (5 psi) less than the pressure of the adjacent piping to alleviate the potential for false positives.

Issues encountered and results of the low head leakage tests included:

- During the first test: the contractor encountered issues with leakage around their test flange and the test was completed without properly expelling air from the crossing. An apparent leakage of 312 L per hour was measured.
- A second test was completed after addressing the above concerns and an inspection within the tunnel was undertaken with the following results
  - Measured apparent leakage: 400 L per hour
  - No evidence of leakage within the tunnel.

Subsequent to the low head leakage tests on the Kildonan-Redwood Feeder Main crossing the City of Winnipeg's Water Services Branch completed two correlator inspections of the crossing with the intent of locating the leak. Unfortunately, the first test, completed on April 18, 2019 was inconclusive due to equipment error. Based on subsequent discussions with the Water Services staff the cause of the error was lack of connectivity between the wireless sensors and the base unit. Water Services completed a second correlator inspection on September 6, 2019 with longer lead cables on the pipe sensors and the base unit located in the middle of the Harry Lazarenko Bridge (see Figure 34). While no equipment errors were noted, the test was still inconclusive as no discrete leak location was identified. While the reason not detecting leak locations during the second correlator inspection is unknown, it is suspected that multiple small leaks may be present resulting in no single large detectable leak.

Figure 34: Site 1 - 2nd Correlator Inspection

Inspection report(s) for the leakage tests can be found in Appendix I.

6.3 Site 2 – Charleswood-Assiniboia Feeder Main

The contractor commenced with the preparation of Site 2 on March 11, 2019. Pipeline modifications included the following:

- Install launch wyes adjacent to existing valve chambers on both sides of the river.
- Pipeline cleaning.
- Drawing: D-15160

6.3.1 EM Inspection

The EM inspections occurred between March 26th and 28th, 2019 without major issues. Figure 35 depicts removal of the tool at Site 2, through the existing east valve chamber. PICA's inspection reports can be found in Appendix H.

In addition to PICA's inspection reports (Appendix H), overlay plots of PICA's inspection results on air photos/GIS plots have been included in Section 6.8. The general findings of the electromagnetic inspections are:

- Ten pipes were identified to have localized wall loss with a total of 111 discrete defect locations.
- The worst corrosion related defect identified had 16% remaining pipe wall, with a majority of the defects having between 40% and 65% remaining pipe wall.
- While there are defects spread along a majority of the pipe, a high percentage of those identified are located near the north bank of the river.

Figure 35: Site 2 - FRT Tool Removal

Issues encountered during the inline EM inspection and preparation work included:

- Changes in the feeder main pipe layout from that depicted on the record drawings and the contractor's shoring system resulted in the launch wyes being installed further from the existing chambers than show in the tendered design.
- Coating failures and corrosion of the existing feeder main piping resulted in the need for some patching and coating repairs. Figure 36 depicts steel patches welded to an existing flanged spool piece (bend) uncovered and reused on the south side of the crossing.

Figure 36: Site 2 - Repaired Spool Piece Prior to Coating

6.3.2 Low Head Leakage Test

Low head leakage tests were completed as part of the CA process. As described in Section 3.5.2 the leakage tests were completed at pressures consistent with normal or worst case normal operating conditions. The water crossings were tested at 34 kPa (5 psi) less than the pressure of the adjacent piping to alleviate the potential for false positives.

Issues encountered and results of the low head leakage tests included:

- The test was completed without pressurizing the adjacent sections of feeder main resulting in a measured apparent leakage of 327 L per hour.
- During the second test (after pressurizing the adjacent feeder main piping), the pressure within the crossing rose from 0 psi to 62 psi over the course of 1.5 hours. Conclusion, the mainline feeder main valves are leaking (this was already known) and there are no apparent leaks on the feeder main itself.

Inspection report(s) for the leakage tests can be found in Appendix I.

6.4 Site 3 – St. Vital Bridge Force Main

Preparation work for the external inspection at Site 3 – St. Vital Bridge Force Main commenced on October 15, 2018 with the stripping of external cladding and insulation, see Drawing 11820. Unfortunately, the contractor uncovered a leak in the force main between supports 5 and 6 north of Pier 8. The hole was approximately 10 mm in diameter, see Figure 37. At the same time, another leak was noticed at the next pipe joint to the north, located over the river. Due to the discovery of two additional leaks, in addition to a leak which had occurred earlier in the year, further preparation work was suspended until AECOM could complete additional investigation into the condition of the pipe.

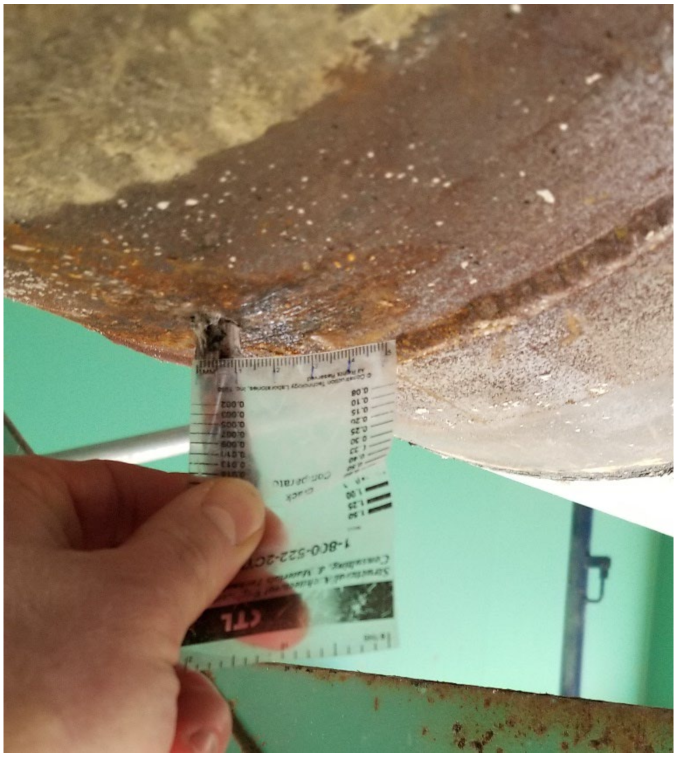


Figure 37: Hole in St. Vital Bridge Force Main

On October 26, 2018 AECOM completed an Ultrasonic Thickness (UT) wall thickness survey on the exposed sections of the force main. The survey found that the force main exhibited severe invert corrosion along the length of the pipe, with areas (outside of leak locations) registering wall thicknesses of less than 1 mm. The UT inspection report can be found in Appendix J.

As a result of the findings, it was concluded that the pipeline was exhibiting signs of severe invert corrosion and in need of immediate rehabilitation or replacement. Thus, the proposed external electromagnetic inspection was cancelled, and the WWD's Design and Construction Branch initiated a concept study for rehabilitation/replacement of the force main.

6.5 Site 4 – Newton Ave Force Main

Preparation works commenced at Site 4 – Newton Ave Force Main on November 1, 2018 with the Hawthorne Pump Station shutdown and upstream valve chamber reconfiguration, see Drawing 11821. Preparation works for this site included:

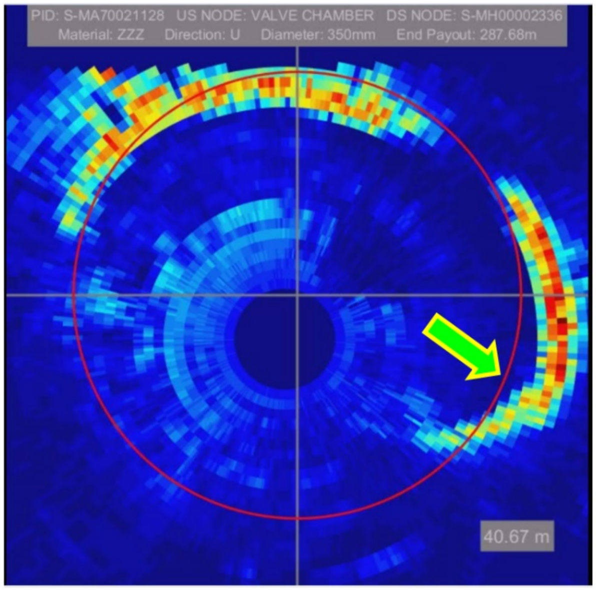
- Reconfiguration of the upstream valve chambers to direct flows to the parallel 350 mm steel force main. The valve chamber was reassembled in its original configuration upon completion of the work.
- Removal of the existing drop pipe in the downstream discharge chamber. The drop pipe was reinstalled upon completion of the work.
- Pipeline cleaning.
- Drawing: 11821

Upon commencement of the cleaning operations, the contractor noticed an approximate 30% deflection in the force main immediately adjacent to the west discharge chamber, see Figure 38. This resulted in a reduced cleaning effort with soft pigs only to ensure navigation through the heavily deflected pipe.

Figure 38: Site 4 - Deflection in HDPE Force Main

6.5.1 SONAR Inspection

The SONAR and CCTV inspection was completed on November 13, 2018 by AquaCoustic. Due to the known 30% deflection in the pipe, the SONAR tool was launched with a reduced centralizing setup to ensure passage through the pipe. This resulted in a reduction in the SONAR quality due to movement of the SONAR unit and increased assessment efforts. Nonetheless, results of the SONAR inspection indicate the pipe to be suffering from severe geometric defects along its length in the form of hinges, reverse curvature, and dents. Figure 39 depicts an example of the SONAR data with a sharp hinge point in the pipe wall. Figure 40 provides a summary of the pipe wall defects identified through the inspection.



SHARP OVAL

HINGE POINT

Figure 39: Site 4 - Sonar Inspection Results (Hinge Point)

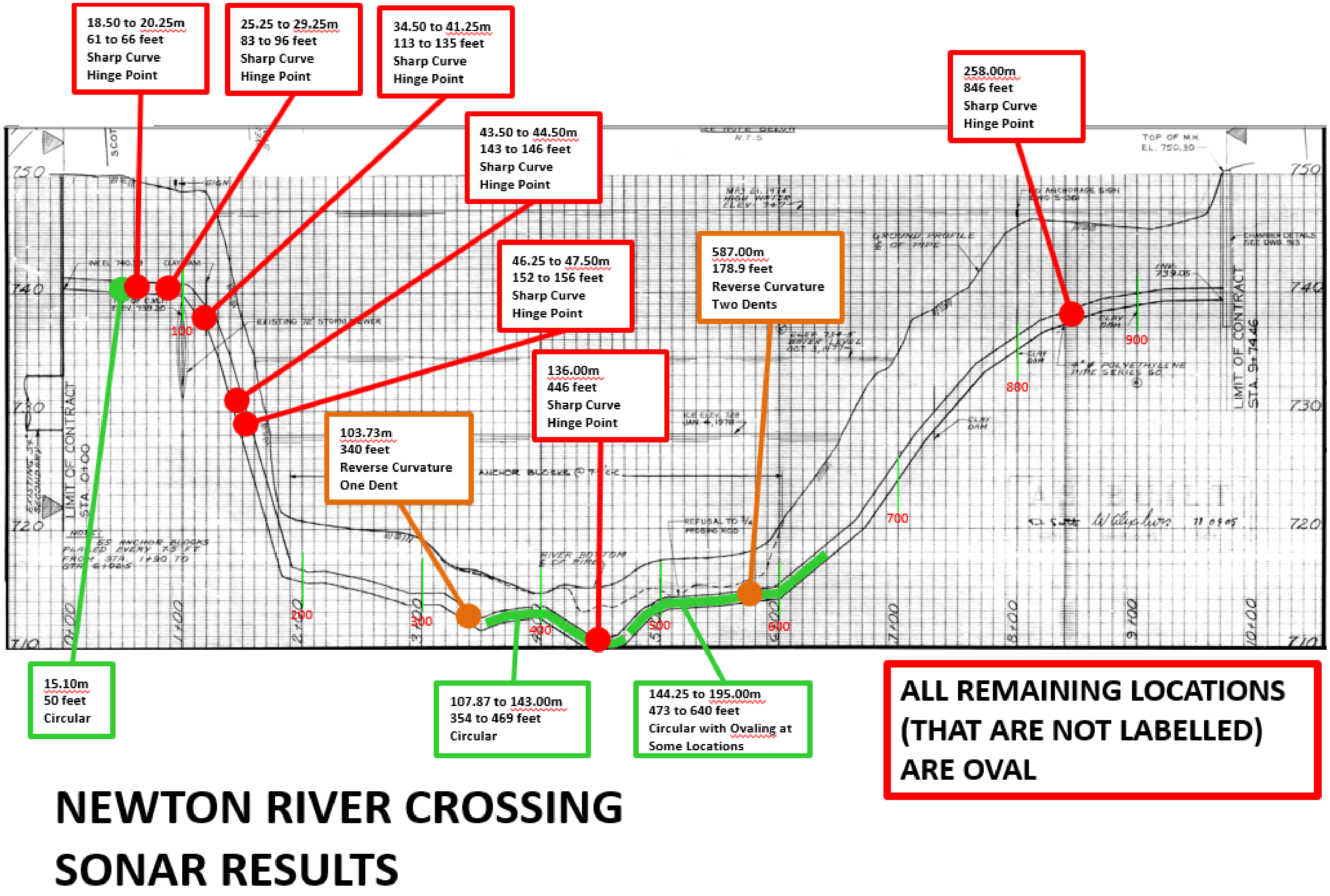


Figure 40: Site 4 - Sonar Results

Issues encountered during the SONAR inspection and preparation work included:

- During the pump station procedure review phase, the City advised the team of new operational procedures which prohibit the use of positive gates for holding back sewage. Thus, completion of the Hawthorne Pump Station shutdown was delayed while the contractor procured an 1800 mm inflatable blocking plug.
- During the first Hawthorne Pump Station shutdown, the contractor did not completely disassemble the piping within the upstream valve chamber and upon reassembly could not install complete gaskets on one of the flange connections resulting in a minor leak from the modified piping arrangement.
- During cleaning, the force main was found to be deflected approximately 30% near the west end of the crossing, see discussion above.
- A leak was found in the force main crossing and ultimately repaired (see discussion below).
- Once the force main was repaired, the subsequent leakage test found the force main had frozen where shallow soil covers were present on the west bank. The City engaged Uni-Jet Industrial Pipe Services (Uni-Jet) to thaw the pipe.
- Due to all of the above issues; the final Hawthorne Pumping Station shutdown and reassembly of the valve chamber piping was pushed back to late March 2019, during the spring thaw. This presented an inflow condition in excess of that accounted for in AECOM’s original hydraulic analysis (see Section 3.4.2.2). To facilitate the pump station shutdown under these conditions, AECOM undertook additional hydraulic analysis and full time monitoring of the accumulated storage in the system to evaluate the remaining shutdown window in real time.

6.5.2 Low Head Leakage Test

Low head leakage tests were completed as part of the CA process. The low head leakage test was completed on the crossing at 49 kPa (7 psi), premised on the normal operating condition of the pipe, which roughly equated to static water level at grade at the valve chambers. The test pressures were intentionally kept as low as practically possible due to the sensitivity of the pipe to a heightened stress condition. To prevent over pressurization the test was carried out using stand pipes (rigid suction hose) and a water tank which was used to fill the line and maintain the static water level within the crossing (see Figure 41).

The first leakage test measured an apparent leak of approximately 800 L per hour. A subsequent test confirmed a leakage rates between 830 and 885 L per hour. Based on the level at which water within the crossing stabilized after the tests (566 mm above the pipe invert within the east valve chamber), the leak was believed to be located within the banks of the river. The City engaged Uni-Jet to complete a CCTV of the upper portions of the siphon, with particular attention being paid to the HDPE discharge chamber connection and the vertical/horizontal bends on the west side of the crossing.

Figure 41: Site 4 - Low Head Leakage Test

The CCTV inspection found that the HDPE pipe was split immediately adjacent to the HDPE/Cast iron connection to the west discharge chamber (Figure 42). This was likely attributable to the 30% deflection found immediately downstream (Figure 38). The City ultimately installed an internal ambient cure CIPP spot repair at the split location and the force main passed the subsequent leakage test.

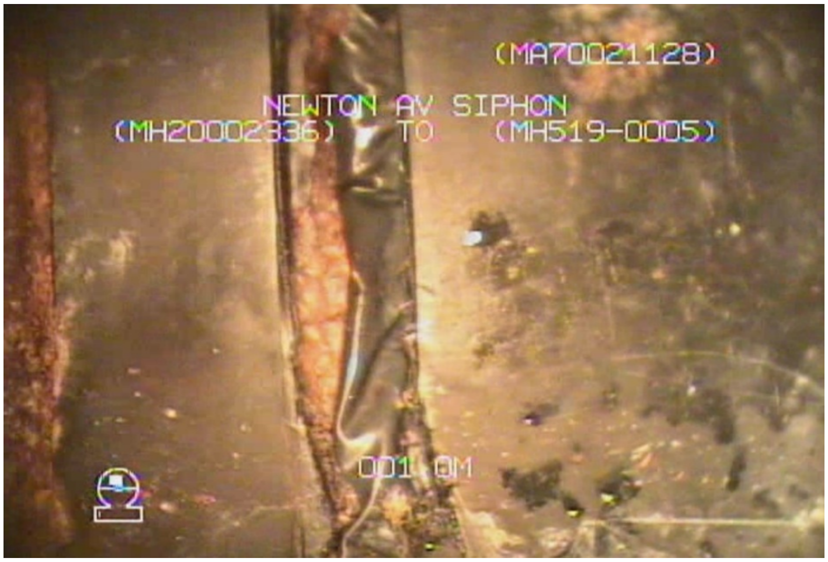


Figure 42: Site 4 - Split HDPE Force Main

Issues encountered and results of the low head leakage tests included:

- The nature of the pressures and desire to prevent overpressures required that the tests were completed with a static water head. Due to the size of the apparent leaks, the contractor's make up water measuring process was not particularly precise.
- The initial leakage tests found an apparent leak. The leak was eventually repaired, and a successful leakage test was completed.
- The pipe was found to be frozen during the first leakage test after the pipe repair. The test on the upstream portion of the force main was successful. The pipe was thawed by the City and a full successful leakage test completed.

Inspection report(s) for the leakage tests can be found in Appendix I.

6.6 Site 5 – Heritage Park Force Main

6.6.1 Material Sampling

Sampling of the Heritage Park Force Main was completed by J-Con on November 22, 2018. An approximately 1 m long sample was obtained from the force main (Figure 43). A portion of the sample collected was sent to PSI labs in Colorado for material testing.

Figure 43: Site 5 - PVC Force Main Sample Retrieval

6.6.2 Low Head Leakage Test

Low head leakage tests were completed as part of the CA process. The test pressure was premised on the (unlikely) scenario where all three pumps are running simultaneously. While normal operating pressures were reported by the City to be approximately 13 psi under a single pump operating scenario, given the pipe material's original operating pressure rating of 235 psi, the test pressure was still considered very conservative.

An apparent leak of 17.5 L per hour was measured during the first leakage test, in excess of what would be allowable under current PVC pipe installation practices. A subsequent leakage test determined that a majority of the apparent leakage was due to bypass valves within the station and determined that a more accurate apparent leakage was <1 L per hour.

Inspection report(s) for the leakage tests can be found in Appendix I.

6.7 Site 6 – FGSV Feeder Main

6.7.1 Sahara Inspection

Preparation works for the Sahara inspection at Site 6 – FGSV Feeder Main commenced on October 19, 2018. Modifications to the existing system included:

- Installation of a 100 mm tapping sleeve on the existing feeder main within the west valve chamber for the purpose of launching the Sahara tool. Configuration of the existing valve chambers required launching the Sahara tool from the west valve chamber, which did not have an air release port on the crossing side of the butterfly valve. The tapping sleeve was installed complete with a reduced port 100 mm stainless steel ball valve.
- Replacement of the existing 200 mm drain valve on the feeder main within the east valve chamber and installation of a temporary connection to the existing 200 mm feeder main drain line, connected to the Branch II Aqueduct drain chamber. The connection was removed after completion of the work and a blind flange installed on the new drain valve.
- A hole was cored in the existing west valve chamber roof to facilitate setup of the Sahara tool launching apparatus.
- Installed a temporary screen within the Branch II Aqueduct drain chamber to facilitate dichlorination of the induced flushing water (Figure 27).

The inspection occurred overnight between October 23rd and 24th, 2018. Figure 44 shows the inspection setup and tool launching apparatus.

Figure 44: Site 6 - Sahara Inspection

Pure's inspection report can be found in Appendix K. Results of the Sahara inspection were:

- The original cast iron pipe was not manufactured with an interior cementitious liner.
- While some corrosion and tuberculation are present, tuberculation is generally minimal, consistent with our experience with uncoated ferrous metal pipelines in the Winnipeg distribution system. Typical internal condition is depicted in Figure 48.
- Debris levels in the siphon were minimal.

Issues encountered during the Sahara inspection and preparation work included:

- AECOM specified a 150 mm diameter hole to be cored in the roof of the existing valve chamber to facilitate the Sahara tool launching apparatus. Unfortunately, Pure required a 275 mm diameter opening, which the contractor was able to core the afternoon prior to the inspection.

Figure 45: Site 6 - CCTV Inspection Image

6.7.2 Low Head Leakage Test

Low head leakage tests were completed as part of the CA process. As described in Section 3.5.2 the leakage tests were completed at pressures consistent with normal or worst case normal operating conditions. The water crossings were tested at 34 kPa (5 psi) less than the pressure of the adjacent piping to alleviate the potential for false positives.

The leakage test passed without any issues.

Inspection report(s) for the leakage tests can be found in Appendix I.

6.8 EM Inspection Results

The EM inspection for Site 1 and 2 are provided in Figure 46 and Figure 47 below:

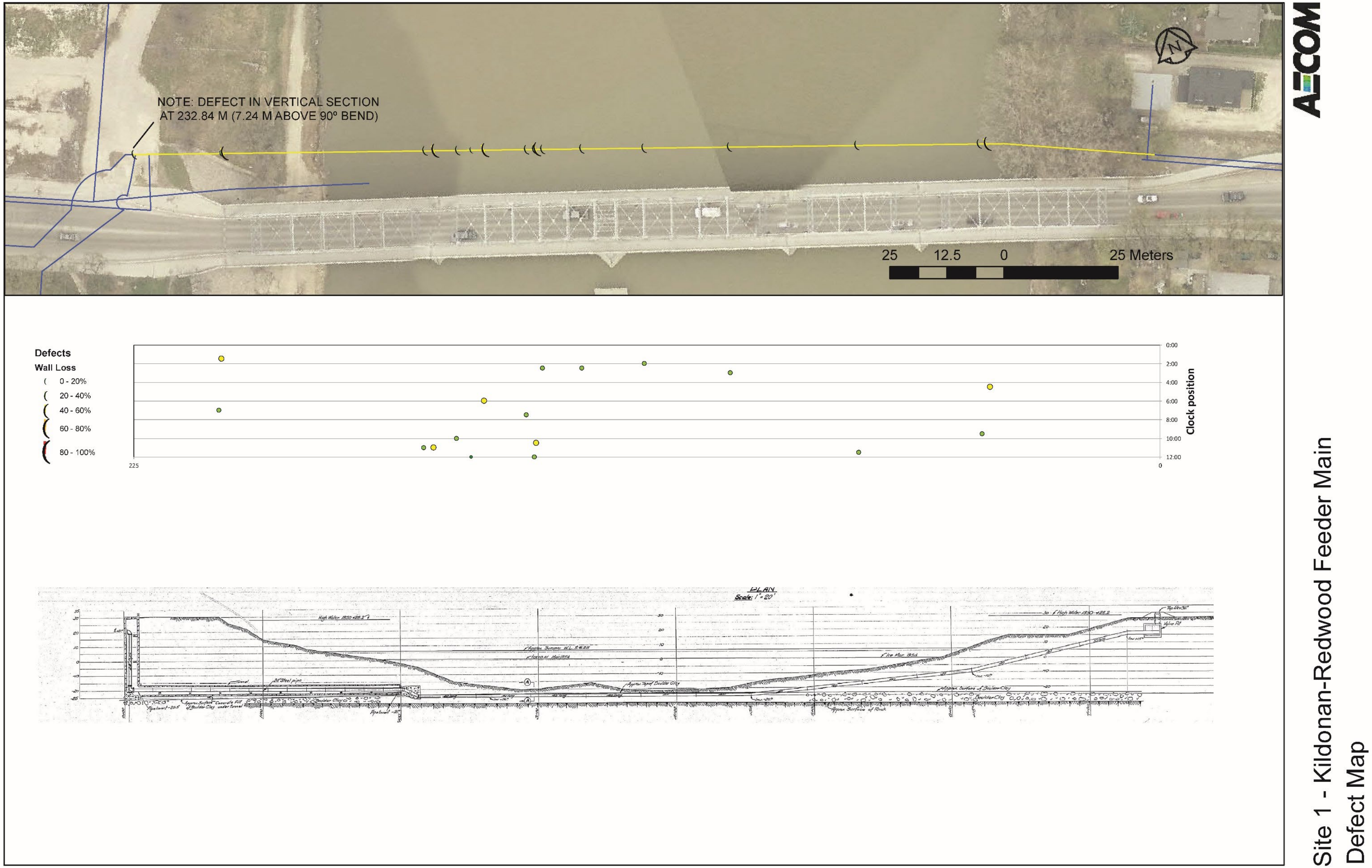


Figure 46: EM Inspection Results – Site 1 – Kildonan-Redwood Feeder Main

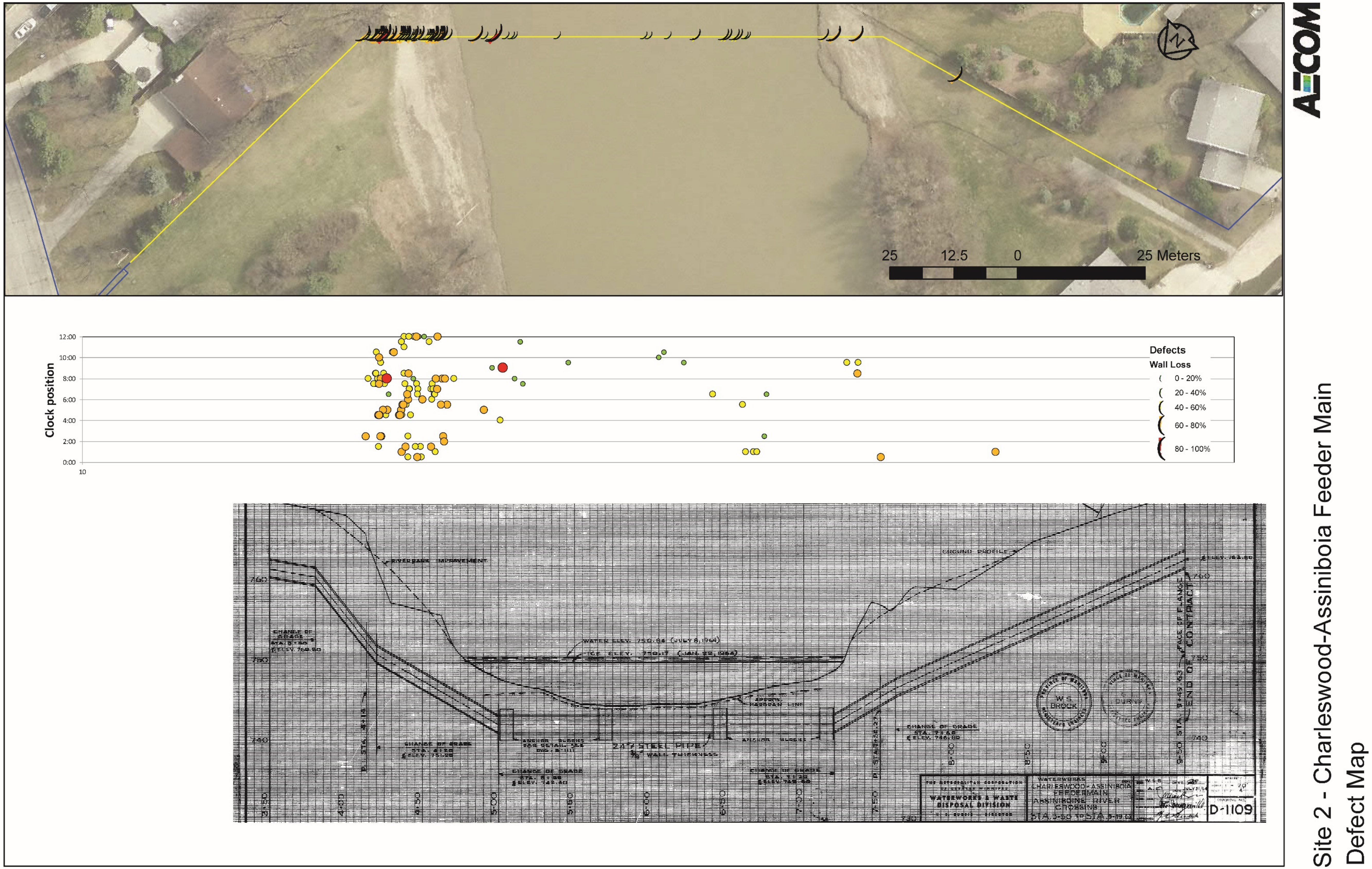


Figure 47: EM Inspection Results – Site 2 – Charleswood-Assiniboia Feeder Main

7. Pipeline Condition Assessment

The river 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. Except where material testing has been completed, 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 8 lists the crossings included in the program along with known dimensional data.

Table 8: Crossing Summary

Site	Crossing Location	Installation Condition	Nominal Diameter (mm)	Material Type	Installation Year	Outside Diameter (mm)	Wall Thickness (mm)
1	Kildonan-Redwood Feeder Main	Tunneled/Buried	600	Steel	1955	609.60 (24”)	12.70 (1/2”) 7.94 (5/16”)
2	Charleswood-Assiniboia Feeder Main	Buried	600	Steel	1965	609.60 (24”)	9.53 (3/8”)
3	St. Vital Bridge Force Main	Aerial	500	Steel	1988	508.00 (20”)	9.53 (3/8”)
4	Newton Avenue Force Main	Buried	350	HDPE	1979	355.60 (14”)	14.1
5	Heritage Park Force Main	Buried	250	PVC	1989	281.94	15.67 (DR18)
6	Fort Garry – St. Vital Feeder Main	Tunneled	600	CI	1958	655.32 (25.8”)	Unknown

7.1 Corrosion of Ferrous Metal Pipelines

7.1.1 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 (Figure 48); 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 pipe is also, typically, a very localized phenomenon classified as pitting corrosion (Figure 49) which will ultimately compromise hydrostatic integrity of the pipe but not necessarily have a significant effect on the overall ring stiffness of the pipe or 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 eventual collapse of the pipe due to loss of soil support. The critical pitting depths and the estimated remaining service life listed in Table 9 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 48: Localized Coating Damage on the Charleswood-Assiniboia Feeder Main

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 corrosion by mass[21]. 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 to account for the loss of structural capacity.

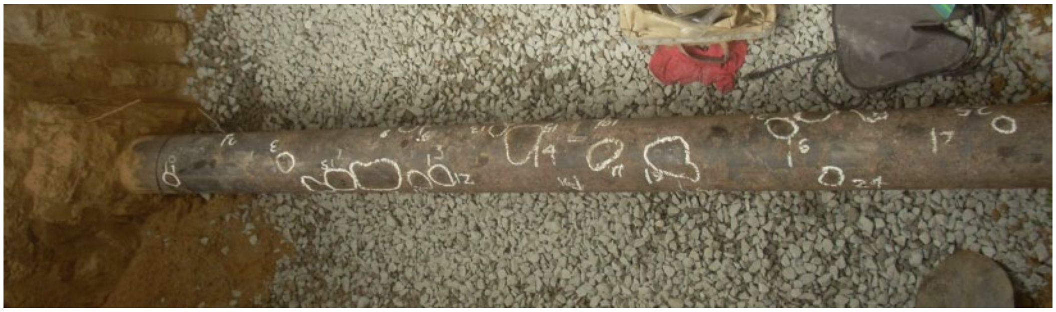


Figure 49: Example of Localized Pit Locations on a Steel Pipeline

7.1.2 Assessment of Measured Corrosion Defects

Corrosion defects were measured on three pipelines as part of the current inventory, Sites 1 and 2 were inspected by PICA using their inline RFT tool and Site 3 was inspected by AECOM using UT inspection. The electromagnetic inspection data collected by PICA was submitted to AECOM in the form of an analysis report (Appendix H). The results from the inspection have been summarised in Table 9.

---

[21] *Handbook of Steel Drainage & Highway Construction Products, First Canadian Edition, 1984.*

Table 9: Ferrous Metal Pipelines - Corrosion Summary

Site	Crossing Location	Nominal Diameter (mm)	Inspection Year	Original Pipe Wall Thickness	Deepest Corrosion Defect (% Remaining Wall)	Equivalent Pitting Depth (mm)
1	Kildonan-Redwood Feeder Main	600	2019	7.94 12.70	40%*	4.76*
2	Charleswood-Assiniboia Feeder Main	600	2019	9.53	16%	8.00*
3	St. Vital Bridge Force Main	500	2019	9.53	0%	9.53
***Measured in 7.94 mm section of pipe.**						

It is important to note that through wall corrosion was found on the Charleswood-Assiniboia on one of the field welded spool pieces adjacent to the existing valve chamber off Berkley Street. The extensive corrosion on this spool piece is believed to be due to coating damage from the original installation and is not reflective of the condition of the majority of the crossing.

7.1.3 St. Vital Bridge Force Main

While originally slated for an external electromagnetic inspection by PICA, the St. Vital Bridge Force Main was inspected by AECOM using our UT wall thickness measurement instrument. Inspection at exposed areas of the pipe at both ends of the force main found the pipe was exhibiting severe invert loss along the length of the force main. Figure 50 depicts the pipe after UT measurements near the leak on the south river bank. The dark spots are the locations where UT measurements were taken. The UT measurements found wall thicknesses as low as 1 mm along the invert of the pipe. AECOM's inspection report can be found in Appendix J.

Figure 50: Site 3 - UT Inspection Locations

Exterior inspection of the pipe found minimal corrosion, compared to that measured by the UT tool (Figure 51). Thus, the cause of the wall loss is believed to be a combination of corrosion and abrasion on the interior of the pipe (commonly referred to as erosion corrosion). Based on a review of record drawings and the original specifications, the pipe was installed without any interior coating. Typically, the corrosion process results in the formation of corrosion product which acts to reduce the corrosion rates. However, the configuration of the force main where it crosses the Red River will result in the transported sediments coming out of suspension causing the sediment to be deposited in the invert between pump runs. When the pumps start up again this sediment will be transported along the invert acting to abrade the formed corrosion product. This can greatly accelerate the corrosion process along this localized area of the pipe. Based on the condition of the pipeline the pipe was recommended for immediate rehabilitation or replacement.

Structural checks completed on the force main have assumed a wall thickness of 1 mm as per AECOM's UT measurements.

Figure 51: Site 3 - Exterior Pipe Corrosion

The City installed a temporary bypass for the force main in 2019 and after additional leaks were found, put it into service. Further, the City is currently in the process of developing recommendations for rehabilitation or replacement of the crossing, consistent with AECOM's recommendations for the crossing.

7.1.4 Fort Garry-St. Vital Feeder Main

As described in Section 3, the FGSV Feeder Main was installed with the 1650mm Branch II Aqueduct within a tunnel and fully encased in concrete. The concrete encasement elevates the pH of the outer pipe wall to a very high value which creates a very reliable, very low-corrosive environment for the cast iron pipe. Thus, external face corrosion is anticipated to be minimal to non-existent on the crossing.

Internal face corrosion is typically generalized and not a governing failure mode for ferrous metal pipelines in Winnipeg. The Sahara inspection of the FGSV Feeder Main has indicated that the pipe is in good condition with typical levels of interior corrosion and tuberculation. Based on the relatively benign environment in which the pipe is located, we do not anticipate any issues with this crossing in the near future.

7.1.5 Remaining Pipeline Life

A critical pitting depth was calculated for each pipe with relevant data for same and used to estimate the remaining service life based on the pipe's structural limit state (see Section 7.6). For the steel pipes in the current inventory, the critical pitting depth was perforation of the pipe wall resulting in leakage to the environment. While corrosion rates are not linear throughout the lifespan of an asset, factors that contribute to changes in corrosion rates (e.g. build up of corrosion product) can be difficult to account for with certainty and a linear assumption is the most conservative assumption. Where corrosion pitting can be physically measured or inferred from EM inspection, linear corrosion rates are typically assumed in absence of known significant changes in the pipe's operating conditions. The remaining service life of the pipeline was then calculated using Equation 1.

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

Equation 1: Remaining Service life

As active leaks were found on the St. Vital Bridge Force Main and the remaining wall thickness minimal along a majority of its length, the remaining lifespan is considered to be zero. The Kildonan-Redwood and Charleswood Feed Mains both have a remaining life spans of 43 and 10 years respectively based on the corrosion pitting determined by PICA. Note, this does not take into account the apparent leak found on the Kildonan-Redwood Feeder Main. The results of the analysis is presented in Table 10.

As corrosion on the FGSV Feeder Main is considered to be minimal as remaining service life due to corrosion has been estimated to be >50 years.

Table 10: Ferrous Metal Pipelines - Remaining Service Life

Site	Crossing Location	Nominal Diameter (mm)	Inspection Year	Original 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)
1	Kildonan-Redwood Feeder Main	600	2019	7.94 / 12.7	40%	4.76	0.0744	7.94	43**
2	Charleswood-Assiniboia Feeder Main	600	2019	9.53	16%	8.00	0.1482	9.53	10
3	St. Vital Bridge Force Main	500	2018	9.53	0%	9.53	0.3175	9.53	0
6	FGSV Feeder Main	600	2018	unknown	N/A*	N/A*	N/A*	N/A*	>50*

* No direct corrosion measurements made. **50 years of remaining service life estimated**.

** The apparent leak on the crossing makes the actual remaining service life 0 years.

7.2 Thermoplastic Pipe Deterioration

Two force mains in this program, constructed from thermoplastic pipes, were evaluated using material testing in combination with SONAR and structural evaluations to determine their current structural state and remaining lifespan. These are:

- Site 4 – Newton Ave Force Main (HDPE)
- Site 5 – Heritage Park Force Main (PVC)

7.2.1 Site 4 – Newton Ave Force Main

A sample of the HDPE (north) Newton Ave Force Main was collected during the Phase One Program and tested by NSF Canada Labs in Aurora, Ontario in 2016/2017. AECOM provided a follow up technical memorandum to the Phase One Program Report[22] to discuss the results and their significance with respect to the long term performance of the crossings tested, attached in Appendix L. While the testing indicated cell classifications of a PE3XXX series resin in accordance with ASTM D3350, an increase from a PE2XXX series resin that was assumed in the original assessment, the PENT tests demonstrated a very low resistance to SCG. PENT tests, also known as a notch tensile test, are used to assess the susceptibility of HDPE materials to SCG in accordance with ASTM F1473. Failure of the Newton Ave Force Main samples were at 0.21 and 1.14 hours which is substantially below the 10 hour requirement for the identified HDPE cell classification.

The life expectancy of a HDPE pipe is controlled by material, environment, and loading factors. The force main material was determined to be a true high density PE as opposed to low or medium density material which is a positive aspect of the testing. The testing yielded mechanical properties of PE3354 in accordance with ASTM D3350. These resin classifications are typical of many pre-1980 resins which typically exhibit much poorer resistance to SCG than common pressure pipe resins manufactured post 1980, see Figure 52. The newer (post 1980) PE3408 resins are known to have a high enough resistance to SCG such that SCG does not control design life under the vast majority of circumstances.

---

[22] *High Risk River Crossings Condition Assessment Report – Sewer Crossings, September 2016, AECOM.*

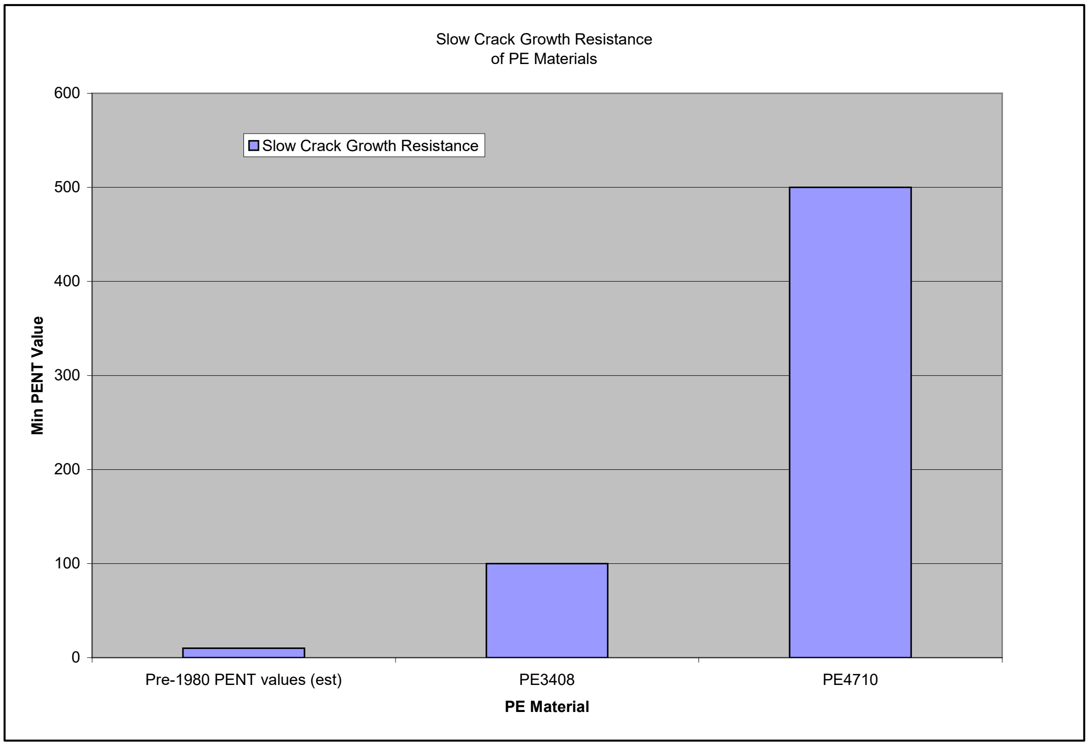


Figure 52: Typical Slow Crack Growth Resistance by Resin Type

The primary environmental exposure concern for HDPE pipelines is exposure to extreme oxidizing agents (e.g. chlorine) that can break down inferior HDPE resins. The exposure conditions for the Newton Force Main are generally considered to be relatively benign to HDPE, as municipal waste streams are not commonly rich in HDPE oxidizing agents.

Polyethylene (PE) materials have three modes of failure, depending on the stress level and chemical resistance evident in the material as shown in Figure 53:

- Stage I failures are ductile in nature, and only occur at very high stress levels.
- Stage II failures, however, are brittle types of fractures and can occur at moderate to low stress levels. Stage II failures are associated with SCG and the resistance of the pipe to SCG and can usually be assessed in a PENT test. PENT test values lower than 10 hours are indicative of the material with a much higher risk factor to incur brittle fractures over time at increasing lower stress levels.
- Stage III failures occur as a result of chemical degradation of the material, and the steep curve associated with Stage III failures indicates that the material no longer has the capacity to withstand any load at all.

While based on the resin classification and the exposure condition for the force main, they are very unlikely to experience Stage III failure modes. The force main is, however, at risk of both Stage I and Stage II failures. The extreme deformation found throughout the SONAR inspection results in high imparted wall stresses which may result in ductile failures (Stage 1). Based on the very low PENT values in testing, the pipe materials will become increasing less ductile over time and will continue to have increasing risk for brittle failures.

Based on the above Newton Force Main has reached the end of its service life and needs to be replaced in the near term. It is recommended that regular leakage testing be undertaken to monitor for inadvertent leaks to the environment.

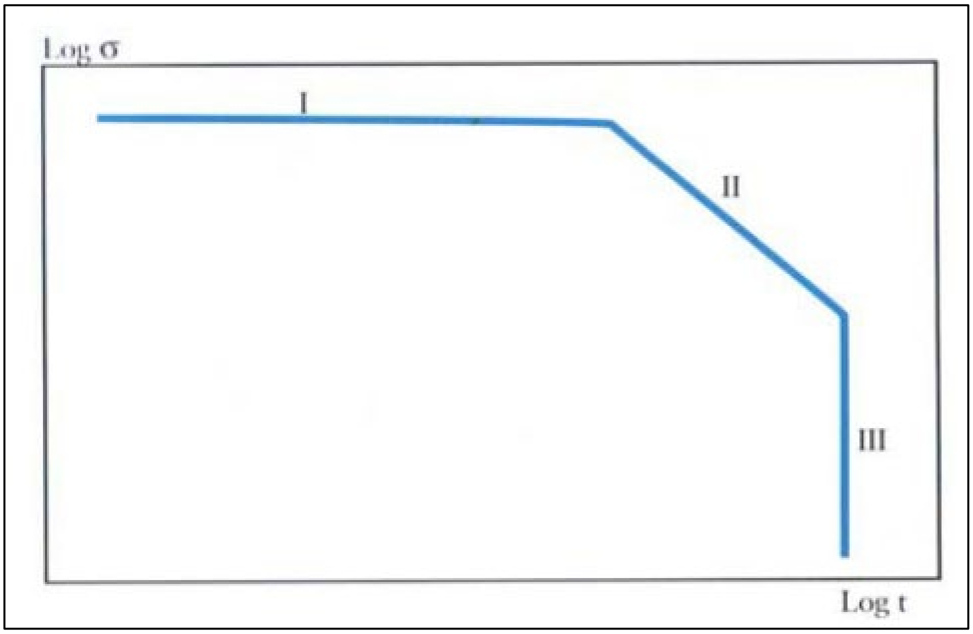


Figure 53: Relationship between Stress Level and Time to Failure for HDPE

7.2.2 Site 5 – Heritage Park Force Main

A sample of the Heritage Park Force Main was tested by PSI Labs in Longmont, Colorado in 2019. PSI's testing report can be found in Appendix L and the test results are presented in Table 11.

Table 11: PSI Lab Test Results

Properties	Test Results
IZOD Impact Resistance (ft-lb/in)	1.27
Tensile Strength @ Yield (psi)	7,168
Tensile Modulus (psi)	446,070
Heat Reversion	FAIL
Acetone Immersion	<1% / No Reaction
Ash Content (%)	6.04%

Material property testing confirms the cell classification is consistent with the requirements of AWWA C900-81 which stipulated the use of Class 12454 compounds as defined in ASTM D1748-81.

The purpose of the heat reversion test (ASTM F1057) is to test the quality of the original extrusion and/or to test for the contamination of the pipe wall through exposure to volatile solvents. The test procedure brings the pipe above the melt temperature, which activates the volatile solvents and splits the pipe. As shown in Figure 54, the sample from the Heritage Park Force Main shows extensive evidence of both splitting and burning. Burning is another sign of contamination of the PVC matrix and the exterior pipe burning evidenced in the test samples is similarly located where the worst of the interior splitting is located.

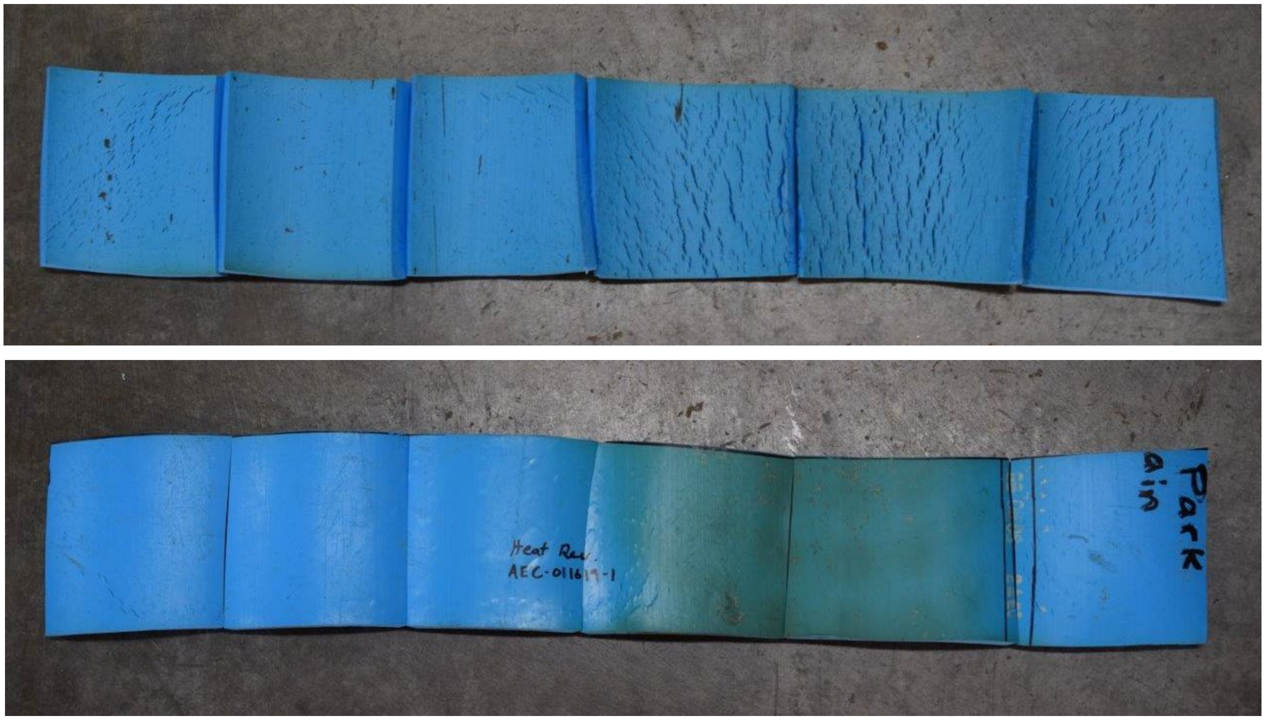


Figure 54: Site 5 - Heat Reversion Test Photos

For new pipe, heat reversion tests are utilized to test the quality of the extrusion process. Failure of a heat reversion test indicates that volatile solvents have been trapped within the pipe wall during the extrusion process. For CA purposes it can also be an indication of exposure to volatile solvents, such as gasoline, during the lifetime of the pipe. Where pipelines are not exposed to volatile solvents (e.g. water mains) failure of a heat reversion test is indicative of poor manufacturing. In WWS applications, the test results could be result of either poor manufacturing or exposure. However, as the level of exposure required to exhibit failure evidenced would need to be high and the level of dilution anticipated, should solvents have entered the waste stream, would be low, the test results are likely the result of poor manufacturing.

Previous CA programs completed by AECOM within North America have found that despite exhibiting favourable short term material properties, failure of heat revision tests in PVC pressure mains often correlate to an increase in failures under high stress operating conditions. For example, where water mains are operating near their rated internal operating pressure for long periods of time, they have a much higher failure rate than those exhibiting similarly poor extrusion quality but operating at lower prolonged stresses. Thus, given the normal operating pressure within the pipeline is around 13 psi and the external loading conditions on the pipeline are relatively benign we do not anticipate long term issues with failure on this force main.

The material testing indicated the mechanical properties of the pipe were more than adequate for the applied loading at the site and resulted in very low wall stresses. As the quality of the original extrusion was deemed to be poor from the failed heat reversion test, a more robust test was carried out using the Differential scanning calorimetry (DSC) method. This is an ISO test that provides more definitive data on original extruded quality (ISO 18373-1) throughout the wall section. Based on the DSC results, the original extrusion quality of the pipe is poor; however, the pipe is not deemed to have any active deterioration processes present due to the very low wall stresses that are present in its current operating mode.

7.3 Low Head Leakage Tests

As discussed in Section 2.2 low head leakage tests were completed on all pipelines in the program. All pipelines eventually passed the leakage tests within acceptable tolerances except for the Kildonan-Redwood Feeder Main.

A definitive leak location on the Kildonan Redwood Feeder Main could not be identified or confirmed by the City's correlator system. While only a theory at this time, it is believed that there may be multiple leaks at the pipe's flanged and welded joints. This is based on:

- An apparent leak of 300 to 400 L per hour was measured during the leakage tests and should have been detectable by the City's correlator system if coming from a single source. All indications from Water Services staff during the second inspection were that the equipment was working correctly, and the leak was confirmed to be present during the second correlator inspection.
- The correlator system listens for the noise generated by a leak. Thus, a series of small leaks may not be registered by the equipment (i.e. may be considered background noise) and thus, no definitive leak location identified.
- PICA inspection point to corrosion related defects in the pipe barrels that are > 40% RW. However, EM inspection tools typically cannot see corrosion in flanges and/or at weld locations as the discontinuity and change in metal signal will mask the signal. Thus, while the pipe barrels themselves appear to be in satisfactory condition, however, leaks due to corrosion or mechanical failure of fasteners could be present at flanged joints.
- Based on record information, the pipes and joints for the buried sections of the crossing were coated prior to installation. Coatings within the tunnel section appear to be in good condition within the tunnel but the joints were noted to have not been coated after assembly. Figure 55 depicts the typical condition of the joints within the tunnel section. The pipe exposed on the Charleswood-Assiniboia Feeder Main showed extensive corrosion at field assembled flange joint and adjacent field welded joint where poor coating and stainless steel bolts were used (Figure 56). Based on the records for the Kildonan-Redwood Feeder Main it appears that stainless steel bolts were also used which could cause localized corrosion at the flanges similar to what was seen at Site 2.

Other means of confirming the leak location on the crossing include the use of inline leak detection technologies, such as Pure's Sahara or smart ball technologies. While insufficient for proper advanced CA on their own, their ability to accurately pinpoint leaks is extremely useful when repairs are required. However, given that there is a high probability of the leaks being located beneath the river in locations where excavations and traditional repairs are not feasible, the decision has been made to rehabilitate the crossing CIPP technology. Ultimately, this demonstrates the need to complete both electromagnetic inspection and leak detection as part of the advanced CA program.

Figure 55: Site 1 - Bolts and Flange Within the Tunnel

Figure 56: Site 2 - Flange and Fitting Corrosion

The results of the low head leakage tests are presented in Table 12. All of the crossings passed the low head leakage test except for the Kildonan Redwood Feeder Main as discussed above and the St. Vital Bridge Force Main where none was completed.

Table 12: Low Head Leakage Test Results

Site	Notes	Pass/Fail
Site 1	Apparent Leak of 300 to 400 L/Hr	Fail
Site 2		Pass
Site 3	Not Completed	N/A
Site 4		Pass
Site 5		Pass
Site 6		Pass

7.4 Buoyancy

Buoyancy 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. A buoyancy FS check is prudent to understand any inherent short term or long term vulnerability with respect to flotation. Each crossing was conservatively assessed with 10% air entrainment of the cross sectional area for the water crossings and 25% for the sewer crossings due to the increased potential for air entrainment. Table 13 lists the state that each siphon was assessed with and the FS against floatation for each siphon. A generally accepted FS against floatation is 2.0[23], for pipes intended for permanent embedment.

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

- Mass of pipe contents, less appropriate air allowance.
- Buoyant Soil weight (prism), where applicable.
- Mass of pipe.
- Buoyant mass of Concrete Anchor blocks.

There are several methods for assessing pipeline buoyancy all with varying methods for calculating the beneficial effects of the embedment and backfill soil in resisting the buoyant forces. Accounting for soil loads can range from the pure soil load above the pipe, also know as the prism load (Figure 57) to larger triangular and parabolic soil wedges which account for soil beyond the width of the pipe. AECOM's assessment includes a review of all three methods, but as the prism load method is the most conservative it governs the assessment for all pipelines. Given the variable and potentially evolving conditions that river crossing pipelines are exposed to, the use of a conservative design approach is recommended without additional investigation into the current embedment conditions. As shown in Table 13, despite limiting the contribution of the back soil to that of a prism load, our analysis indicates that all of the water crossings are operating at a FS greater than 2.0 against buoyancy.

---

[23] *Buried Pipe Design, Moser, Folkman,, 2008, McGraw-Hill.*

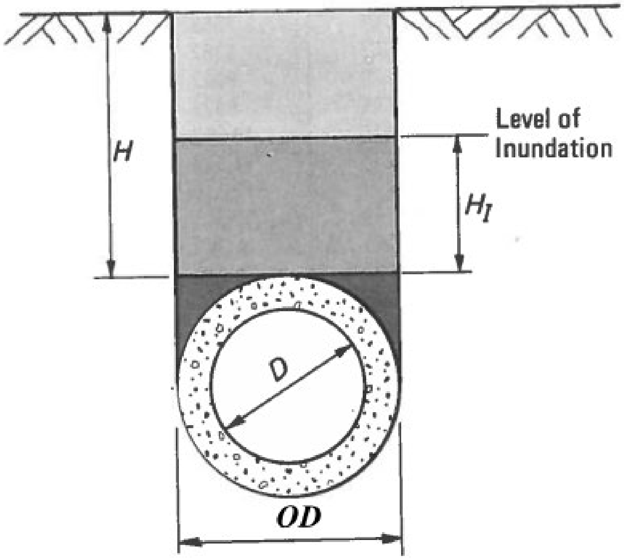


Figure 57: Buoyancy Check - Soil Prism Load

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.

Based on the results none of the pipelines are at risk of buoyancy during normal operation.

Table 13: Buoyancy Assessment Results

Site	Crossing Location	Nominal Diameter (mm)	Buried	Anchor Blocks	Operational Condition	Factor of Safety
1	Kildonan-Redwood Feeder Main	600	Yes	No	Full, 10% Air Entrainment	3.09
2	Charleswood-Assiniboia Feeder Main	600	Yes	Yes	Full, 10% Air Entrainment	3.14
3	St. Vital Bridge Force Main	500	No	N/A	Aerial	N/A
4	Newton Avenue Force Main	350	Yes	No	Full, 25% Air Entrainment	2.33
5	Heritage Park Force Main	250	Yes	No	Full, 25% Air Entrainment	8.24
6	Fort Garry – St. Vital Feeder Main	600	No	N/A	Tunneled	N/A

7.5 Pipeline Embedment and Loading

7.5.1 External Loading

While river crossings are subject to some combination of external and internal loading conditions like 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 include soil loads, live loads, and external hydrostatic pressure. For illustrative purposes, Figure 58 shows the range of external loading conditions imparted on a typical river crossing site. This would include full Flood Protection Level (FPL) river conditions.

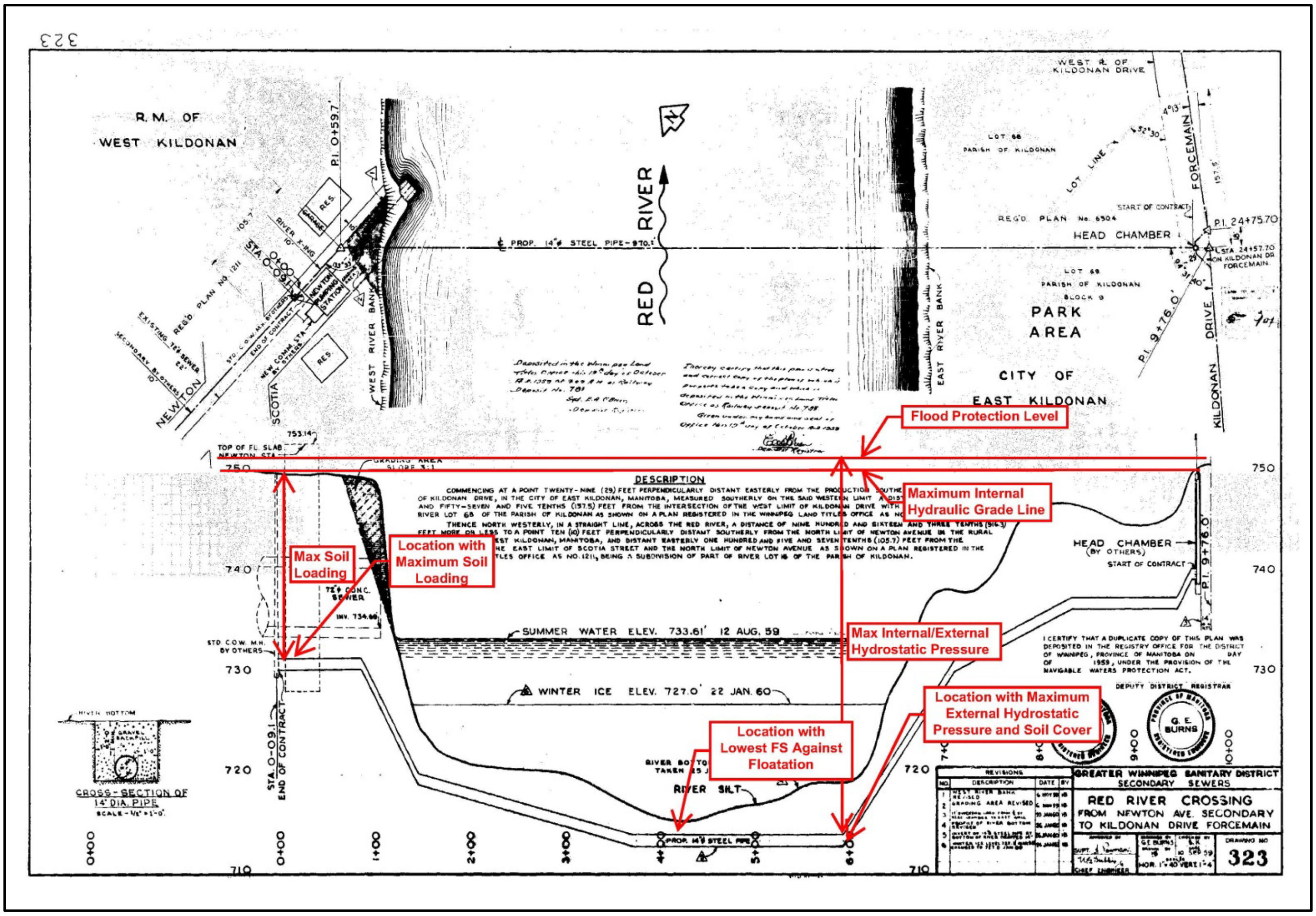


Figure 58: External and Internal Loading Conditions (Example)

The pipelines included in this program are constructed of steel, cast iron, HDPE, and PVC pipe materials. Steel, HDPE, and PVC are considered flexible piping as it can deflect (typically defined as 2% ring deflection) without damage. Cast iron pipe is, however, considered a rigid pipe as it cannot deflect to this degree without damage. The stiffness of the pipe ring is an important consideration when determining the applicable external soil loading.

The long term external soil load on flexible pipes is limited to the column of soil directly over the pipe, known as the prism load, see Figure 59. This limit is assumed, because as the pipe deflects, soil arching above the pipe is induced, which limits magnitude of the soil load which can act on the pipe. The long term external soil loading on rigid pipes is larger than that applied to flexible pipes as they see additional loads due to settlement of the adjacent embedment soils relative to the pipelines themselves. Modern standard practice is to estimate external loads on rigid pipelines positive projection methods, which are considered to be a conservative (worst case) approach as they are independent of trench geometry, which can be variable in a river crossing. Heger Positive Projection load methods, which use a prism loading and a vertical arching factor to account for settlement loads are typically used.

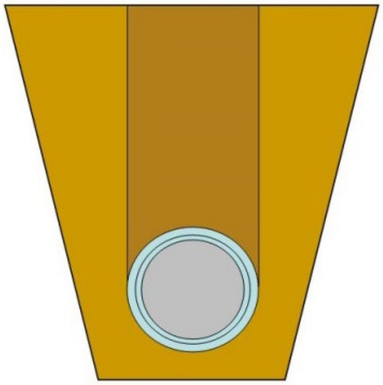


Figure 59: Soil Prism Load

Live loads have also been considered in our analysis and calculated using an AASHTO HS 20 design vehicle in accordance with AASHTO LRFD, 7th Ed. Live loads have been included where they can be reasonably assumed to act on the pipe. For river crossings, this is typically on the edges of the banks where accessible by vehicles. Assessment of the pipe beneath the river or in other places practically inaccessible by vehicles do not consider live loads.

7.5.2 Embedment Conditions

The existing river crossings were installed with a mixture of native, fine grained, and granular soils as embedment material. Given the typical soils present throughout the Winnipeg region, installation procedures for river crossings, and likely poor backfill practices, particularly below water line, 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 M45 and anticipated to be conservative for the crossings in this program's inventory.

7.5.3 Internal Pressure

The proper assessment and application of internal hydrostatic pressures is critical for assessing a pipe's risk of failure and/or governing failure modes. Many factors need to be taken into account, including larger system impacts, normal operating conditions vs. worst case scenarios, and potential transient events.

7.5.3.1 Water Crossings

All water crossings in this program are part of the City's regional distribution system and have been modeled in accordance with the normal operation on the system. Pipeline assessments for components within the City's regional distribution system typically assume the following:

- A steady state operating pressure of 551.6 kPa (80 psi).
- A transient overpressure of the greater of 275 kPa (40 psi) or 40% of the steady state pressure.

While the regional distribution system is reasonably protected from transient conditions due to the looped configuration and regular offtakes, accounting for some degree of transients is considered good practice. As noted above, a 40% transient allowance is typically accounted for when assessing components in the City's regional distribution system. This is based on the recommendations found within AWWA C304[24]. While none of the inventory in our program is Prestressed Concrete Cylinder Pipe (PCCP), all of the crossings are connected directly to PCCP feeder mains and thus the overarching design principal is applicable. The 40% allowance utilized is considered to be conservative based on the configuration and operation of the system.

In addition to transient overpressures, a full vacuum transient has also been considered in our analysis. While also conservative, good practice dictates the use of full vacuum allowances where protection is not provided or where a complete system transient analysis has not been completed.

7.5.3.2 Force Main Crossings

Sewage force mains were modeled using the steady state discharge pressure from the pump and a transient overpressure based on the Joukowsky equation. The operating pressures utilized in our assessment were determined through both hydraulic modeling and discussions with City Wastewater Operations staff.

Site 3 – St. Vital Bridge Force Main

Pressures utilized in the analysis of the St. Vital Bridge Force Main are based on hydraulic modeling of the force main, which identified operating pressures in the section across the bridge of 48 kPa (7 psi).

Site 4 – Newton Ave Force Main

AECOM modeled the Hawthorne and Linden Pumping Station force mains and found that the HGL for the HDPE force main under normal operating conditions was below grade at the upstream chamber location or approximately 228.8 m. This equates to 122 kPa at the lowest point of the siphon.

Site 5 – Heritage Park Force Main

Based on discussions with City of Winnipeg Wastewater Operations staff we understand the Heritage Park Force Main typically operates at 90 kPa (13 psi) with a single pump running at the pump station. AECOM's hydraulic analysis of the force main indicates a maximum worst case operating condition of 345 kPa (50 psi) at the lowest point of the siphon. While we understand that this is a highly unlikely scenario and inconstant operational scheme for the station, the worst case pressure is well below the rated operating pressure of the pipe material and has thus been carried through in the structural calculations.

Transient Pressures

The Joukowsky equation is used for estimating transient pressures in pipe systems. It is considered a conservative approach and typically produces transient pressures in excess of those determined through more intensive transient modeling. Equation 2 and Equation 3 are the Joukowsky and pressure wave velocity calculations, respectively, used in our analysis.

---

[24] *Design of Prestressed Concrete Cylinder Pipe, AWWA C304-14(R19), American Water Works Association, 2019.*

$$\Delta H = \frac{\alpha}{g} \Delta V$$

Equation 2: Joukowsky Equation

$$\alpha = \frac{12\sqrt{\frac{K}{\rho}}}{\sqrt{1+\left(\frac{K}{E}\right)\left(\frac{D}{t}\right)C_1}}$$

Equation 3: Joukowsky Equation - Pressure Wave Velocity

In addition to transient overpressures, a full vacuum transient has also been considered in our analysis. While also conservative, good practice dictates the use of full vacuum allowances where protection is not provided or where a complete system transient analysis has not been completed.

The operating and transient conditions used in our structural calculations for three force mains have been listed in Table 14.

Table 14: Internal Pressures – Force Main Crossings

Site	Operating Pressure (At Lowest Point)	Transient Overpressure	Transient Vacuum
Site 3	121 kPa	345 kPa	98.6 kPa
Site 4	45 kPa	153 kPa	98.6 kPa
Site 5	345 kPa	272 kPa	98.6 kPa

7.5.4 External Hydrostatic Head

Working near the river, consideration must be given potential flood events. Therefore, City of Winnipeg Flood Protection Levels (FPL's) have been used at each site in our assessment (see Table 15).

Table 15: Flood Protection Level

Site	Flood Protection Level (m)
Site 1	230.76
Site 2	233.00
Site 3	N/A
Site 4	229.51
Site 5	232.12
Site 6	230.89

7.6 Structural Checks – Steel Pipe

Structural checks on the steel river crossing pipelines were undertaken using both the ASCE Manual of Practice 119[25] (ASCE 119), and AWWA Manual of Practice 11[26] (AWWA M11), and ASME Manual B31G[27] (ASME B31G). The steel crossing pipes were generally checked for the following failure modes:

- Deflection.
- Wall Crushing.
- Lateral Soil Pressure.
- AWWA Buckling.
- Internal Pressure.
- Pressure Capacity Reduction due to Corrosion Pitting.
- Longitudinal bending (aerial crossings only).

As the St. Vital Bridge Force Main is not exposed to external soil loading, our analysis was limited to internal pressure and longitudinal bending.

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 1 – Kildonan-Redwood Feeder Main.
- Site 2 - Charleswood-Assiniboine Feeder Main.
- Site 3 – St. Vital Bridge Force Main.

A summary of our loading assessment results can be found in Table 16.

Deflection

Deflections have been estimated using both the Modified Iowa Formula and soil strain methods outlined in ASCE 119. The Modified Iowa Formula (Equation 4) 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 (ASCE 119). While both evaluation methods are utilized, as a majority of the City’s water crossings fit this description, the Modified Iowa Formula governs in most cases.

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

Equation 4: Modified Iowa Formula

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

---

[25] *Buried Flexible Steel Pipe: Design and Structural Analysis (ASCE Manual of Practice 119), ASCE, 2009.*

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

[27] *Manual for Determining the Remaining Strength of Corroded Pipelines, ASME, 2012.*

- AWWA deflection limits:
  - 5% for pipes with flexible linings or coatings.
  - 3% for pipes with rigid linings and flexible coatings.
- Soil slippage causing buckling.
- Outer fibre stress in the pipe wall.

Deflection is generally considered a serviceability criterion to minimize damage to coatings on steel pipelines and reduce the risk of buckling failure. Thus, a FS of unity (1) is typically applied. 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.

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) (AWWA M11), where applicable. Equation 5 and Equation 6 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 5: Constrained Buckling Pressure

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

Equation 6: Unconstrained Buckling Pressure

Applied buckling pressures include transient pressures (both internal and external) and are therefore not applied concurrently to the pipe. Equation 7 and Equation 8 are used to calculate the applied buckling pressures on the pipe.

$$q_a \geq \frac{\gamma_w H_w}{1000} + R_w \frac{W_s}{OD} + \frac{W_L}{OD}$$

Equation 7: Applied Buckling Pressure (Live Load)

$$q_a \geq \frac{\gamma_w H_w}{1000} + R_w \frac{W_s}{OD} + P_v$$

Equation 8: Applied Buckling Pressure (Internal Vacuum)

The following FS are applied to AWWA M11 buckling checks:

- Constrained buckling – 2.0.
- Unconstrained buckling – 1.3.

Internal Pressure

Internal pressures impart a tension stress in the circumferential direction of pipelines called hoop stress and is calculated using the Barlow formula, see Equation 9. 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.

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

Equation 9: Hoop Stress

For steel pipe design, the imparted hoop stress is limited to 50% and 75% of the yield strength of the wall material for normal operation and transient surge pressures, respectively.

Corrosion Effects on Internal Pressure Capacity

In addition to evaluating the hoop stress for reduced wall thicknesses, a Level 1 assessment was undertaken for all steel pressure pipe crossings to assess the effects of pitting on the internal pressure capacity as per ASME B31G. Using this methodology, a failure stress is estimated based on the axial length and maximum depth of pitting in a single corrosion feature (a string of corrosion pits). Figure 60 depicts the assumed condition for an ASME B31G analysis. 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 by PICA, the depth of pitting and length of defect was provided. Further, AECOM UT measurements on the St. Vital Bridge Force Main indicate a strip of corrosion that extends from end to end of the pipe. 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 reduction in pressure capacity at the maximum extent of the potential corrosion defect string.

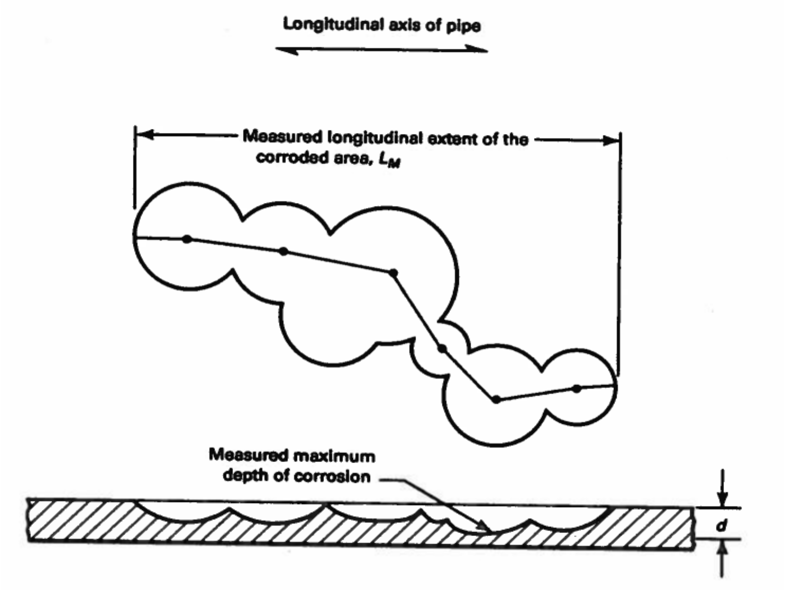


Figure 60: ASME B31G - Pressure Capacity Analysis of Corrosion Pitting

Longitudinal Bending

Aerial pipeline crossings essentially act as beams supporting their self weight and that of the material within. Thus, they are exposed to longitudinal bending moments, governed by the spacing of the pipe supports and applied loads. AECOM completed an assessment of the St. Vital Bridge Force Main in accordance with AWWA M11. Longitudinal bending must consider not only the imparted bending moments but also internal pressure and vacuum loads through the superposition of stresses within the pipe wall.

The calculation of longitudinal bending stress must consider both the support spacing and end conditions (fixed, continuous, or free). Fixed or continuous supports end conditions will reduce the magnitude of the imparted bending stresses and for a majority of the pipe would be applicable. However, this is not the case at the ends of the pipeline and thus, a free end support has been conservatively used in our analysis.

Equation 10, Equation 11, and Equation 12 are utilized to calculate the maximum imparted stresses in the pipe wall when assessing longitudinal bending. As per

$$M = \frac{WL^2}{8}$$

Equation 10: Longitudinal Bending - Maximum Bending Moment

$$\Delta x = \frac{5wL^4}{384EI}$$

Equation 11: Longitudinal Bending - Maximum Deflection

$$\sigma = \frac{M * OD}{2I}$$

Equation 12: Longitudinal Bending - Outer Fibre Stress

Similar to the pressure checks for steel pipe, stresses are limited to 50% and 75% of the yield strength of the wall material for normal operation and transient surge pressures respectively.

7.7 Structural Checks – Cast Iron

The FGSV Feeder Main, the only Cast Iron pipe in this program not exposed to external loading and corrosion of the pipeline is, based on our assessment, expected to be negligible. No structural checks have been undertaken on the FGSV Feeder Main.

7.8 Structural Checks – HDPE Pipe

Structural checks on the Newton Ave HDPE Force main (Site 4) were undertaken in accordance with AWWA Manual of Practice 55[28] (AWWA M55) and the Plastic Pipe Institute (PPI) Handbook of Polyethylene Pipe[29] (PPI Handbook). The HDPE crossing was checked for the following limit states:

- Deflection.
- Wall Crushing.
- AWWA Buckling.
- Internal Pressure.

Deflection

Deflection of the pipe ring was estimated using the Modified Iowa Formula (Equation 4) as described in 7.6. Deflection limits for HDPE pipe are based on a maximum deflection of 7.5%, in accordance with the PPI Handbook and outer fibre stress in the pipe wall. In accordance with the PPI handbook, outer fibre stress is limited to 5% with a FS of 2.0.

Deflection is generally considered a serviceability criteria to minimize damage to coatings on steel pipelines and reduce the risk of buckling failure. Thus, a FS of unity (1) is typically applied.

Wall Crushing

Wall crushing of the HDPE pipe wall was checked using externally applied soil loads.

AWWA Buckling

AWWA buckling checks were undertaken as described in 7.6 except that unconstrained buckling includes an ovality correction factor (f_o), (see Equation 13):

---

[28] *AWWA, PE Pipe Design and Installation (M55), First Edition, 2006*
[29] *PPI, Handbook of Polyethylene Pipe, Second Edition, 2008.*

$$q_a = \frac{f_o}{FS}\frac{24EI}{(1-\nu^2)d^2}$$

Equation 13: Unconstrained Buckling - HDPE Pipe

The following FS are applied to the buckling checks based on the PPI Handbook:

- Constrained buckling – 2.0.
- Unconstrained buckling – 2.5.

Internal Pressure

Internal pressures impart a tension stress in the circumferential direction of pipelines called hoop stress and is calculated using the Barlow formula as noted herein. For HDPE pipe design, the imparted hoop stress is limited to the specified Hydrostatic Design Stress (HDS) for the constituent HDPE resin. Factors of safety are built into the determination of the HDPE resin's HDS.

7.9 Structural Checks – PVC Pipe

Structural checks on the PVC Heritage Park Force Main (Site 5) were undertaken in accordance with AWWA Manual of Practice 23[30] (AWWA M23) and the Uni-Bell Handbook of PVC Pipe[31] (Uni-Bell Handbook). The PVC crossing was checked for the following limit states as described above for the steel pipe:

- Deflection.
- Wall Crushing.
- AWWA Buckling.
- Internal Pressure.

Deflection

Deflection of the pipe ring was estimated using the Modified Iowa Formula (Equation 4) as described in 7.6. Deflection limits for PVC pipe are based on a maximum deflection of 7.5%, in accordance with the Uni-Bell Handbook and outer fibre stress in the pipe wall. Outer fibre wall strain is limited to 5% with a FS of 2.0.

Deflection is generally considered a serviceability criterion to minimize damage to coatings on steel pipelines and reduce the risk of buckling failure. Thus, a FS of unity (1) is typically applied.

Wall Crushing

Wall crushing of the PVC pipe wall was checked using externally applied soil loads with a FS of 2.0.

---

[30] *AWWA, PVC Pipe – Design and Installation (M23), Second Edition, 2002*

[31] *Uni-Bell, Handbook of PVC Pipe, Fourth Edition, 2001.*

AWWA Buckling

AWWA buckling checks were undertaken as described in 7.6 except that PVC also uses an ovality correction factor (f_o) for unconstrained buckling calculations.

The following FS are applied to the buckling checks based on the Uni-Bell Handbook:

- Constrained buckling – 2.0
- Unconstrained buckling – 2.0

Internal Pressure

AWWA M23 dictates the following design checks for internal pressure:

- Operating Pressure < Pipe Pressure Rating.
- Operating Pressure < Pipe Working Pressure Rating.
- Maximum Internal Pressure < Pipe Short Term Surge Pressure Rating.

Factors of safety are built into the PVC pipe's defined pressure rating and short term surge pressure rating.

7.10 Structural Assessment Results

Table 16 lists the external loading conditions checked for each crossing along with the governing failure mode and resulting FS. Table 17 lists the results and FS's from the internal pressure structural checks. Where noted, the FS's presented in Table 16 and Table 17 represent the FS above accepted design criteria for the respective pipe material.

AECOM's structural checks indicate that all sites except for the Newton Ave Force Main have sufficient structural capacity to support all imparted internal and external loading. This may preclude catastrophic failure but does not preclude other modes of failure (loss of hydrostatic integrity, river bank movement, etc.).

The St. Vital Bridge Force Main, despite extreme wall loss along the entire invert of the pipe has adequate strength in both the longitudinal and hoop directions to support the imparted service loads should it be rehabilitated. As noted above, structural checks on the St. Vital Bridge Force Main have conservatively assumed a wall thickness of 1 mm around the entire circumference of the pipe. This is consistent with AECOM's UT measurements for the remaining pipe wall along the invert of the force main. Based on our structural checks, with a 1 mm wall thickness the pipe has adequate strength in the hoop and longitudinal directions to support all applied loads with a FS of 2.23 above acceptable design practice in longitudinal bending.

Table 16: Structural Assessment Results – External Loading

Site	Crossing Location	Pipe Material	Nominal Diameter (mm)	Reviewed Condition	Governing Failure Mode	Factor of Safety	Notes
1	Kildonan-Redwood Feeder Main	Steel	600	Max Soil Cover	Deflection	2.00*	
1	Kildonan-Redwood Feeder Main	Steel	600	Max External Hydraulic Head	Buckling	1.58*	
2	Charleswood-Assiniboia Feeder Main	Steel	600	Max Soil Cover	Deflection**	3.04*	

Site	Crossing Location	Pipe Material	Nominal Diameter (mm)	Reviewed Condition	Governing Failure Mode	Factor of Safety	Notes
2	Charleswood-Assiniboia Feeder Main	Steel	600	Max External Hydraulic Head	Buckling**	3.06*	
3	St. Vital Bridge Force Main	Steel	500	Longitudinal Bending	Longitudinal Bending**	1.79*	
4	Newton Avenue Force Main	HDPE	350	Max Soil Cover	Buckling	1.15***	
4	Newton Avenue Force Main	HDPE	350	Max External Hydraulic Head	Buckling	1.18***	
5	Heritage Park Force Main	PVC	250	Max Soil Cover	Buckling**	3.58*	
5	Heritage Park Force Main	PVC	250	Max External Hydraulic Head	Buckling**	3.53*	
6	FGSV Feeder Main	CI	600	N/A****	N/A****	N/A****	
* FS above recommended design conditions							
** Governing condition excluding internal pressure							
*** Does not include transient vacuum							
**** No structural assessment undertaken							

Table 17: Structural Assessment Results – Internal Pressure

Site	Crossing Location	Pipe Material	Nominal Diameter (mm)	Reviewed Condition	Governing Failure Mode	Factor of Safety	Notes
1	Kildonan-Redwood Feeder Main	Steel	600	Internal Pressure	ASME B31G Internal Pressure	2.37*	
2	Charleswood-Assiniboia Feeder Main	Steel	600	Internal Pressure	ASME B31G Internal Pressure	1.66*	
3	St. Vital Bridge Force Main	Steel	500	Internal Pressure	Hoop Stress	7.61*	
4	Newton Avenue Force Main	HDPE	350	Internal Pressure	Hoop Stress	1.80*	
5	Heritage Park Force Main	PVC	250	Internal Pressure	Hoop Stress	3.00*	
6	FGSV Vital Feeder Main	CI	600	Internal Pressure	N/A**	N/A**	
* FS above recommended design conditions							
** No structural assessment undertaken							

7.10.1 Newton Ave Force Main

Our structural calculations indicate that the Newton Ave Force Main is extremely sensitive to transient vacuum conditions as well as dewatering due to a lack of buckling capacity. Utilizing the design parameters identified herein, the Newton Ave Force Main has insufficient capacity for full vacuum conditions and fails under buckling when dewatered and exposed to flood conditions. Normal flexible pressure pipe design practice dictates that external loading and internal pressure is checked independently from each other as internal pressure acts to resist external loading, which wouldn't be present when depressurized. While we would normally apply this approach to river crossings for consistency with industry good practice, where river crossing siphons cannot be dewatered without a third party effort, accounting for internal pressure when assessing external loading is an accepted reasonable approach.

The results presented in Table 16 include the following special considerations:

- The internal operating pressures are included in the buckling calculations for both scenarios considered.
- The max soil cover scenario utilizes a constrained soil modulus of 6.9 MPa. As this condition is located well away from the river bank the use of a higher soil modulus is justifiable. A constrained soil modulus of 6.9 MPa is consistent with minimum industry recommendations for the installation of flexible pipelines.
- A transient vacuum has not been included.

Logically, the critical location for a transient vacuum to occur at is the lowest point of the siphon where the greatest external hydrostatic pressure is applied. Additional analysis to determine the real FS against a transient vacuum found that under the following conditions the pipeline has a true FS greater than one (1) with a full transient vacuum:

- A constrained soil modulus of 6.9 MPa. This is in excess of what was assumed for the structural assessments in this program but within the realm of feasible.
- An external hydrostatic pressure equal to a normal summer river level of 223.74 m.

Therefore, while the pipeline is operating at an extreme risk of buckling failure; under most normal operating conditions it will have a FS against a transient vacuum of near unity (1).

7.11 Condition Assessment Summary

The following is a site by site summary of the CA process.

7.11.1 Site 1 – Kildonan Redwood Feeder Main

PICA's inspection of the Kildonan Redwood Feeder Main found it to be in relatively good condition with pitting corrosion defects limited to 40% remaining pipe wall. Structural checks also found the feeder main to have adequate remaining structural capacity. However, leakage tests measured an apparent leak in the feeder main of 300-400 L/hr. Thus, planning is currently underway to rehabilitate the crossing before putting it back into service.

7.11.2 Site 2 – Charleswood-Assiniboia Feeder Main

PICA's inspection of the Charleswood-Assiniboia Feeder Main found pitting corrosion defects with as little as 16% remaining pipe wall. This equates to remaining service life of 10 years. Beyond the identified pitting corrosion, the pipeline passed a leakage test and has adequate remaining structural capacity. Given the wide variability in assessment methods, rehabilitation of the pipeline is recommended in the 5 year term.

7.11.3 Site 3 – St. Vital Bridge Force Main

Leaks were discovered on the St. Vital Bridge Force Main during preparation for inspection and subsequent UT measurements by AECOM revealed widespread pipe wall loss along the entire invert of the aerial crossing. AECOM measured wall thicknesses as low as 1 mm along the invert. Structural checks confirm that the pipe, despite extreme wall loss, has sufficient hoop capacity to support internal loads. However, erosion corrosion patterns in the invert are problematic even in the short term and have resulted in several active leaks on the pipeline. Based on the condition of the force main, planning is currently underway to rehabilitate or replace the crossing in as short a time frame as practical and emergency by-pass capability remains in place.

7.11.4 Site 4 – Newton Ave Force Main

Through the SONAR inspection by AquaCoustic, the Newton Ave Force Main was found to be exhibiting extreme pipe wall deformation, resulting in the application of high wall stresses. Coupled with the pipe's low resistance to SLG, the pipeline needs to be replaced in the short term. Further, structural checks have confirmed the pipe has extremely limited buckling resistance to transient vacuum conditions. If the pipeline remains in service, regular leakage tests should be undertaken an annual basis.

7.11.5 Site 5 – Heritage Park Force Main

Structural checks on the Heritage Park Force Main determined the pipe was designed very conservatively for the current operating conditions. Material testing found that the pipe material itself suffers from manufacturing quality issues but based on the operating conditions the force main is subjected to, this is not likely to result in premature failure of the main. No action is required in the near term.

7.11.6 Site 6 – FGSV Feeder Main

Based on the installation and operating conditions, the FGSV Feeder Main is not exposed to external face corrosion, a major driver for ferrous metal pipe deterioration in Winnipeg. Internal inspection by Pure found internal face corrosion and tuberculation to be minor, consistent with ferrous metal pipelines in the Winnipeg distribution system. No action is required in the near term.

8. Failure Risks and Rehabilitation

This section of the report discusses the failure risk of each asset based on the findings of the inspection program and the CA process and recommends future inspection frequency and approaches and/or rehabilitation works for each site.

8.1 Risk Based Rating System

Failure risks associated with the crossings included in the current program were initially categorized in the 2006 WWS River Crossing Risk Assessment Report[32] and Water Main Criticality Study[33] completed by AECOM (UMA Engineering). Each river crossing within the City of Winnipeg was rated for both its probability and consequence of failure based on information known and available at the time. This rating was used to some extent to drive the development of the current inspection program. Overall, the risk based rating system is a reconciliation of the consequence of a failure, vs the probability of that failure occurring.

By nature, the river crossing inventory are highly critical assets. Failure of the asset in many cases could result in widespread loss of service, and in all cases the release of contaminated fluids to the environment. This changes the nature of the management of the HRRC program to one that needs to mitigate the occurrence of failure to very low levels.

At the time of the original risk assessments, no direct CA had been undertaken on the river crossings, thus the probability of failure was based on record information and the characteristics of pipeline materials, environment, and ability to inspect and monitor the asset, including:

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

All assets in this program are considered to have varying degrees of high criticality. Through initial screening in the studies referenced earlier in this section, the risk of failure of these assets was considered to be the high end of low to medium. A plot of the probability of failure rating vs the consequence of failure rating for each crossing is provided in Figure 61. Without completing CA on the crossings, four of the crossings would have been identified for planned replacement in the near term (the crossings with medium probability and high consequence ratings).

---

[32] *UMA/AECOM, "WWS River Crossing Risk Assessment", Report for the WWD, December 2006*

[33] *AECOM, "Water Main Criticality Study Technical Memorandum 2.1 – City of Winnipeg Watermain River Crossing Design, Construction and Management Standard" report for WWD, July 2011*

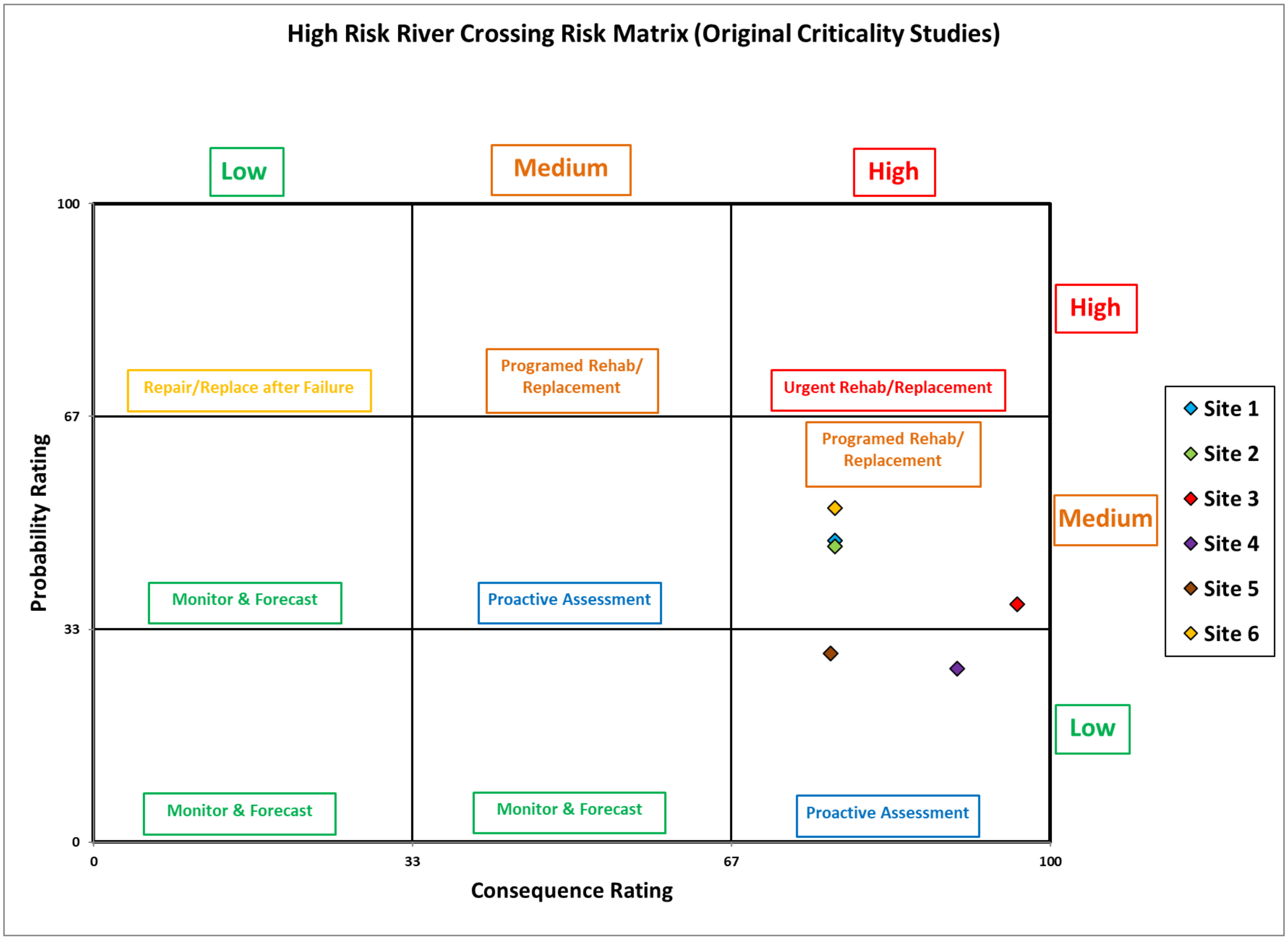


Figure 61: Risk Matrix – Original Criticality Studies

8.2 Pipeline Failure Risk

The remaining service life for each asset has been estimated using the information gathered throughout the program, including corrosion defects reported by PICA, sonar inspection, material testing, and leakage tests. As outlined in Section 7, the remaining service life for the assessed pipelines ranges from 0 years (Site 1, 3, and 4) to those with no appreciable defects and anticipated long remaining service life (Site 5 and 6). 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 5% was applied to all sites to reflect level of certainty of the inspection technology and criticality of river crossing infrastructure in general. The calculated estimated remaining service life for each asset is listed in Table 18.

Table 18: Estimated Remaining Service Life

Crossing	Estimated Remaining Service Life (Years)
Site 1 - Kildonan-Redwood Feeder Main	0*
Site 2 - Charleswood-Assiniboia Feeder Main	10
Site 3 - St. Vital Bridge Force Main	0
Site 4 - Newton Avenue Force Main (HDPE)	0
Site 5 - Heritage Park Force Main	50**
Site 6 - Fort Garry – St. Vital Feeder Main	50**
***Apparent Leakage in the feeder main has driven rehabilitation.**	
****No discrete deterioration discovered. 50 years of remaining service life estimated.**	

8.3 Geotechnical Failure Risk

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 assigned a probability of failure as noted in Table 19.

Table 19: Probability of Failure - Visual Assessment

Visual Assessment Rating	Probability
1	1
2	10
3	80

Where a slope stability analysis was warranted (typically where a visual rating of 3 was assigned), the resulting FS was rated in accordance with the rating scale presented in Table 20.

Table 20: Probability of Failure - Slope Stability Analysis

Slope Stability Analysis - Factor of Safety	Probability of Failure (%)
Greater than 1.5	<1
Between 1.4 and 1.5	5
Between 1.3 and 1.4	10
Between 1.0 and 1.2	80
Equal to or less than 1.0	100

There is no direct published correlation between FS and Probability of Failure (POF) in terms of geotechnical assessment. Probability of failure would consider the Coefficient of Variation (COV) of the various engineering properties, as well as the geometry in determining risk of failure. A slope condition with a lower FS and a low COV may be at less risk of failure than a slope with a higher FS and a higher COV of the input data.

However, it is generally accepted that a long-term FS of 1.5 is considered acceptable for critical infrastructure and it would be generally accepted that this would be associated with a probability of failure of less than 1% in the life of the asset.

Within the geotechnical risk assessments completed, there can be numerous FS associated with a single embankment, with highly variable consequences of failure. e.g. a failure that would engage the pipe would have a higher consequence of failure than a shallow rooted slip surface above the pipe, or a toe of slope failure. However, the latter two conditions, if left unattended for a long period, could result in retrogressive slips that will lower the overall FS, and ultimately lead to global failures that could intercept the pipe. Table 21 summarizes safety factors of analysis completed.

Table 21: Geotechnical Summary- Analysed Sites

Site	Site Description	Global Slip Stability		Global Stability Engaging the Pipe		Toe Slip Surface	
		East	West	East	West	East	West
1	Kildonan-Redwood Feeder Main	1.1 - 1.2	N/A*	1.4 -1.6	N/A*	0.8 – 0.9	N/A*
5	Heritage Park Force Main	1.5	1.8	1.5	1.8	1.5	1.4
6	FGSV Feeder Main	1.5	1.3	1.5	1.5**	1.5	1.3
NOTES	* Not analysed. River bank failures do not intercept pipe ** Failure surface does not intercept the pipe, but may intersect adjacent infrastructure						

The three other sites included in this program were not analysed, but there is no evidence, based on site inspection, of instabilities that may affect the pipe. As noted in Section 4, the St Vital site does exhibit some toe instabilities, however, the risk to the pipe is considered low, as the pipe is shielded from movement by the bridge piers. All sites do have noted toe erosion which should be addressed to prevent retrogressive failures in the future.

The only critical site that should be addressed in the short term is the Kildonan-Redwood east bank. Currently, there is adequate stability against slope failures engaging the pipe. However, shallower global slip surfaces that could reach the top of the pipe are inadequate for critical infrastructure. Shallower failures and toe erosion will alter the safety factors for a deep rooted failure, should they occur. It is recommended that a more deterministic study be completed of the site, including subsurface investigations, monitoring and further analysis be completed in the near term.

8.4 Updated Risk Matrix

The risk profile for the assets included in this program were updated as follows:

- Sites 1, 3, and 4 were elevated to a high probability of failure. In fact, they had failed and were actively leaking.
- Failure risk increased for Site 2 but it remained in the medium probability of failure classification.
- Site 5 and 6 were reduced to a low probability of failure.

Figure 62 displays current risk rating, based on assessed pipe condition and geotechnical conditions.

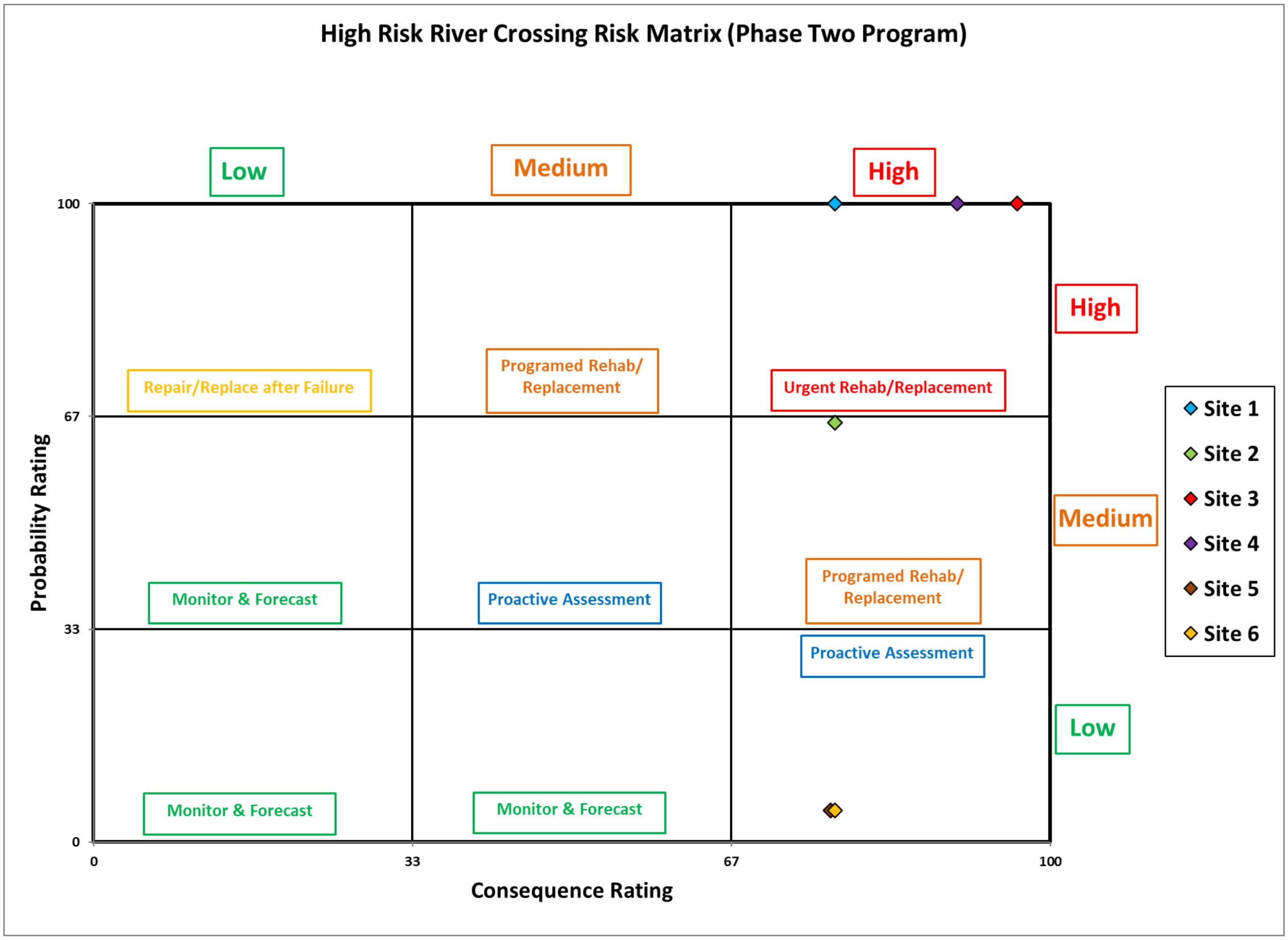


Figure 62: Current Pipe Failure Risk Rating

8.5 Pipe Rehabilitation and Rehabilitation Recommendations

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 armouring for toe erosion control, or improving FS against buoyancy.

Replacement of river crossing assets carries a very high cost. Changes in the regulatory environment have changed the technical requirements for reconstructing many of the assets by which they were originally constructed. Department of Fisheries and Oceans (DFO) requirements now put a very strong emphasis on using construction techniques such as tunnelling or horizontal directional drilling (HDD) that have lower risk impact on fish habitat. Additionally, regulatory changes, particularly for pressurized sewer crossings require dual encasement of water course crossings (unless removed by special application), and fines for inadvertent discharge of liquids to the environment continue to increase.

These factors have elevated the replacement cost of river crossings 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 due consideration for potential habitat destruction. Rehabilitation of existing assets can be significantly more economical if they are technically feasible.

Based on review of the pipe location, asset conditions and available technologies, the following rehabilitation or replacement method recommended, see Table 22. Sites 5 and 6 do not require rehabilitation in the foreseeable future.

Table 22: Rehabilitation Methods

Site	Crossing Location	Nominal Diameter (mm)	Length (m)	Recommended Pipe Rehabilitation or Replacement
1	Kildonan Redwood Feeder Main	600	228.6	Rehabilitate by pressure CIPP Lining
2	Charleswood-Assiniboia Feeder Main	600	239.3	Rehabilitate by pressure CIPP lining or Structural Liner
3	St. Vital Bridge Force Main	500	186.0	Rehabilitate by CIPP Lining
4	Newton Ave Force Main	350	295.0	Replace by HDD

8.5.1 Pipeline Rehabilitation Methods

A summary of pipe rehabilitation methods is provided below for general reference.

Cured in Place Pipe

Cured in Place Pipe (CIPP) lining process involves the inversion of a resin impregnated felt tube into an existing pipe using water or air pressure. The resin within the felt tube is then cured using hot water, steam, or ultraviolet light. CIPP is a mature rehabilitation technology with considerable deployment experience in the Winnipeg non pressure market.

Installation of CIPP liners through siphons poses technical challenges over conventional gravity sewers. However, it has been demonstrated across North America and locally (through successful repair of the St James Interceptor siphons in the Fall of 2015) that CIPP lining is a viable method of rehabilitation of many siphons including pressure applications. Some of the challenges to be addressed in design include:

- Siphons often cannot be dewatered, either from a practical construction standpoint (dewatering cost and complexity) or due to floatation concerns.
- Cleaning and inspection operations become more complex where dewatering is not possible. Alternate inspection methods such as SONAR may be required.
- Traditional cleaning methods may not be 100% effective at cleaning the entire pipe’s circumference. Pigging or scraper discs pigs will likely be required.
- Longer inversion lengths, horizontal and vertical bends may introduce additional risks.
- Submerged crossings will require hot water curing due to potential variable heat loss. Use of continuous monitoring technology would be advised.

Additionally, potable water pressure crossings include additional restrictions:

- Water crossing require specialised resins such as epoxies and must meet drinking water standards such as NSF61.
- Higher pressure crossings such as distribution water systems require reinforced liners to adequately deal with hydrostatic pressures.
- Due to the resin requirements, low moisture tolerance and concerns over resin washout, crossings must be capable of being fully dewatered for liner installation.

From review of constraints, CIPP technology is a viable rehabilitation method for Kildonan-Redwood Feeder Main and the St. Vital Bridge Force Main. The Charleswood-Assiniboia Feeder Main crossing would pose additional challenges for dewatering and a review with pressure CIPP installers at the time of the work is recommended to gauge the suitability of the crossing for rehabilitation using CIPP.

Other Technologies

For pressurized potable water crossing, where the pipe cannot be fully dewatered, one technology that in theory can be deployed are flexible pulled in place Kevlar reinforced tubes such as Primus Line. These types of tubes are pulled into the water main and anchored on either end, creating a hydrostatically integral conduit. They, however, have minimal external load carrying capacity and rely on the host pipe and internal pressure to resist overburden loads and external hydrostatic pressure. Primus Line will result in downsizing of the crossing. Recent discussions with Primus Line have indicated that their maximum liner diameter is 500 mm, which would result in an appreciable reduction in the cross section and hydraulic capacity of the 600 mm feeder mains included in this program.

On a site-specific basis, this translates to the following constraints and opportunities:

8.5.1.1 Site 1 - Kildonan Redwood Feeder Main

The Kildonan-Redwood Feeder Main (Site 1) has been discussed with Insituform Technologies, one of the largest pressure lining companies in North America, regarding the feasibility of installing a CIPP liner within the existing pipeline. With the apparent leaks present on the crossing, there will be technical challenges, but is believed to be technically feasible and the WWD is currently working towards tendering a CIPP rehabilitation tender for the crossing.

8.5.1.2 Site 2 - Charleswood Assiniboine Feeder Main

CIPP technology may be viable for the Charleswood-Assiniboine Feeder Main (Site 2), if it can be adequately dewatered deploy potable water epoxy resins.

A further consideration for the Charleswood-Assiniboine crossing would be installation of an active cathodic protection system to arrest external face corrosion. Addition of a cathodic protection system could extend service life of the pipe by several years. From a budgeting standpoint, we have included replacement cost only in cost estimates.

8.5.1.3 Site 3 - St Vital Bridge Force Main

The St Vital Bridge Force Main (Site 3) crossing is feasible to line with CIPP technologies. Preliminary design has been discussed with at least one vendor, who believes this to be feasible with use of air inversion/steam curing methods. Other considerations at the site, in terms of overall modification of the system have not been reviewed herein.

8.5.1.4 Site 4 - Newton Avenue Force Main

The Newton Ave Force Main (Site 4) is not a good candidate for any type of lining due to the level of deformation present and challenges associated with the use of CIPP in HDPE siphons. The most suitable replacement of this crossing will be by Horizontal Directional Drilling (HDD) of a new crossing. It is noted that current treatment plant regulation requires sewage crossings to be dual contained, which will increase the size and cost of this crossing.

8.6 River Bank Stabilization

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.

Site 1 – Kildonan-Redwood Feeder Main should be further assessed for required rehabilitation. While the current FS against failure engaging the pipe is moderate, it is based on limited subsurface information. Shallower failures and continued toe erosion will affect the overall FS.

9. Conclusions and Recommendations

9.1 General

AECOM completed CA on six high risk river crossing assets the Phase Two Program. This is an extension of Phase One Program completed by AECOM in 2016. Four of the six assets assigned to this program were included in the Phase One Program but were removed from that program or required additional inspection for various logistical and planning reasons.

Of the six assets assessed, three were found to be actively leaking during program testing (Kildonan-Redwood Feeder Main, St. Vital Bridge Force Main, and Newton Force Main), one was found to be in serviceable condition (Charleswood- Assiniboine Feeder Main), but should be considered for rehabilitation within a five year planning cycle, and two were found to be in good condition (Heritage Park Force Main and FGSV Feeder Main), and require no remedial works in the foreseeable future.

A detailed summary of the CA findings is provided below.

9.2 Site by Site Recommendation

9.2.1 Site 1 – Kildonan-Redwood Feeder Main

Electromagnetic inspection results at the Kildonan Redwood Feeder Main, as provided by PICA Corp, suggest that the pipe material is in good condition. The lowest reported average wall thickness was 96% RW, with only 7 indications of localized wall loss of 40-65% RW, and 11 indications of RW greater than 65% RW. While the Feeder Main crossing was found to be in reasonably good condition material wise, a low pressure leakage test could not be successfully completed. Attempts by the City of Winnipeg to locate the leaks by acoustic leak detection correlator were unsuccessful. It is suspected that leaks may be occurring at flange joint connections, however, this could not be confirmed. The pipe remains out of service, and the apparent leaks must be addressed prior to putting the pipe back into service.

Structurally, the pipe appears to be adequate, and would be a suitable candidate for rehabilitation. AECOM has reviewed potential remedial technologies and recommends completing rehabilitation using CIPP technology.

The west riverbank is known to be unstable, however, the pipe is located outside any zone of instability. The eastern riverbank slope currently has a reasonable FS for global failures engaging the pipe, however, has marginal stabilities for shallower rotational failures above the pipe, and unacceptable FS against toe erosion. Toe erosion will ultimately result in retrogressive failures that will lower the factors of safety and eventually result in slope engagement of the pipe. Additional geotechnical investigations, monitoring and slope stability analysis are recommended. At a minimum, erosion protection of the toe of the slope are recommended to protect the pipe from a slope movement related failure.

9.2.2 Site 2 – Charleswood-Assiniboia Feeder Main

Inspection of the Charleswood-Assiniboia Feeder Main was completed in March 2019 by PICA. The RFT inspection identified numerous corrosion related defects with RW thicknesses of as low as 16% of the original wall thickness. During removal of buried fittings in preparation for inspection, several through wall defects were discovered. The removed fittings were repaired by welding and reinforcement, sandblasted, and recoated prior to reinstallation.

Structural assessment estimated remaining lifespan could be as long as 10 years based on extrapolated corrosion rates alone. Based on the limitations of the analysis and assessment it would be prudent to plan on rehabilitating the crossing within the next 5 year capital cycle. The most cost-effective rehabilitation technique that is technically feasible would be a pulled in place flexible reinforced liner, such as Primus Line. Based on a more detailed assessment, CIPP technology may be determined to be technically feasible as well. Consideration should also be given to the application of external corrosion protection as this will increase reliability of the main in the short and long term.

There is no visual indication of slope instability, however, toe erosion is evident near the waterline. Toe armoring of the lower river banks is recommended to address erosion issues and minimize future retrogressive failures.

9.2.3 Site 3 – St. Vital Bridge Force Main

During preparation for the EM inspection in October 2018, several small active leaks were discovered by the inspection support contractor. The City had also discovered and repaired a leak in the summer of 2018, prior to commencement of the inspection program. Subsequent inspection by AECOM, using UT wall thickness measurements revealed significant pipe wall thinning (<1 mm remaining in places), preferentially along the invert of the pipeline. Based on the discovered leaks, level of deterioration found, and confirmation that the force main had no internal lining it was concluded that the pipeline had reached the end of its useful service life, and further inspection was not warranted. The proposed external EM inspection program was canceled, and the pipeline insulation was temporarily restored. Due to the numerous leaks that have recently occurred, this main should be renewed or replace in the immediate future.

As a precautionary measure, the City procured and installed a bypass system routed along the St. Vital Bridge over the Red River. The City issued a separate request for proposal to review rehabilitation and replacement options for the force main.

Based on the assessment carried out under this program it was concluded that in-place rehabilitation using CIPP technology was technically feasible if in-place rehabilitation was desired to be carried out. Design life of CIPP liners is dependent on design and quality assurance measures but can be made to deliver useful design lives in excess of 100 years. Restoration would include CIPP lining of the bridge crossing, and the riser pipes. New slip joints would be installed at locations of the existing joints.

Generally, the WWD infrastructure is protected from slope instabilities by the bridge at the site. There is evidence of toe instabilities, however, they would affect the bridge before engaging the pipes. Toe armoring of the lower river banks is recommended to address erosion issues. Slope works are not required to protect WWD infrastructure if the force main is replaced at a different location.

9.2.4 Site 4 – Newton Ave Force Main

Material testing of the force main completed during the Phase One Program found the HDPE pipe to have very low resistance to SCG, which can make the pipe susceptible to brittle failure in response to the long-term exposure to either sustained pressure or intermittent short term over-pressure. This could be inferred to be consistent with other HDPE force mains of this era.

SONAR imagery completed under this program identified the pipe had localized areas with very high deflection, hinging and “dents” (possibly related to third party damage). Subsequent CCTV inspection under partially dewatered conditions verified the observations in those areas. Low head leakage test identified an apparent leak of over 800 L/hr. Inspection by the City identified the leak location to be immediately adjacent to the downstream (west) end of the siphon and was due to a circumferential pipe split. The leak was repaired by the City using an internal point repair. Additional leakage testing performed under this program suggests the repairs were successful.

Due to the evidence of excessive pipe deflections, poor material traits and documented leaks, it is recommended to replace the crossing in the very near term (1-3 years) and provide ongoing monitoring in the interim in general conformance to Environment Act License 2684 RRR.

Full hydrostatic leakage tests can be carried out in a manner consistent with those described in Section 6.5.2. As the force main is currently in service, the test procedure should generally include the following:

- Isolate the HDPE force main and ensure the isolation valve between the upstream valve chambers is open.
- Remove the 90 deg bend and drop pipe from within the discharge manhole.
- Install a blind flange with test port, valve, and stand pipe.
- Complete test and put the crossing back to its operating configuration.

A further opportunistic test may be to utilize river elevation to provide indicative evidence of leakage into or out of the pipe. Based on Record Drawing 911, the upstream pipe invert is 225.26 metres (739.05'), and the downstream invert is 225.67 metres (740.39'). Simply closing the upstream knife gate valve should result in the crossing level stabilizing at the invert elevation of the upstream manhole, or 225.67 metres. A normal winter water level of zero James of 221.76 metres (727.57'). This would result in a static pressure differential to the environment of 3.9 metres (approximately 5.4 psi). Observing water levels in the discharge pipe, which slopes slightly east (back) to the river may provide indication of leakage. Observation in the downstream MH could be made by removal of the downpipe in the manhole, and use of a CCTV camera to observe level in, and condition of the above water part of the pipe. A similar test could be used during river flood stage as well, to observe inflow into the pipe from the river. 225.67 metres is approximately 12.82‘ James Datum, so a relatively high flood stage would be required to develop reasonable differential pressure.

Replacement of the existing 90 degree bend on the outfall discharge manhole with a tee, with the run orientated vertically, would allow for easy deployment of hydrostatic testing or inserting of a camera to monitor water levels, without future removal of the drop pipe.

Both test procedures would require diverting both Linden and Hawthorne pump stations to the south forcemain, and also assumes the knife gate valve is reasonably tight.

9.2.5 Site 5 – Heritage Park Force Main

The assessment of the Heritage Park Force Main concluded that it originally had a very high FS against internal and external loading. The governing failure mode was hoop stress and the "as-constructed" FS was >3.6. It was concluded that the most appropriate assessment would include assessment of material properties to confirm they remain consistent with design, and to conduct a low-head leakage test of the crossing.

The material testing indicated the mechanical properties consistent with the original specifications for the pipe material, but the quality of the original extrusion was deemed to be poor as the pipe failed a heat reversion test. This was confirmed in a more robust test using the Differential scanning calorimetry (DSC) method. This is an ISO test that provides more definitive data on original extruded quality (ISO 18373-1) throughout the wall section. Based on the DSC results, the original extrusion quality of the pipe is poor; however, the pipe is not deemed to have any active deterioration processes present due to the very low wall stresses that are present in its current operating mode.

The low-head leakage test indicated that there were no issues with hydrostatic integrity. The only recommended action at the site would be reinspection including a low head leakage test in approximately 25 years as per the River Crossing Management Guidelines[34].

This site has an adequate safety factor against global stability engaging the utility. Toe armoring of the lower river banks is recommended to address erosion issues.

9.2.6 Site 6 – FGSV Feeder Main

The FGSV feeder main crossing was installed within the 1650mm Branch II Aqueduct tunnel in 1959 within the limestone bedrock stratum under the river. The tunnel annulus is fully filled with concrete, which offers a high pH environment to the outer pipe wall which creates a very reliable, very low-corrosive environment for the cast iron pipe. The program included an inspection with the Sahara acoustic and visual inspection platform under the City's existing contract with Pure Technologies. The inspection did not detect any leaks and the visual interior corrosion was considered minor, consistent with other ferrous metal pipelines in the City's water system. Based on the above, the pipeline is considered to be in good condition with no short-term rehabilitation or replacement requirements.

In accordance with the River Crossing Management Guidelines[35]. it is recommended to reinspect the crossing in approximately 20 years in the same manner (i.e. visually and with acoustic leak detection).

This site was found to have an adequate FS against global stability engaging the FGSV feeder main or Branch II Aqueduct, while slope stability doesn't impact the water utilities, the FS for the embankment engaging the adjacent interceptor sewers is estimated between 1.3 and 1.7. Standard protocol in Winnipeg has been to maintain FS for bank stability intercepting critical infrastructure at 1.5 or higher. Armoring of the lower river banks is recommended to address erosion issues affecting the adjacent WWS siphon crossing.

9.3 Summary of Recommended Works

A summary of all recommended works is presented in Table 23. Rehabilitation timing has been grouped into the following time horizons:

- 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.

All of the assets should be considered for recommended geotechnical upgrades in the short term planning horizon. Geotechnical works range from full bank stabilization, to minor toe armouring, to prevent possibility of retrogressive slope failures over time.

Below is a summary of the recommended pipeline works:

- The Kildonan-Redwood Feeder Main (Site 1) is actively leaking, but the pipe was found to be in good structural condition. It remains out of service. It should be rehabilitated as soon as possible. The pipe should be suitable for CIPP lining.

---

[34] *AECOM, "Water Main Criticality Study Technical Memorandum 2.1 – City of Winnipeg Watermain River Crossing Design, Construction and Management Standard" report for WWD, July 2011*

[35] *AECOM, "Water Main Criticality Study Technical Memorandum 2.1 – City of Winnipeg Watermain River Crossing Design, Construction and Management Standard" report for WWD, July 2011*

- The Charleswood-Assiniboine Feeder Main (Site 2) was found to be in serviceable condition, however, several through wall defects were discovered at pipe inspection insertion point. These were repaired, and pipe successfully pressure tested. Based on projected corrosion rates, the pipe should be rehabilitated in the five year capital planning cycle. Consideration should also be given to the application of external corrosion protection to increase reliability of the main.
- The St Vital Bridge Force Main (Site 3) was found to be actively leaking during this program. Several spot repairs have been made by the City and support contractors. Remaining pipe wall based on UT inspection and visual observation is very thin. The pipe should be replaced or rehabilitated as soon as possible. The pipe could be rehabilitated using CIPP technology, however, assessment of the overall system in the area is prudent to confirm that this solution is the most cost effective course of action overall.
- The Newton Avenue Force Main (Site 4) was found to be actively leaking during the program, and was found to contain excessive geometric deformation at several locations. The active leak was successfully repaired by the City. The pipe should be replaced in the immediate future. The pipe likely cannot be rehabilitated due to the geometric deformations.
- The Heritage Park Force Main (Site 5) was found to be in good condition. No remedial pipe work is required. The pipe should be re-inspected in approximately 25 years.
- The Fort Garry-St Vital Feeder Main (Site 6) was found to be in good condition. No remedial pipe work is required. The pipe should be re-inspected in approximately 20 years.

9.4 Recommended Capital Program

The capital program budget has been prepared in accordance with the AACE, Cost Estimate Classification System – As Applied in Engineering, Procurement, and Construction for the Pipeline Transportation Instructure Industries[36] (AACE). Based on the AACE classification system, a Class 5 estimate has been developed. The costs included in Table 23 account for capital costs, 15% engineering, and a 30% contingency. All costs are presented in 2020 dollars. Recommended timing has been provided based on the broad criteria of:

- Short term – Should be carried out within the next 5 year capital ($10,105,000.00).
- 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 ($310,000.00).

The proposed capital program includes the following considerations:

- 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.
- 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 pipes by directional drilling versus other traditional methods.
- For the bank stabilization works, costs range from approximately $20,000 for minor armouring, to $100,000 for regrading and toe stabilization, to upwards of $1,000,000 for full stability upgrades such as rock caissons.

---

[36] *Cost Estimate Classification System – As Applied in Engineering, Procurement, and Construction for the Pipeline Transportation Instructure Industries, AACE 97R-18, AACE, August 5, 2019.*

Table 23: Rehabilitation Methods, Costs and Timing

Site	Crossing	Nominal Diameter (mm)	Estimated Replacement Cost (2020)	Proposed Work	5 year Capital Program	10 year Capital Program	Reinspection (15-25 Year Frequency; dependent on size of crossing)
1	Kildonan-Redwood Feeder Main	600	$5,400,000.00	Geotechnical: Toe Armouring & Regrading 2 Sites	$105,000.00		
				Rehabilitation: CIPP	$2,015,000.00		
				Reinspection			
2	Charleswood-Assiniboia Feeder Main	600	$5,600,000.00	Geotechnical: Toe Armouring 2 Sites	$60,000.00		
				Rehabilitation: CIPP	$2,095,000.00		
				Reinspection			
3	St. Vital Bridge Force Main	500	$4,200,000.00	Geotechnical: Toe Armouring 2 Sites	$60,000.00		
				Rehabilitation: CIPP	$805,000.00		
				Reinspection			
4	Newton Avenue Force Main	350	$7,700,000.00	Geotechnical: Toe Armouring 2 Sites	$60,000.00		
				Replacement	$7,700,000.00		
				Reinspection			
5	Heritage Park Force Main	250	$1,300,000.00	Geotechnical: Toe Armouring 2 Sites	$60,000.00		
				Rehabilitation - NR			
				Reinspection/Pressure Test			$15,000.00
6	Fort Garry – St. Vital Feeder Main	600	$6,100,000.00	Geotechnical: Toe Armouring 1 Site	$30,000.00*		
				Rehabilitation - NR			
				Reinspection/Video			$360,000.00
		Replacement Total:	**$30,300,000.00**	**Geotechnical Total:**	**$375,000.00**	**$0.00**	**$0.00**
				Pipeline Total:	**$12,615,000.00**	**$0.00**	**$375,000.00**
				Combined Total:	**$12,990,000.00**	**$0.00**	**$375,000.00**
						Total Work Program:	**$13,365,000.00**

- * For adjacent interceptor sewer crossing

Based on the estimated replacement costs and projected capital program, this project has saved the City of Winnipeg over $15,000,000 in capital expenditures over cost of asset replacement.

Appendix A

Hydraulic Assessment Technical Memorandums

AECOM
99 Commerce Drive 204 477 5381 tel
Winnipeg, MB, Canada R3P 0Y7 204 284 2040 fax
www.aecom.com

To: File

Date: May 2, 2018

Project #: 60549028 (500)

From: Mike Gaudreau, P. Eng.

cc: Armand Delaurier, City of Winnipeg
Marv McDonald and Adam Braun, AECOM

Memorandum

Subject: **Newton Force Main Crossing the Red River - Hydraulic Analysis**

1. Introduction

A hydraulic analysis of the force main crossing of the Red River from the Hawthorne and Linden Combined Sewer Districts (CSDs) (east of the Red River) to the Newton CSD (west of the Red River) was undertaken to assess operating procedures required during the proposed inspection of the Hawthorne force main crossing.

The crossing to the Newton CSD consists of a 350 mm steel force main from the Linden CSD to the south and a 350 mm HDPE force main from the Hawthorne CSD to the north. The Linden and Hawthorne CSDs represent approximately 400 ha of commercial/residential area. Figure 1 shows the extents of the Hawthorne and Linden CSDs.

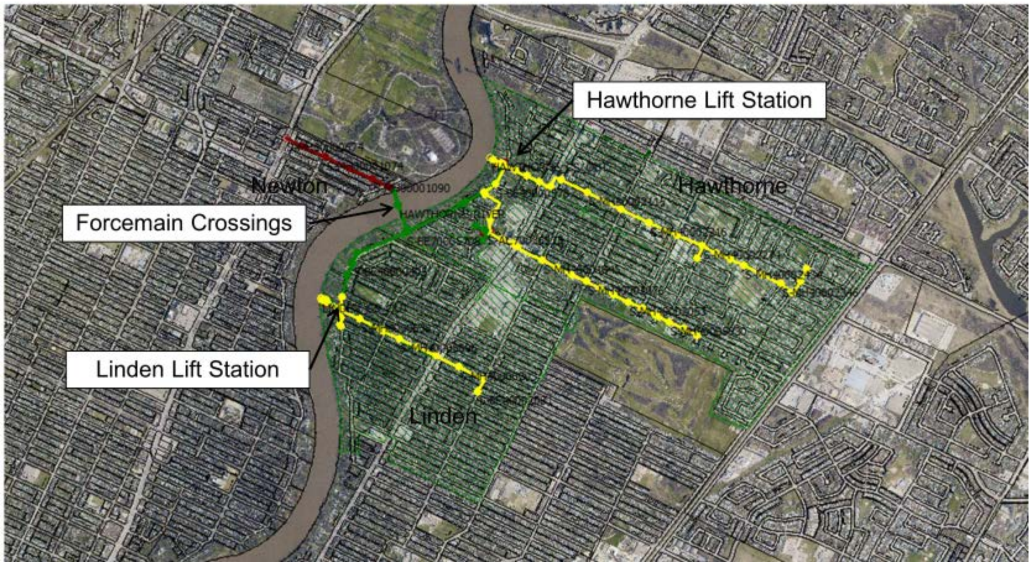


Figure 1 - Hawthorne and Linden CSDs

As part of the *Condition Assessment of High Risk Water and Wastewater River Crossings Phase Two*, the 350mm HDPE force main which currently services the Hawthorne CSD is to be inspected. The 350 mm Steel

Force main was inspected in 2014 under a previous program. The inspection of the 350mm force main will require partial disassembly and plugging of the 350 mm tee in the Hawthorne valve chamber. This would then allow flow from the Hawthorne CSD to be routed through the Linden force main. Installation of the valve between the two chambers in 2014 will greatly simplify flow re-routing. See Record Drawing D-14389 attached to this memorandum.

Previous analysis in 2014 was completed using an InfoWorks CS hydraulic model to develop an operational plan to accommodate the inspection. The model was used to confirm the proposed plan would protect basements against flooding and prevent combined sewer overflows (CSOs) during the inspection. The current plan to inspect the Hawthorne HPDE crossing is a mirror of work completed in 2014.

2. Inspection Plan

The inspection of the 350mm Hawthorne force main will require that the Linden force main accommodate flow from both the Linden and Hawthorne CSDs. It should be noted that prior to the installation of the dedicated 350mm HDPE force main crossing for the Hawthorne CSD, the 350mm steel force main which now serves only the Linden CSD, would likely have served both districts.

Reconnection of the two valve chambers was completed in 2014, and an isolation valve installed between the two chambers. To isolate the Hawthorne force main, a short duration shutdown of the Hawthorne Lift Station will be required to either blind flange the existing tee in the Hawthorne chamber, or temporarily remove the existing tee and installing of a temporary spool piece in its place, and again to restore piping on completion of the work. There will be no requirement to shut down the Linden pumping station.

The hydraulic model was used to determine the maximum allowable shutdown period for the Hawthorne Lift Station. The low basement elevation of 225.4m, which was taken from the Flood Manual’s Sector Action Plan (SAP) for the Hawthorne-Linden-Munroe CSDs, was used as the constraint for the maximum allowable level in both districts.

Following the reconnection of the two force mains, flow from both districts would be routed through the single 350mm force main. Pumps in the Linden and Hawthorne LSs can be potentially operating simultaneously. The hydraulic model was used to verify there would be no adverse effects of this operating condition. This was verified during Fall 2014 inspection activities.

The following operational plan required to complete the inspection of the 350mm HDPE force main was therefore, evaluated using the hydraulic model of the collection systems for the Hawthorne and Linden CSDs:

- Temporary disconnection of Hawthorne river crossing by Installation of 350 mm spool or blind flanging existing tee.
    - Closure of positive gates at Hawthorne FPS to prevent CSOs
    - Turn off pumps at Hawthorne LS
    - Monitor levels at Hawthorne LS to ensure a level of 225.0 m is not exceeded (0.4 m below lowest basement)
- Once reconnection of the two valve chambers is complete.
    - Open gate valve between the two force main chambers
    - Turn on pumps at Hawthorne LS
    - Once levels have subsided below weir height, open positive gates
        - Hawthorne diversion weir elevation: 224.27 m

A test should be completed in advance of the proposed shutdown to confirm the available working window in the Linden and Hawthorne CSDs. This scenario was successfully tested during 2014 shutdowns. In the unlikely event that levels reach 225.0m during the shutdown and the LSs cannot be reactivated, opening the positive gates would allow basements to be protected against flooding but would generate a CSO.

3. Hydraulic Model

As a part of the *North End Collection System Model Update*, AECOM developed and calibrated an InfoWorks CS hydraulic model for the North End System to flow monitoring data collected in 2011. This update included the calibration of dry weather flow (DWF) for catchments contributing to the Main Interceptor which included the Linden and Hawthorne CSDs. The calibrated model should therefore, provide a reasonably accurate representation of the DWF flows from these two districts.

3.1 Flow Generation

During the DWF calibration process, the daily per capita flow based on the equivalent population on the three branches of the North End Collection was determined:

- Main Interceptor: 94 l/person/day
- Northeast Interceptor: 118 l/person/day
- Northwest Interceptor: 140 l/person/day

The groundwater infiltration (GWI) was also determined based on the DWF calibration process:

- Main Interceptor: 5,700 l/ha/day
- Northeast Interceptor: 5,400 l/ha/day
- Northwest Interceptor: 2,200 l/ha/day

Although the Linden and Hawthorne CSDs are connected to the Main Interceptor (via the force main river crossings and the Newton secondary sewer), a flow monitoring point was not available directly downstream of their connection to the Main Interceptor. Due to proximity of these CSDs to the Northeast Interceptor, the Northeast Interceptor flow generation values were assigned to the Linden and Hawthorne CSDs. These values are approximately 20% larger than those assigned to the other districts along the Main Interceptor, and therefore, provide a factor of safety in the analysis.

The diurnal pattern developed for the Northeast Interceptor, and assigned to the Linden and Hawthorne Districts for this analysis is shown in Figure 2. The timing of the diurnal pattern applied in the model calibration was shifted slightly in time to account for the effects of routing in the larger North End model.

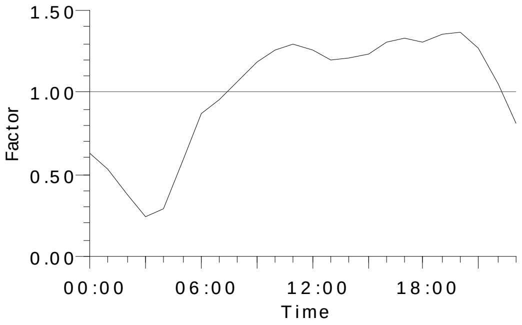


Figure 2 - Diurnal Pattern

3.2 Model Configuration

The hydraulic model included the Linden and Hawthorne CSDs, the force main connection to the interceptor sewer along Newton Avenue, and its connection to the Main Interceptor (Figure 1). The sluice gates at the Linden and Hawthorne FPSs and the pumps at the Linden and Hawthorne LSs, were already included in the model. The operation of these was controlled using the Real Time Control (RTC) facility in InfoWorks CS, and were simultaneously shutdown at the beginning of shutdown period.

To model the operation of the system with only the Hawthorne force main in service, the Linden force main downstream of the of the Linden valve chamber was removed from the model, and the connection between the Linden and Hawthorne valve chambers inserted. Also, in order to properly assess the effects of having both LSs pumping simultaneously down the single force main, pump curves which were provided by the City were entered in the hydraulic model.

3.3 Modelling Results

The model was run to determine the available shutdown window for the reconnection of the Linden and Hawthorne valve chambers and to review the effects of having both LSs share one force main.

To determine the beginning of the shutdown period, the existing DWF through the force mains was reviewed (Figure 3). It was assumed that the work period would begin around 10 pm, at which point the combined flow from both CSDs begins to drop from roughly 57 l/s to a daily minimum of 30 l/s at approximately 4 am.

3.3.1 Reconnection of Linden and Hawthorne Force Mains

To determine the beginning of the shutdown period, the existing DWF through the force mains was reviewed (Figure 3). It was assumed that the work period would begin around 10 pm, at which point the combined flow from both CSDs begins to drop from roughly 57 l/s to a daily minimum of 30 l/s at approximately 4:00 a.m.

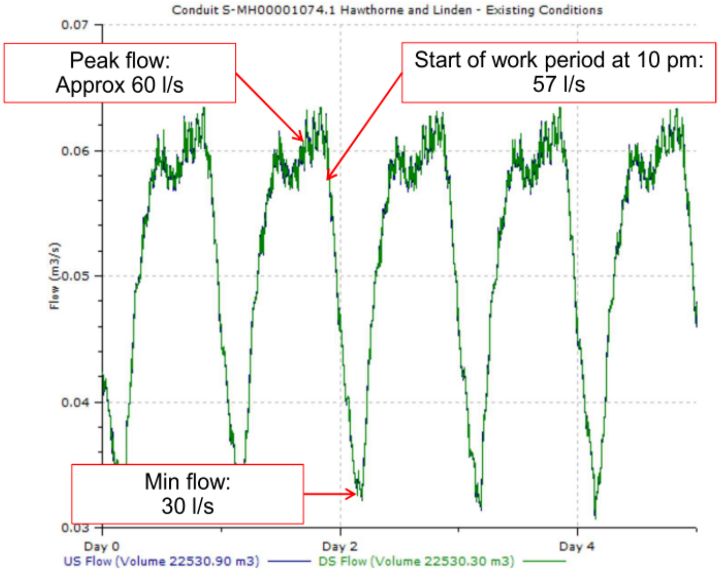


Figure 3 - DWF through FMs

In the simulation, the positive gates at the Hawthorne and Linden outfalls were closed and the pumps at the Hawthorne and Linden LSs were shut off at 10:00 p.m.

Based on the levels at the Hawthorne and Linden LSs, the maximum shutdown windows before levels reach the critical elevation of 225.0 m are as follows (Figure 4):

- Hawthorne CSD: 12 hours
- Linden CSD: 21 hours

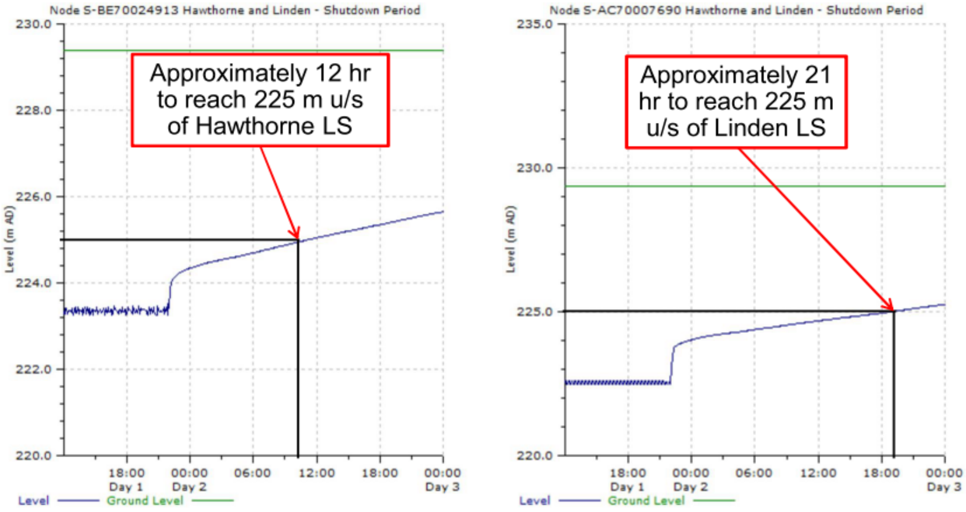


Figure 4 - Shutdown Window

Once the lift stations have been shut down, it takes approximately 1 hour for wastewater in the force mains to drain up to the valve chambers upstream of the river crossing. Therefore, taking into account drain time, the actual working period available for modifications to piping in the valve chambers roughly 11 hours, based on the Hawthorne CSD which has a shorter working window. If necessary, the valve chamber modifications could be completed over two shutdowns, with modifications to the Hawthorne chamber completed first followed by modifications to the Linden chamber.

3.3.2 Hawthorne Force Main Offline

The impact of combining flow from both the Linden and Hawthorne CSDs through the Hawthorne force main crossing the Red River was reviewed in 2014 and described below. As the inside diameter of the steel force main is greater than the HDPE, we expect the impact on the pumps when the Hawthorne force main is shut down and all flow is routed through the Linden force main to be less than those observed during the inspection program in 2014.

The effects on the pumps are relatively insignificant for both the Hawthorne and Linden LSs. It is estimated there would be a rise of 0.3 m in the pumping head at the Hawthorne Lift Station and no significantly increase at the Linden Lift Station since the Linden force main essentially operates as a gravity conduit. Relative to the lift of between 4.3 m and 7.0 m provided by the pumps under existing normal operating conditions, an increase of 0.3 m in the lift required would not materially impact the pumping capacity from either lift station.

Anticipated peak levels at the Linden and Hawthorne diversions weirs was also reviewed with the Linden river crossing taken offline for the inspection. The analysis indicates that operating under these conditions would not result in levels overtopping the overflow weirs:

- Hawthorne diversion weir: 0.63 m below weir crest
- Linden diversion weir: 0.48 m below weir crest

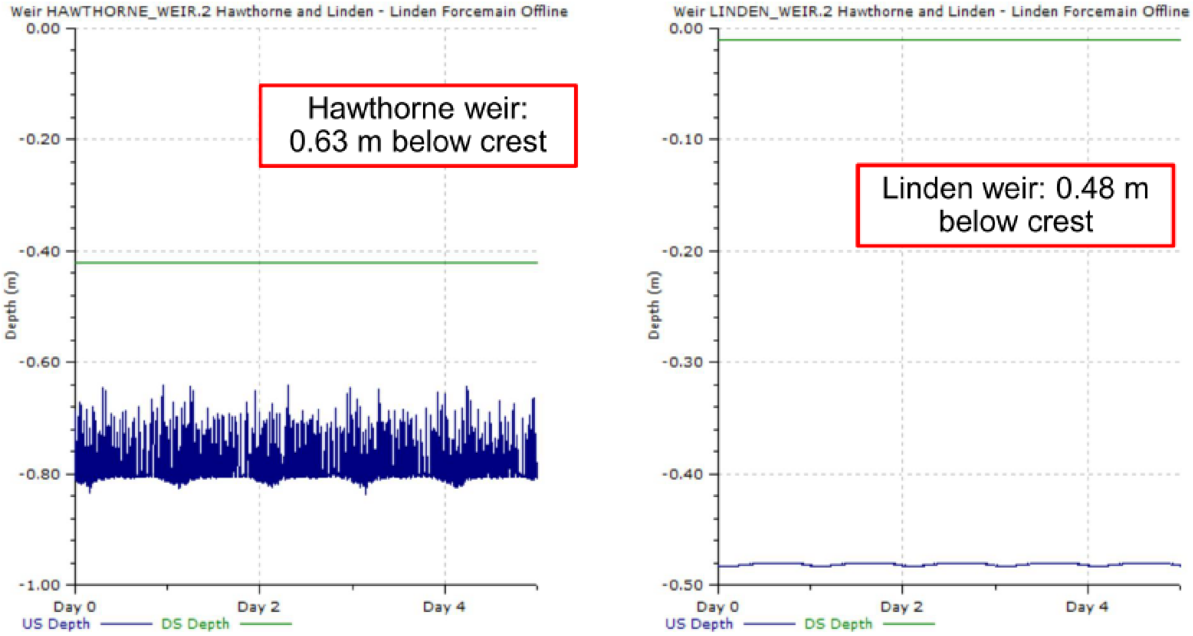


Figure 5 - Depth below Weir Crest

4. Conclusions

Based on the hydraulic review, there is approximately a 12 hour lift station shutdown window, with an available 11 hour working period to install a temporary spool or blind flange at the Linden and Hawthorne valve chambers. During this shutdown window, the sluice gates at the Hawthorne FPSs should be closed to prevent a CSO.

With the force mains reconnected and the LSs are sharing one force main crossing, there is no significant impact on the operation of the pumps in either lift station. Furthermore, once the levels upstream of the LSs subside below weir elevations, the positive gates at the Hawthorne and Linden outfalls can be reopened. The gates would however, once again need to be temporarily closed to complete valve chamber modifications once the inspection has been completed.

5. Recommendations

The recommended operational procedures to accommodate inspection of the 350mm HDPE force main crossing for the Hawthorne CSD are outlined in Table 1.

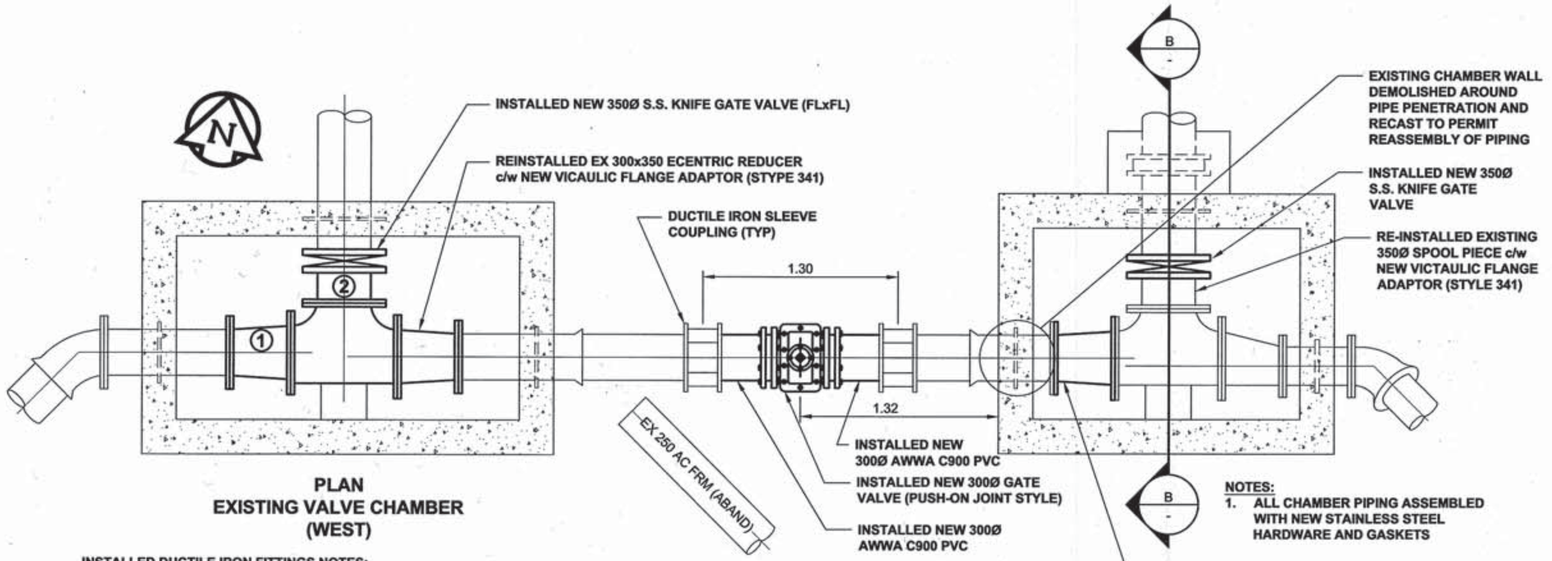
Table 1 - Inspection Operational Procedures

Task	Location	Time/Notes
1. **Close Sluice Gate**	Hawthorne FPS/Outfall Hawthorne Ave and Kildonan Dr Linden FPS/Outfall Linden Ave and Kildonan Dr	Any time before shutdown of pumps at Lift Stations.
2. **Shutdown of pumps**	Hawthorne Lift Station Hawthorne Ave and Kildonan Dr	Pumps to be turned off at 10 pm.
3. **Monitor levels**	Hawthorne Lift Station	Maximum allowable level of 225m. In unlikely event levels reach 225m, open positive gate to protect basements.
4. **Reconnection of Hawthorne and Linden FMs**	Hawthorne and Linden valve chambers. Kildonan Dr between Larchdale Ave and Rossmere Ave	Commence work at approximately 11 pm, allowing one hour for the force main to drain up to the valve chambers. Shutdown 1:Installation of 350 mm spool or blind flange on tee in Hawthorne VC Open valve between Linden and Hawthorne Chambers
5. **Turn pumps back on**	Hawthorne Lift Station Hawthorne Ave and Kildonan Dr	Ensure downstream infrastructure in valve chambers is operational, and turn pumps back on before 10 am.
6. **Reopen sluice gates**	Hawthorne FPS Hawthorne Ave and Kildonan Dr	Once levels have dropped below weir elevation by 0.2 m, open positive gate. Weir elevation is 224.27 m

Task	Location	Time/Notes
7. Complete inspection of 350 mm Hawthorne FM	Hawthorne Valve Chamber Hawthorne Ave and Kildonan Dr	Any time after installation of 350 mm spool or blind flange.
8. Reconnection of Hawthorne FM	Hawthorne Valve Chamber Hawthorne Ave and Kildonan Dr	Repeat Tasks 1 through 6, except at Task 4, re instatement of the tee in the Hawthorne valve chamber.

Mike Gaudreau, P. Eng.
Municipal Engineer
Conveyance
MG/pab

D SIZE 22" x 34" (559mm x 864mm)



PLAN
EXISTING VALVE CHAMBER
(WEST)

INSTALLED DUCTILE IRON FITTINGS NOTES:

1. NEW 300x350 AWWA C110 CONCENTRIC REDUCER (FLxFL) c/w VICTAULIC FLANGE ADAPTOR (STYLE 341)
2. NEW 350 (FL x VICTAULIC) AWWA C151 DUCTILE IRON PIPE SPOOL PIECE (241mm LONG) c/w VICTAULIC FLANGE ADAPTOR (STYLE 341).

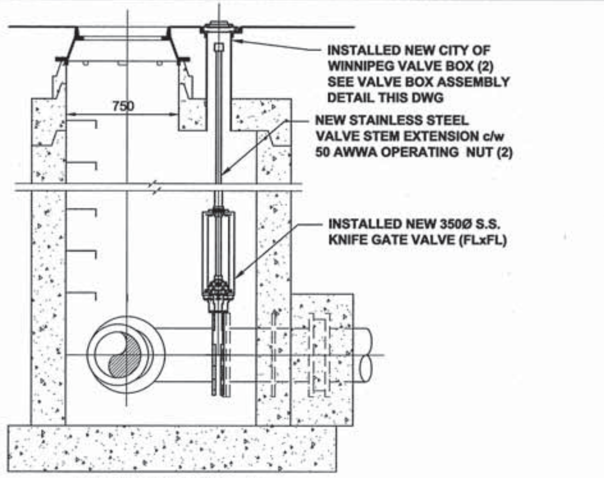
CHAMBER MODIFICATIONS

Scale 1:25

NOTES:

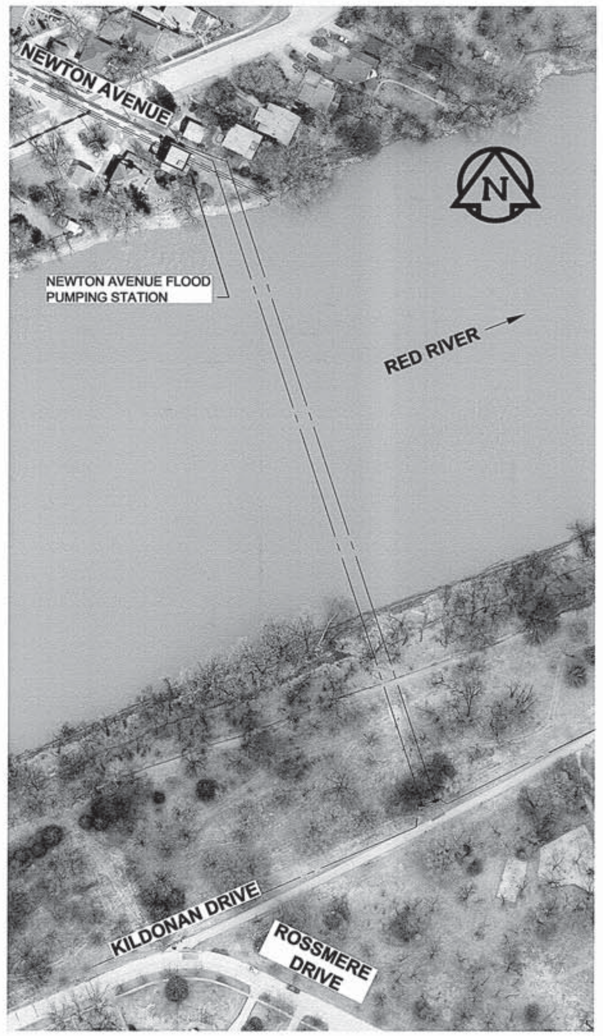
1. ALL CHAMBER PIPING ASSEMBLED WITH NEW STAINLESS STEEL HARDWARE AND GASKETS

PLAN
EXISTING VALVE CHAMBER
(EAST)



SECTION B - B

Scale 1:25



SITE PLAN

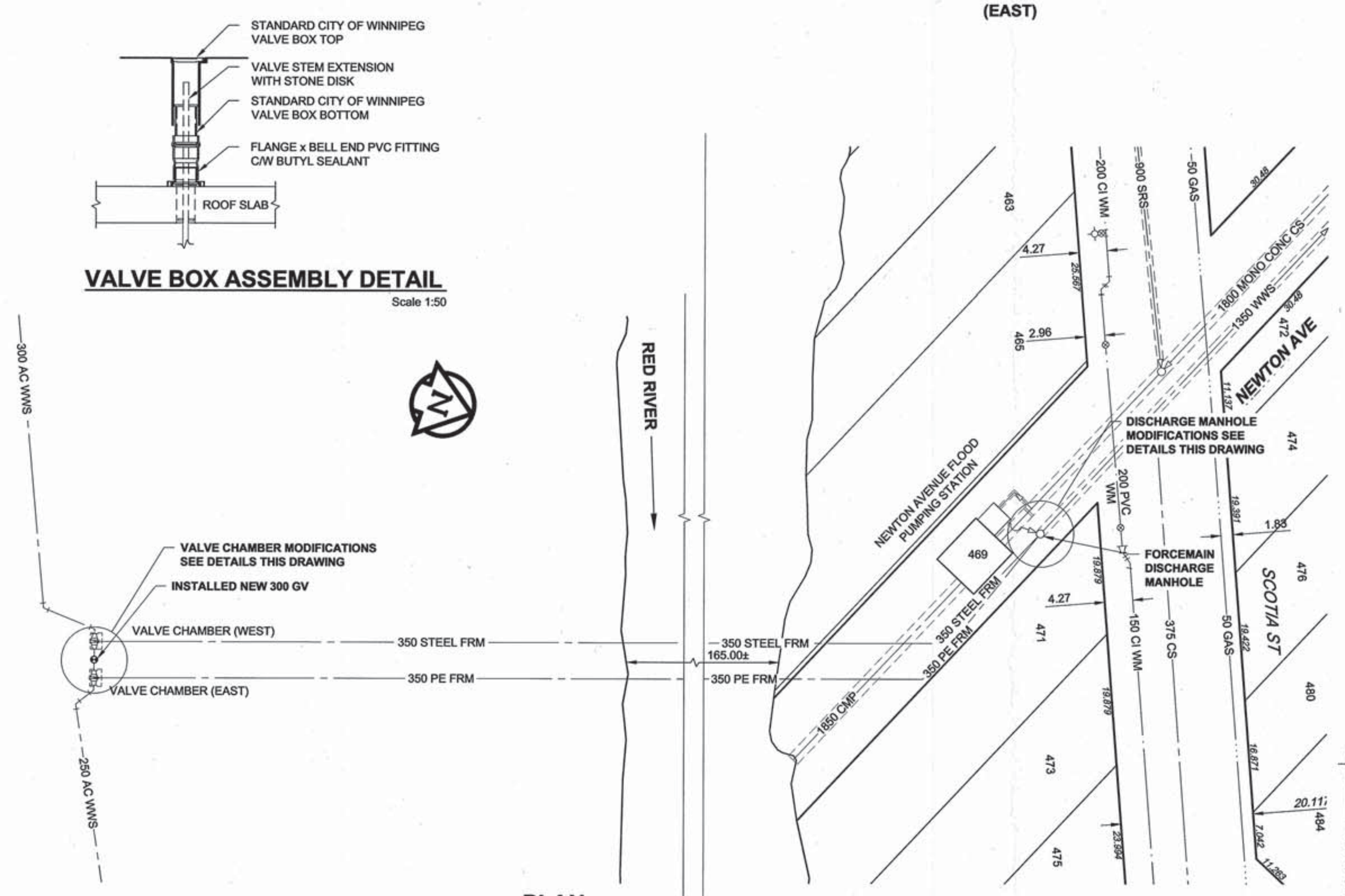
Scale 1:1500

0 37.5 75 m

MATERIAL LIST

ITEM	MANUFACTURER	MATERIAL SPECIFICATION	SPECIAL FEATURES
WATER WORKS			
300 PVC PIPE (150 to 300)	IPEX INC.	AWWA C900 CSA B137.3	"BLUE BRUTE" DR-18
300 GATE VALVES (150 to 300)	CLOW CANADA	AWWA C509 MODEL F-6112	RESILIENT WEDGE GATE VALVE CW TO OPEN (RED TOP)
IRON COUPLINGS (STRAIGHT or TRANSITION) (300)	ROBAR INDUSTRIES LTD.	AWWA C219 AWWA C213 ASTM A536 GRADE 65-45-12 MODEL "1506"	FBE-COATED D.I. SLEEVE & END RINGS TYPE 304 S.S. NUTS, BOLTS, WASHERS AND DI-ELECTRIC WASHERS
VALVE BOX UPPER CASING	W.D. VALVE BOXES LTD.	C.I. - ASTM A-48 CLASS 30B CONFORMS TO AP-001	CASING MARKED "WDVB", "AIL" & "WD46" LID MARKED "WDVB" & "S"
150 PVC PIPE (150 to 300)	IPEX INC.	AWWA C900 CSA B137.3	IPS, NON-PRESSURE
150 SOLVENT WELD PVC FITTING	IPEX INC.	CSA B137	
STAINLESS STEEL VALVE STEM EXTENSION	N/A	ASTM A312	50 MM AWWA OPERATING NUT
350 STAINLESS STEEL KNIFE GATE VALVE	DEZURIK	AWWA C520	BODY, GATE, BOLTS 316 STAINLESS STEEL
300 VIC-FLANGE ADAPTER	VICTAULIC	AWWA C606	STYLE 341
350 VIC-FLANGE ADAPTER	VICTAULIC	AWWA C606	STYLE 341
350 DUCTILE IRON TEE (FLANGED)	METALFIT INC.	AWWA C111, AWWA C104, AWWA C210	EPOXY COATED CEMENT MORTAR LINED
300 X 350 DUCTILE IRON CONCENTRIC REDUCER (FLANGED)	METALFIT INC.	AWWA C111, AWWA C104, AWWA C210	EPOXY COATED CEMENT MORTAR LINED
350 DUCTILE IRON SPOOL PIECE (FLANGE X VICTAULIC)	METALFIT INC.	AWWA C115, AWWA C606, AWWA C104, AWWA C210	EPOXY COATED CEMENT MORTAR LINED
350 DUCTILE IRON 90 DEGREE BEND (FLANGED)	METALFIT INC.	AWWA C111, AWWA C104, AWWA C210	EPOXY COATED CEMENT MORTAR LINED
350 HDPE PIPE	WL PLASTICS	AWWA C906, ASTM F714	IPS, DR 17, PE3608 RESIN
ANODES - ZINC	INTERPROVINCIAL CORROSION CONTROL COMPANY LIMITED	ASTM B418 - TYPE II	ZINC WEIGHT 10.9 kg
PETROLATUM TAPE CORROSION PROTECTION	DENSO NORTH AMERICA	AWWA C2017	PROFILING MASTIC PETROLATUM TAPE OUTER WRAP

CONTRACTOR:	TRI-CORE PROJECTS MANITOBA LTD.
CONSTRUCTION PERIOD:	AUGUST to NOVEMBER 2014
FIELD BOOK NO.:	N/A



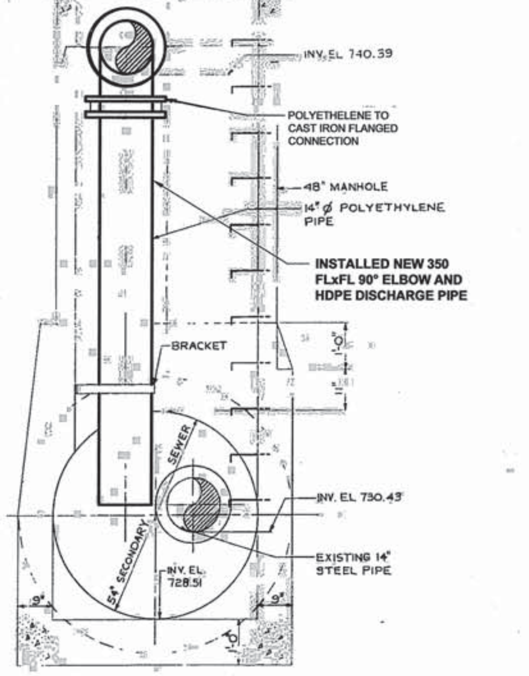
VALVE BOX ASSEMBLY DETAIL

Scale 1:50

PLOT: 3/6/2017 2:35:33 PM

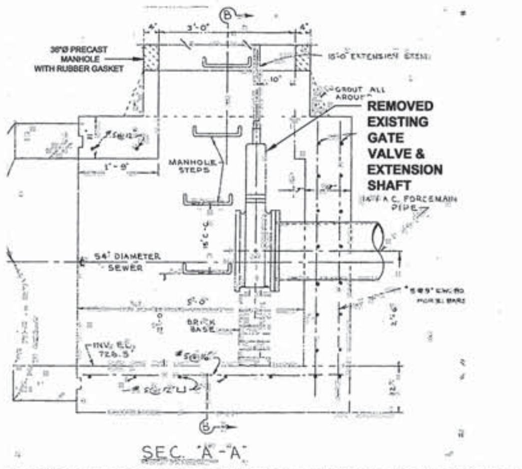
PLAN

Scale 1:500



DISCHARGE MANHOLE MODIFICATIONS

Scale 1:25



DISCHARGE MANHOLE MODIFICATIONS

Scale 1:25

KEY PLAN

APEGM
Certificate of Authorization
AECOM Canada Ltd.
No. 4671 Date: Mar 14/17

ALL VALVES TO BE COUNTER-CLOCKWISE TO CLOSE (RED OPERATING NUT)

METRIC
WHOLE NUMBERS INDICATE MILLIMETRES
DECIMALIZED NUMBERS INDICATE METRES

RECORD DRAWING

BID OPPORTUNITY NO. 457-2014

EXISTING	LEGEND - PLAN	NEW	EXISTING	LEGEND - PLAN	NEW	EXISTING	LEGEND - PROFILE	NEW
150 WM	WATERMAIN	150 WM	♂	CURB STOP	●	150 WM	WATERMAIN	150 WM
◇	HYDRANT	◆	—·▷·—	REDUCER	—▶—	+	HYDRANT	+
⊗	VALVE	●	—·✕·—	COUPLING	—✕—	X	VALVE	X
300 LDS	LAND DRAINAGE SEWER	300 LDS	⤳	ANODE	⤳	300 LDS	LAND DRAINAGE SEWER	300 LDS
250 WWS	WASTE WATER SEWER	250 WWS	— — —	HYDRO		250 WWS	WASTE WATER SEWER	250 WWS
○	MANHOLE	●	—··—	MTS		—·—	PAVEMENT CROWN	—·—
□	CATCH BASIN	■	—···—	GAS		—⊖—	N/W PROPERTY LINE	
△	CURB INLET	▲	⊕	TESTHOLE		—⊖—	S/E PROPERTY LINE	
[-------]	CULVERT	[-------]	•—•	LAMP STANDARD		—⊟—	N/W GUTTER	
—⇥	PIPE ABANDONMENTS	—⇥	⊙	TREE		— —⊖— —	S/E GUTTER	
⌖	SURVEY BAR							

LOCATION APPROVED
UNDERGROUND STRUCTURES

SUPR. U/G STRUCTURES COMMITTEE
DATE

NOTE:
LOCATION OF UNDERGROUND STRUCTURES AS SHOWN ARE BASED ON THE BEST INFORMATION AVAILABLE. BUT NO GUARANTEE IS GIVEN THAT ALL EXISTING UTILITIES ARE SHOWN OR THAT THE GIVEN LOCATIONS ARE EXACT. CONFIRMATION OF EXISTENCE AND EXACT LOCATION OF ALL SERVICES MUST BE OBTAINED FROM THE INDIVIDUAL UTILITIES BEFORE PROCEEDING WITH CONSTRUCTION.

NO.	REVISIONS	YY/MM/DD	BY
0	ISSUED FOR RECORD	16/10/20	WJd

AECOM

DESIGNED BY ADB
DRAWN BY ADL
HOR. SCALE AS NOTED
VERT. SCALE N/A
DATE
CHECKED BY
APPROVED BY
RELEASED FOR CONSTRUCTION
DATE

PROFESSIONAL'S SEAL
A.D. BRAUN
Member
24453
Mar 14/17

CONSULTANT DRAWING NO.
60270487-SHT-00-0000-C-0001

Winnipeg
THE CITY OF WINNIPEG
WATER AND WASTE DEPARTMENT

PROVISION OF PIPELINE ACCESS MODIFICATIONS, CLEANING AND SUPPORT SERVICES FOR WATER MAIN RIVER CROSSING INSPECTIONS
SITE 3A
NEWTON AVENUE FORCE MAIN

SHEET 2 OF 2
CITY DRAWING NUMBER
D-14389
REV 0

AECOM FILE NAME: 60270487-SHT-00-0000-C-0001.dwg Saved By: denoudenw

AECOM REVIEW DRFT CHK

AECOM
99 Commerce Drive
Winnipeg, MB, Canada R3P 0Y7
204 477 5381 tel
204 284 2040 fax
www.aecom.com

To: File

Date: June 13, 2018
Project #: 60549028 (500)
From: Mike Gaudreau, P. Eng.

cc: Armand Delaurier, City of Winnipeg
Marv McDonald and Adam Braun, AECOM

Memorandum

Subject: **Heritage Park Force Main Crossing the Red River - Hydraulic Analysis**

1. Introduction

A hydraulic analysis of the Heritage Park waste water district (separated) was undertaken to assess the Heritage Park Lift Station (LS) shutdown window and monitoring procedures for the proposed sampling and pressure testing of the 250 mm force main.

The crossing of Sturgeon Creek consists of a 250 mm PVC force main located just north of the Ness Avenue bridge between Valley View Drive and Alcott Street. The force main connects the Heritage Park Lift Station (LS) located on the east side of Sturgeon Creek to a 525 mm gravity main located at Ness Avenue and School Road.

The force main services the entire waste water district, consisting of approximately 130 ha of mostly residential development. Figure 1 shows the extent of Heritage Park waste water district.

The analysis was undertaken utilizing both the *CoW Regional Baseline 2013* InfoWorks CS hydraulic model, and review of SCADA information with respect to pump operations at the Heritage Park LS. It was determined that the Heritage Park LS can be taken offline during periods of low dry weather flow (DWF) for a period of up to 7 hours. To achieve this, the following tasks are required:

1. Closure of the sluice gate located in manhole immediately east of S-MH20000040 at the Heritage Park LS
2. Monitoring of waste water levels to ensure they do not exceed the overflow elevation at the Heritage Park LS of 231.3 m

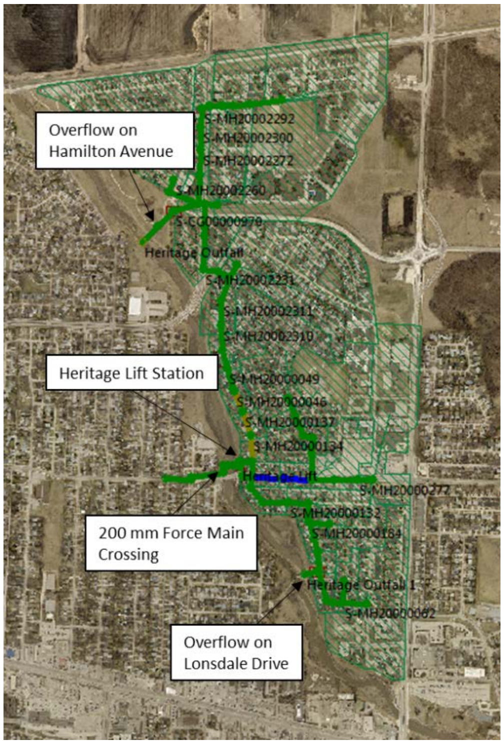


Figure 1 – Heritage Park Waste Water District

2. Inspection Plan

The inspection of the 250 mm PVC Heritage Park LS force main will consist of sampling the 1989 force main, and the completion of a low head pressure test. An operational plan was developed for isolation of Heritage Park LS in order to completion the inspection program.

The operation plan was developed to ensure levels within the waste water sewer (WWS) system remain below both basement elevations and overflow into Sturgeon Creek. Our review of the system indicates the following critical elevations:

1. Hamilton Avenue - S-MH70061235
    i. Overflow to Sturgeon Creek
    ii. Elevation: 233.0 m (City's GIS Data)

2. Heritage Park LS - S-MH20000040
    i. Overflow to Sturgeon Creek
    ii. Elevation: 231.3 m (City's GIS Data)

3. Lonsdale Drive - S-MH20000107
    i. Overflow to Sturgeon Creek
    ii. Elevation: 232.2 m (City's GIS Data)

4. Critical basement - S-MH20000048
    i. 3141 Ness Avenue
    ii. Rim elevation: 235.15 m (City GIS)
    iii. Assumed basement elevation: 232.75 m

The lowest of the critical elevation is associated with the overflow at the Heritage Park LS, at an elevation of approximately 231.3 m. Assuming 0.6 m of freeboard, the maximum allowable elevation that would be allowed in the WWS system is 230.7 m. However, as the Heritage Park LS is at the low end of the system, it is anticipated that this elevation would limit the allowable storage in the upstream system, and therefore prevent completion of the inspection tasks.

Our review of the system indicates that the lowest basement to be at 3141 Ness Avenue based on the lowest rim elevation of 232.75 m from the City's GIS (rim elevation less 2.4 m is assumed to be basement elevation). As such, we would anticipate that basement flood risk is not significantly increased until levels in the system reach 232.15 m.

However, as overflows are typically developed for basement flood protection, we would recommend limiting the backup elevation to the elevation of the Heritage Park LS overflow of 231.3 m, and to not include the 0.6 m freeboard. As shown in Figure 2, this would increase the available storage in the main sewer element (from the InfoWorks CS model) by approximately 40%, or 50 m^3.

In order to limit the risk of a sanitary sewer overflow (SSO) into Sturgeon Creek, the gate at the control manhole of the Heritage Park LS overflow should be closed. Should levels exceed the 231.3 m elevation, the Heritage Park LS should be reinstated to draw down levels. If the Heritage Park LS cannot be reinstated, opening the sluice gate to relieve the system and protect against basement flooding would be required. The associated storage and estimated time to fill the storage beyond the Heritage Park LS critical elevations for the other critical elevations is as follows:

- Hamilton Avenue - S-MH70061235
    - Overflow to Sturgeon Creek
    - Critical elevation: 233.0 m – 0.6 m Freeboard = 232.4m
    - Additional storage = 300 m^3 - 180 m^3 = 120 m^3
    - Time to fill additional storage = 4 hours

- Lonsdale Drive - S-MH20000107
    - Overflow to Sturgeon Creek
    - Critical elevation: 232.2 m – 0.6 m Freeboard = 231.6 m
    - Additional storage = 210 m^3 - 180 m^3 = 30 m^3
    - Time to fill additional storage = 2 hours

- Critical basement - S-MH20000048
    - 3141 Ness Avenue
    - Critical elevation: 232.75 m – 0.6 m Freeboard = 232.15 m
    - Additional storage: 265 m^3 - 180 m^3 = 85 m^3
    - Time to fill additional storage = 3 hours

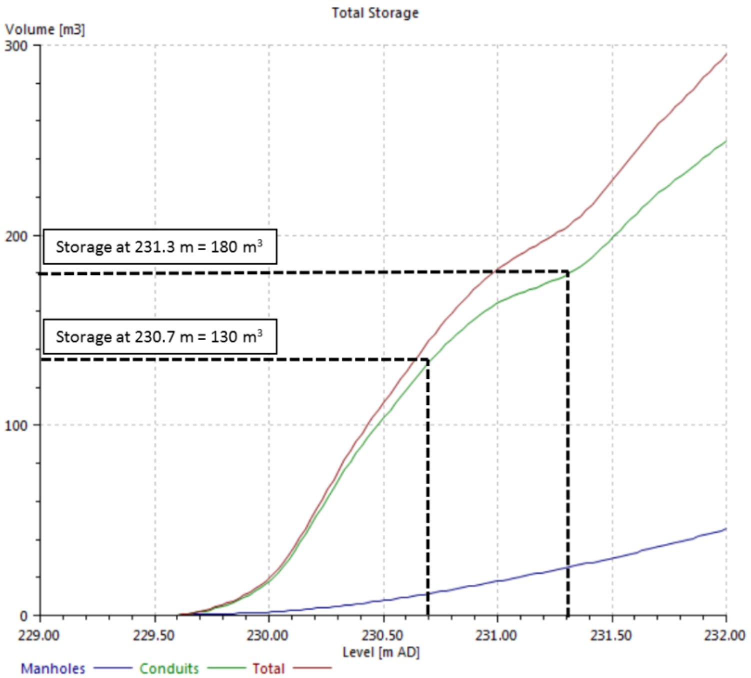


Figure 2 – Available Storage in Main Sewer Elements (from InfoWorks CS model)

3. Hydraulic Analysis

The hydraulic analysis to determine the period of time the Heritage LS could be offline was completed utilizing both pump information from the City's SCADA and a review of the shutdown in the City's *CoW Regional Baseline 2013* InfoWorks CS model.

3.1 Pump Configuration

Based on information provided by the City, the pump configuration at the Heritage LS is as follows:

- Three pumps, with the following capacities
  - Pump 1: 57 l/s (750 IGPM)
  - Pump 2: 72 l/s (950 IGPM)
  - Pump 3: 144 l/s (1900 IGPM)
- City SCADA indicates Pump 2 and Pump 3 are cycled between duty and during DWF periods
- City SCADA suggests Pump 1 does not run frequently during DWF periods

The pump capacities were updated in the InfoWorks CS model.

3.2 Pump Operation Review

The City's SCADA data at the Heritage Park LS was compiled for the week of January 21 to January 2017, 2018, and the pump on/off and cumulative pump volumes are provided in Appendix A.

The pump on and off figures indicate the pumps run more frequently from 7 am to midnight, suggesting the low flow period at the Heritage Park LS is between midnight and 7 am.

Similarly, reviewing the cumulative pump volumes shows the rate of volume increase (i.e. flow) is least during the early morning period (between midnight and 7 am).

Based on the cumulative pump volume curves in Appendix A, the system could be shut down from approximately midnight to 7 am before the 180 m^3 storage capacity is exceeded, resulting in levels in excess of the critical elevation of 231.3 m (Figure 2).

3.3 Hydraulic Model Review

The *CoW Regional Baseline 2013* InfoWorks CS model was utilized to confirm the findings from the pump operation review as discussed in the following sections.

3.3.1 Flow Generation

The flow generation from the hydraulic model was review, and summarized as follows:

- Population
    - Model population is approximately 5,700 ppl
    - Suggests a mix of single family and multi-family residential
- Flow generation
    - Model assumes a 258 l/capita/day
    - Daily flow volume from model is 1,800 m^3/day
    - SCADA data suggests flow volume is approximately 1,100 m^3/day
- Diurnal pattern
    - As shown in Figure 3, the diurnal appears to be at its lowest at 2 am and peaking near 9 am, which is fairly consistent with the Heritage Park LS SCADA data, where the low DWF is from 12 am to 7 am

Based on the flow generation review in the InfoWorks CS model and the review of the Heritage Park LS SCADA data, it appears that the InfoWorks CS model is conservative in flow generation and generally maintains the same diurnal pattern.

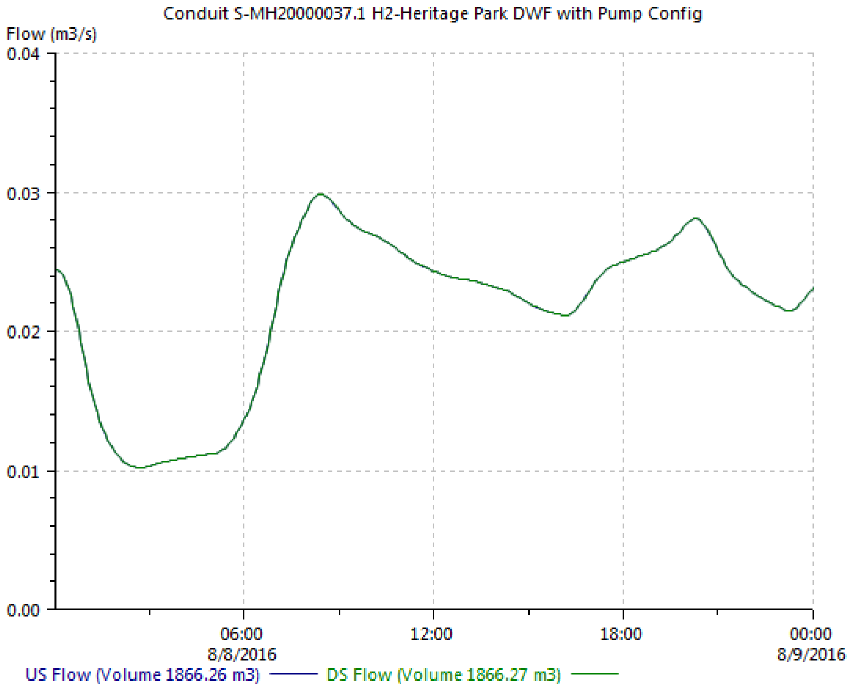


Figure 3 – InfoWorks CS Diurnal Pattern

3.4 Model Results

The model was configured to simulate a Heritage Park LS shutdown to confirm the allowable time the LS could be taken offline before levels exceed the critical elevation of 231.3 m.

As shown in Figure 4, the InfoWorks CS model suggests that the Heritage Park LS can be taken offline for approximately 5 hours (assuming the shutdown begins at 2 am) before levels exceed the critical elevation. This is slightly less of a shutdown window then suggested by the Heritage Park LS SCADA data, which is expected as the daily flow volumes are greater in the model.

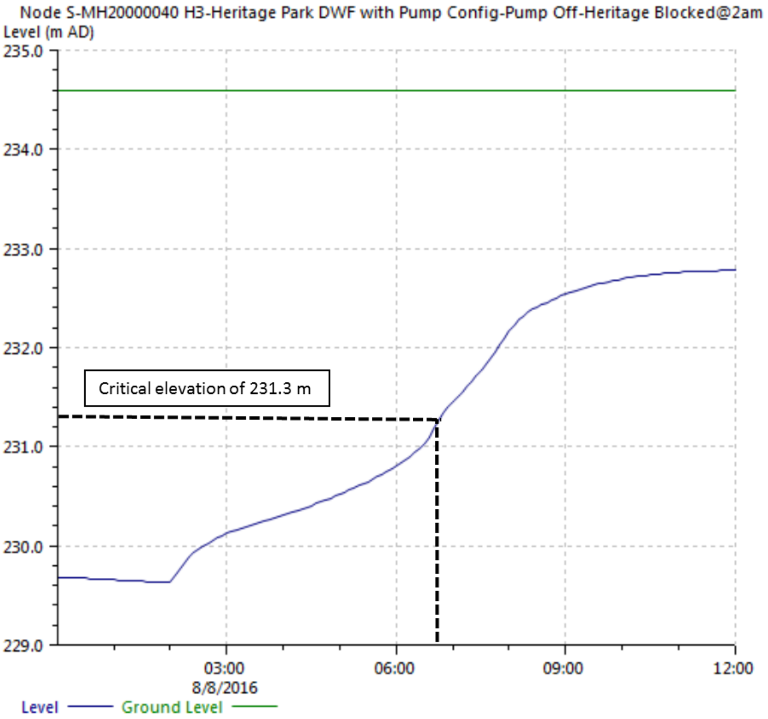


Figure 4 – Levels at Heritage Park LS during Shutdown

4. Conclusions

Our review of the Heritage Park WWS system indicates that the critical elevation is at the overflow at the Heritage Park LS, at an elevation of 231.3 m. The InfoWorks CS hydraulic model suggests that there is approximately 180 m^3 of available storage in the WWS system until levels exceed the critical elevation.

A review of the SCADA data at the Heritage Park LS and results from the InfoWorks CS hydraulic model indicate that there is a window between 5 to 7 hours where the Heritage LS can be taken offline to undertake the inspection program, assuming the shutdown occurs overnight.

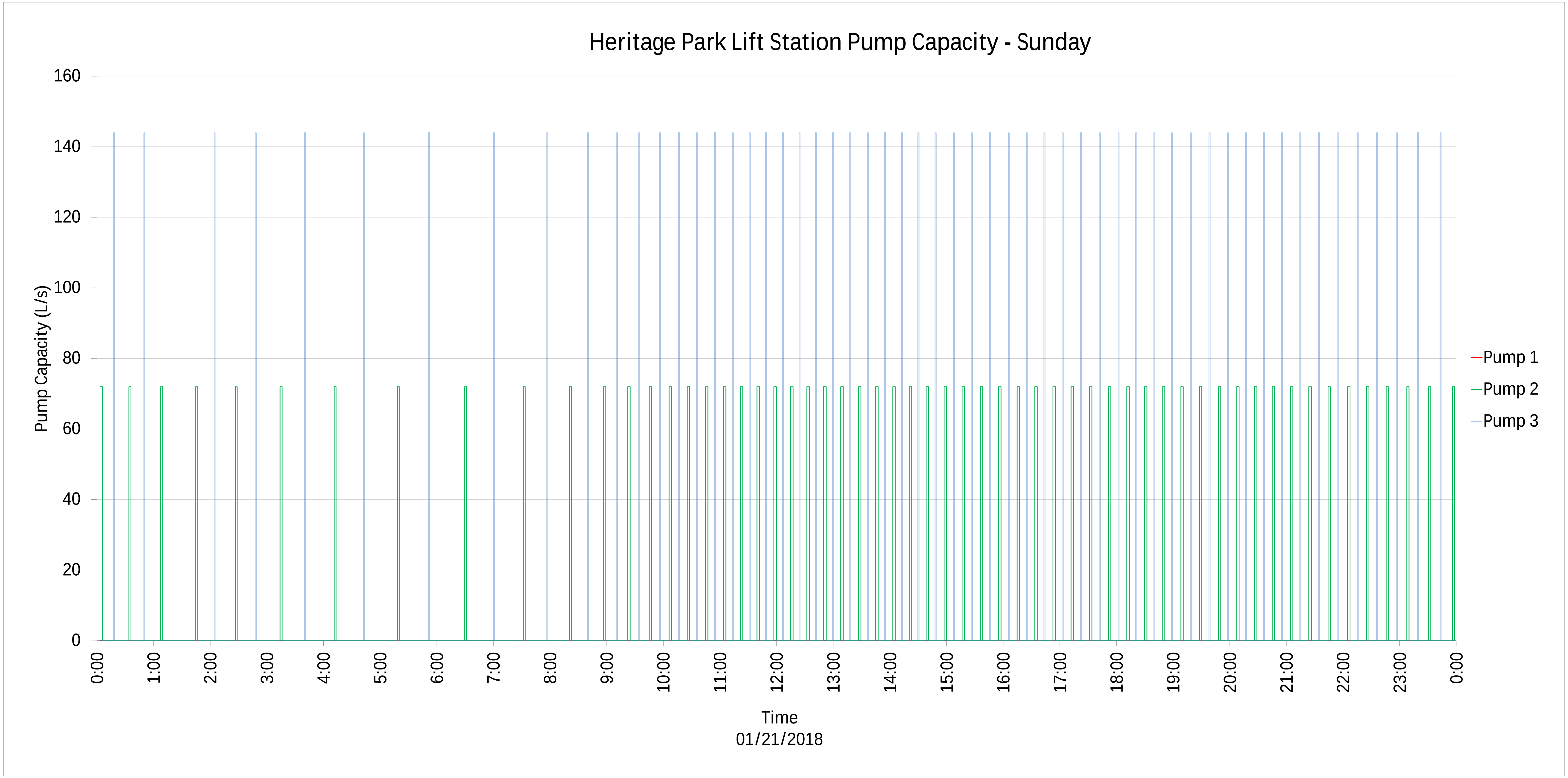
Table 1 provides a summary of the operational procedures required to complete the inspection program.

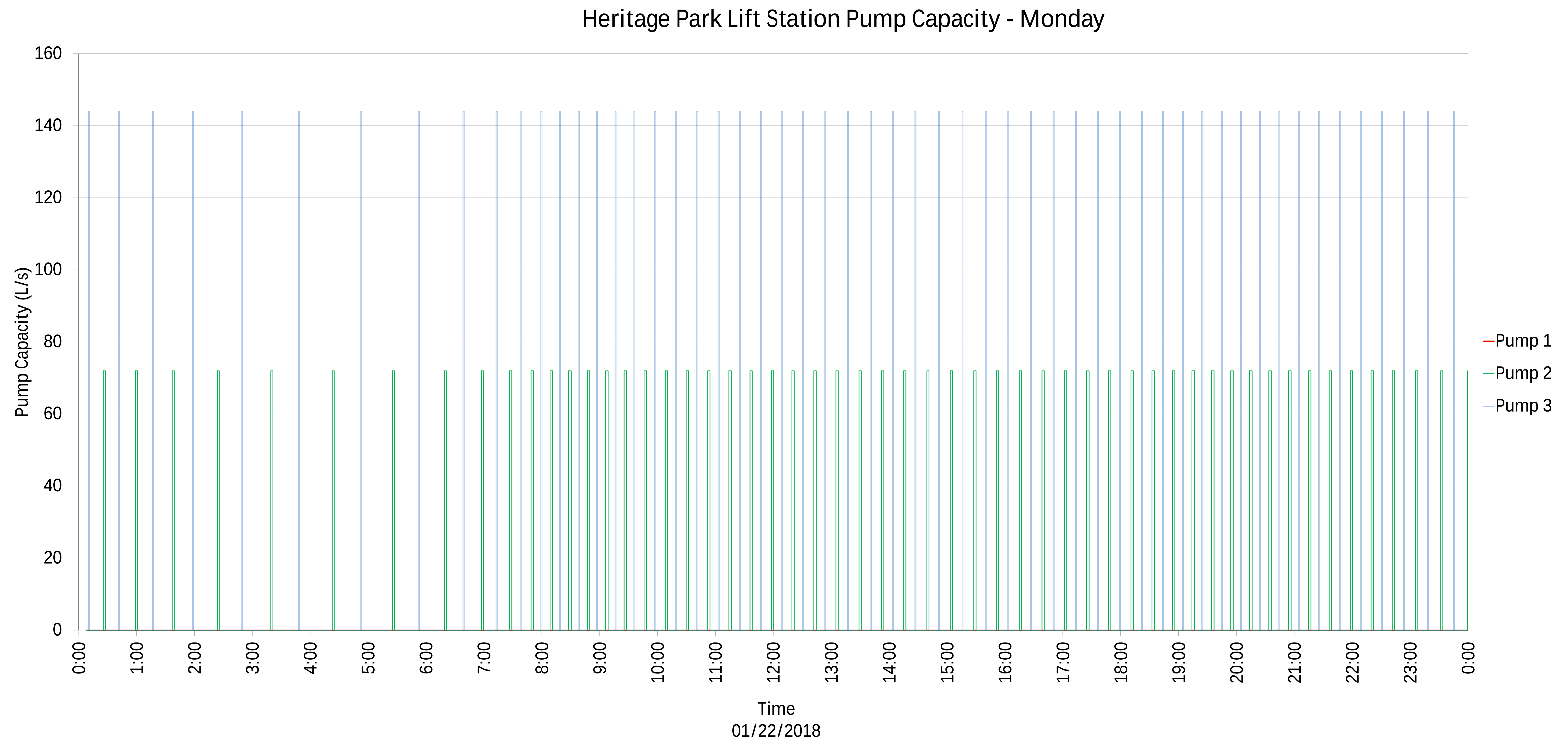
Table 1 - Inspection Operational Procedures

Task	Location	Time/Notes
1. Close Sluice Gate	Heritage Park Lift Station manhole immediately east of S-MH20000040	Any time before shutdown of pumps at Lift Station.
2. Shutdown pumps	Heritage Park Lift Station	Pumps to be turned off at midnight.
3. Monitor levels	Heritage Park Lift Station	Maximum allowable level is 231.3 m. If level is breached, turn pump station on. If not possible, open sluice gate to protect basements.
4. Complete sampling or pressure test	Heritage Park Lift Station force main piping	Undertake tasks sequentially, ensuring sufficient time to complete operation before levels exceed 231.3 m
5. Turn pumps back	Heritage Park Lift Station	Ensure pumps are turned back on before 7 am at the latest.
6. Reopen sluice gates	Heritage Park Lift Station manhole immediately east of S-MH20000040	Once levels at the Heritage Park Lift Station have dropped below 231.3 m. If levels slightly exceed 231.3 m, pumping out residual overflow from the manhole may be required prior to opening sluice gate.

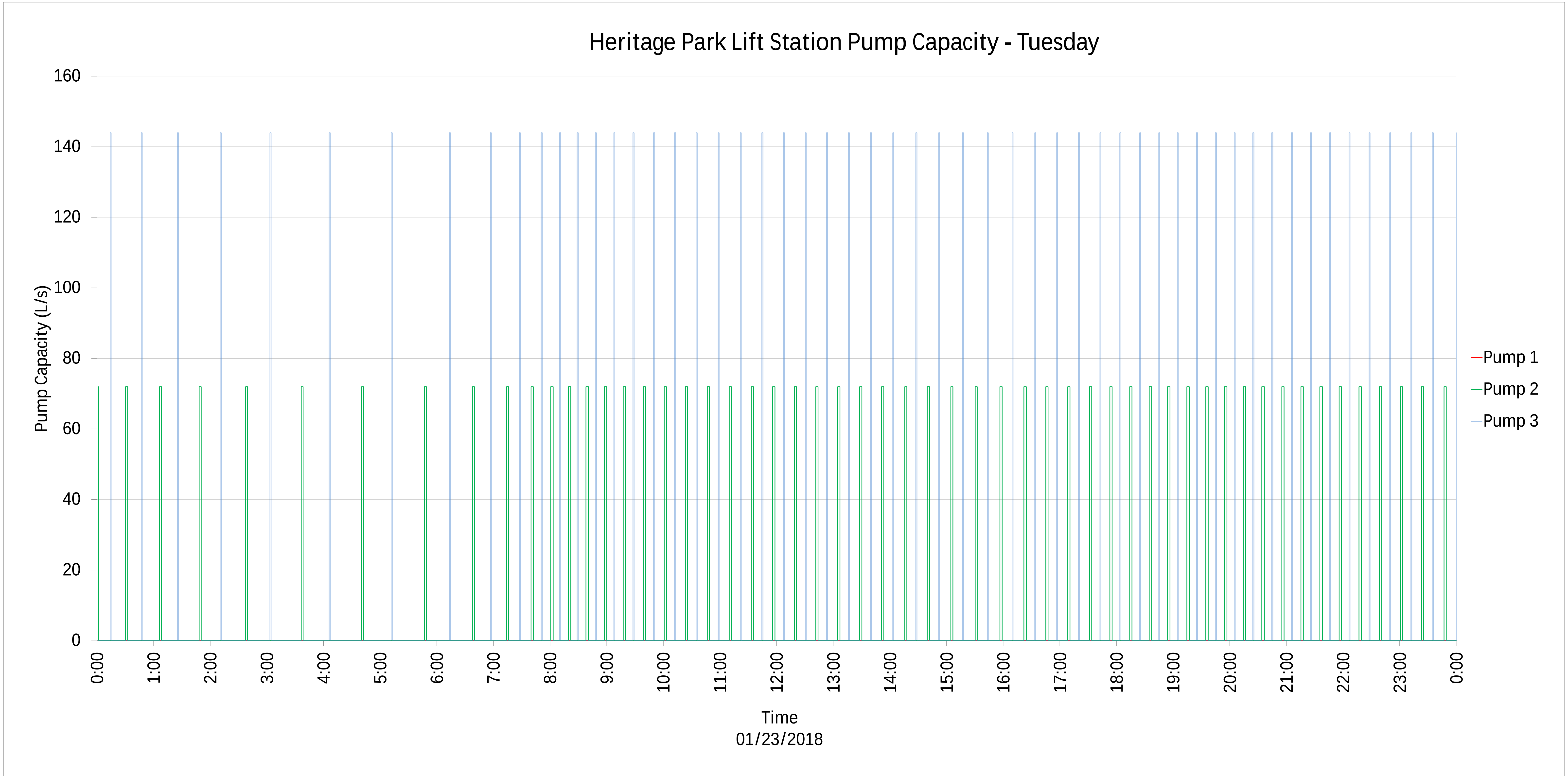
Mike Gaudreau, P. Eng.
Municipal Engineer
Conveyance
MG/gms

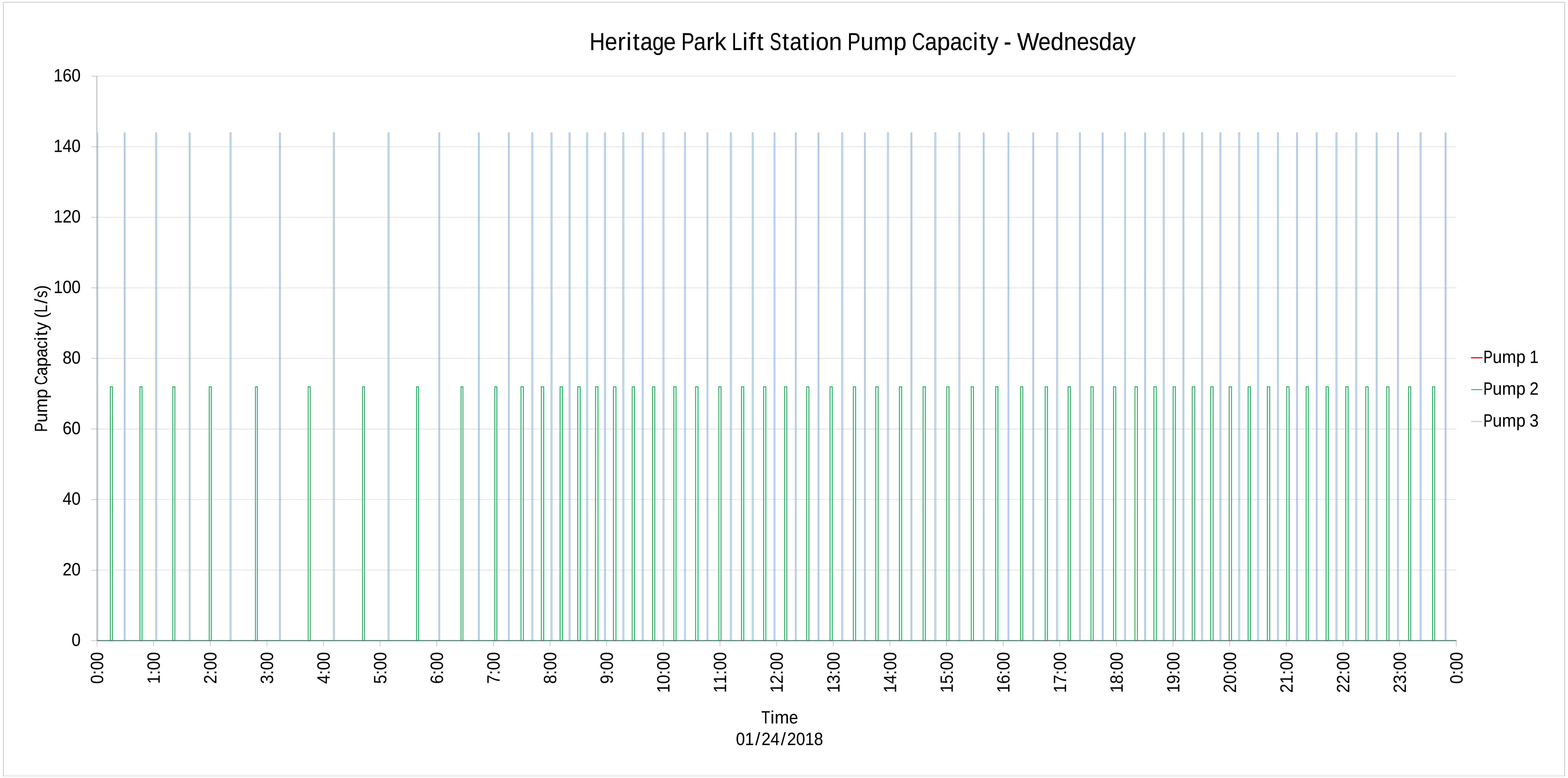
Appendix A

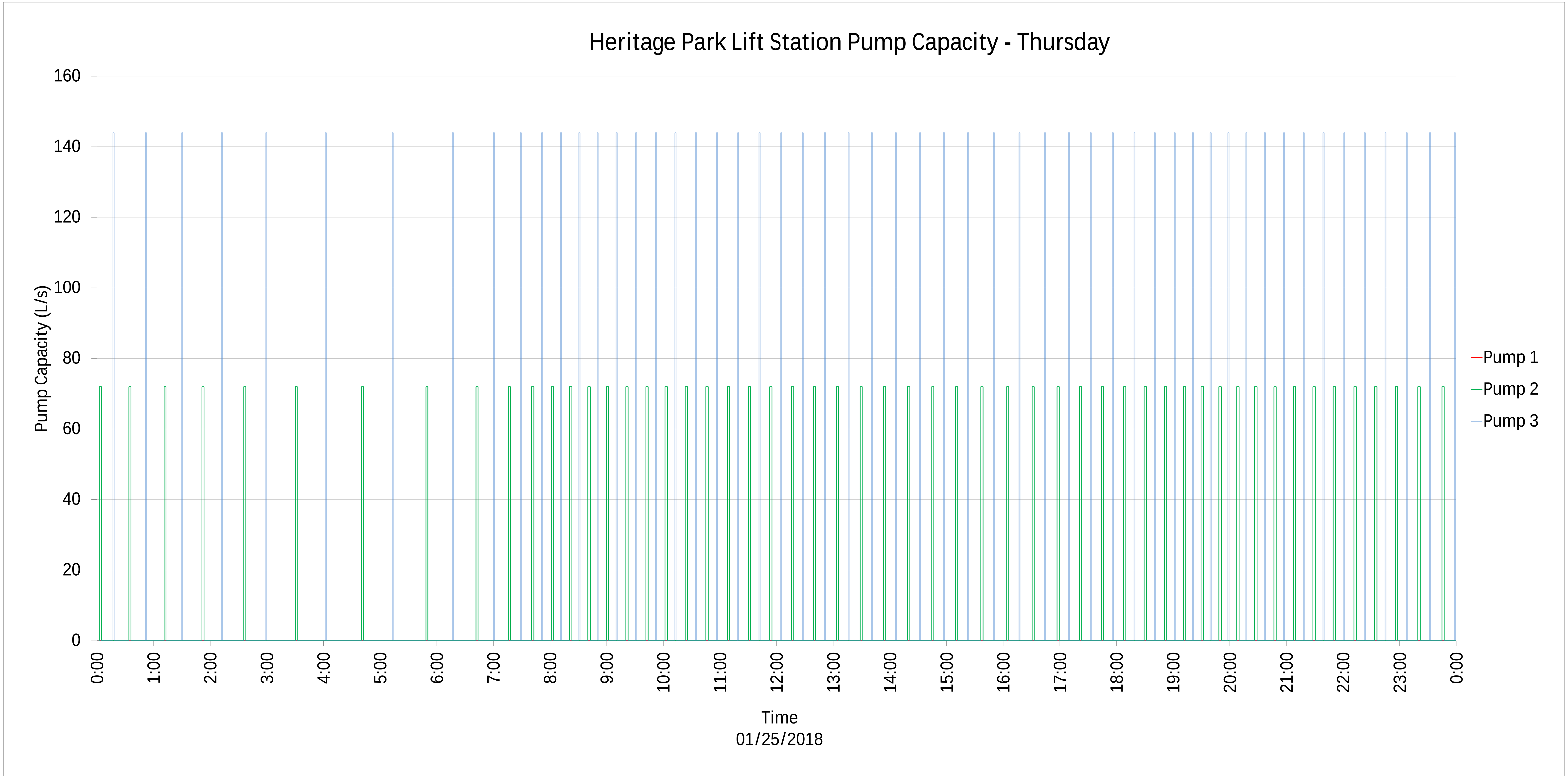

Heritage Park Lift Station Pump Capacity - Sunday
Pump Capacity (L/s)
160
140
120
100
80
60
40
20
0
0:00
1:00
2:00
3:00
4:00
5:00
6:00
7:00
8:00
9:00
10:00
11:00
12:00
13:00
14:00
15:00
16:00
17:00
18:00
19:00
20:00
21:00
22:00
23:00
0:00
Time
01/21/2018
Pump 1
Pump 2
Pump 3

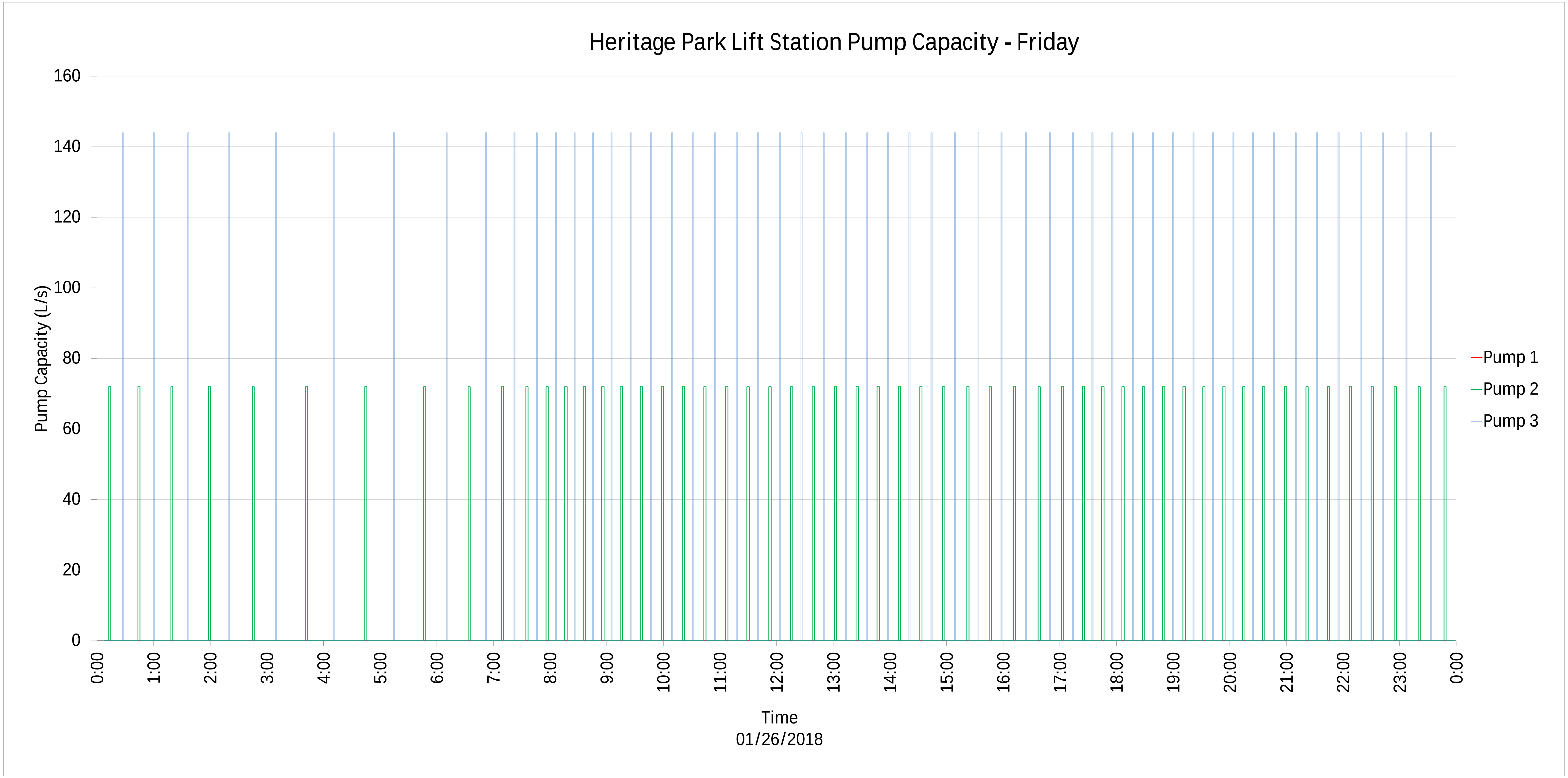


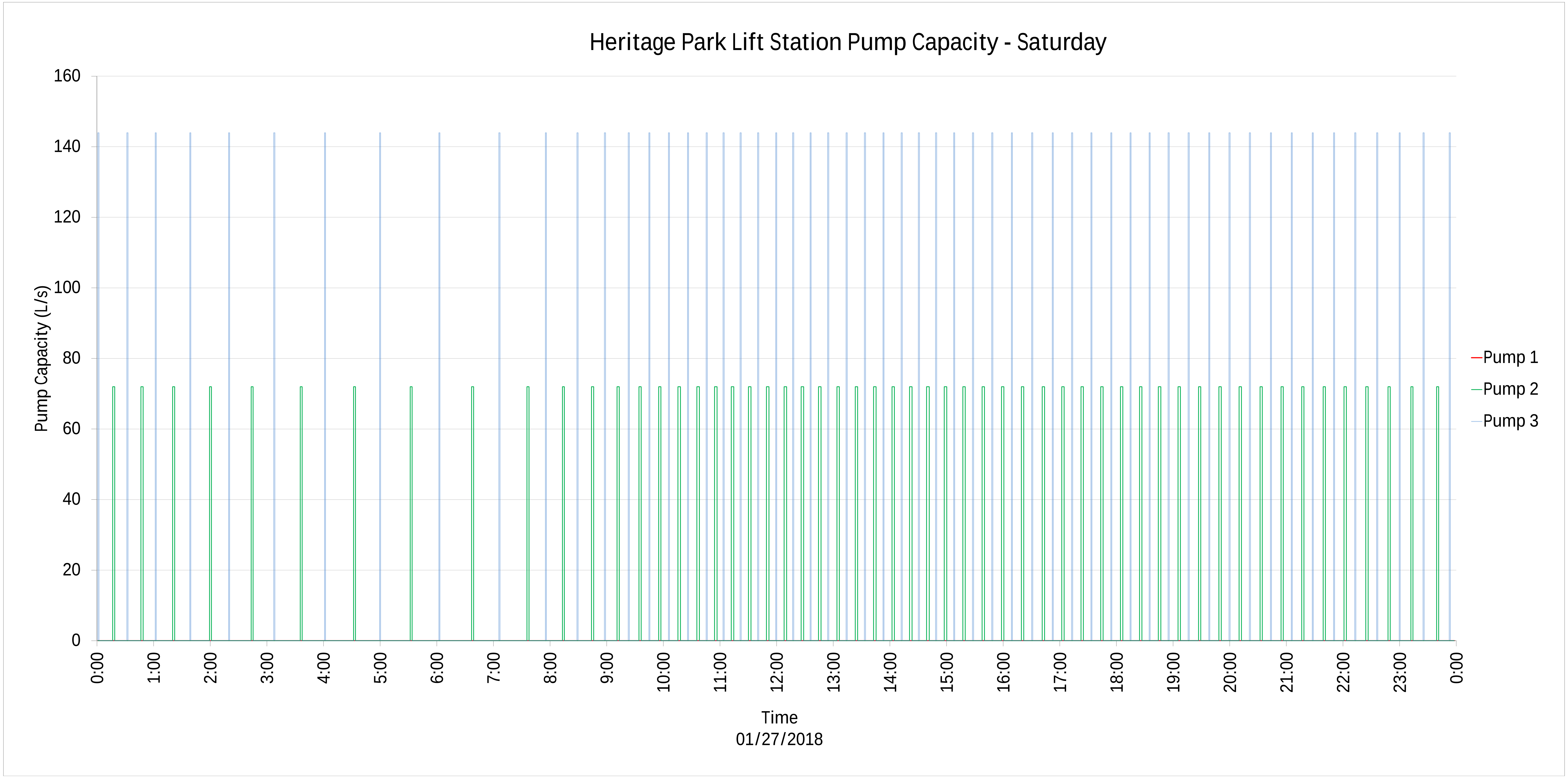
Heritage Park Lift Station Pump Capacity - Monday
Pump Capacity (L/s)
160
140
120
100
80
60
40
20
0
0:00
1:00
2:00
3:00
4:00
5:00
6:00
7:00
8:00
9:00
10:00
11:00
12:00
13:00
14:00
15:00
16:00
17:00
18:00
19:00
20:00
21:00
22:00
23:00
0:00
Time
01/22/2018
Pump 1
Pump 2
Pump 3

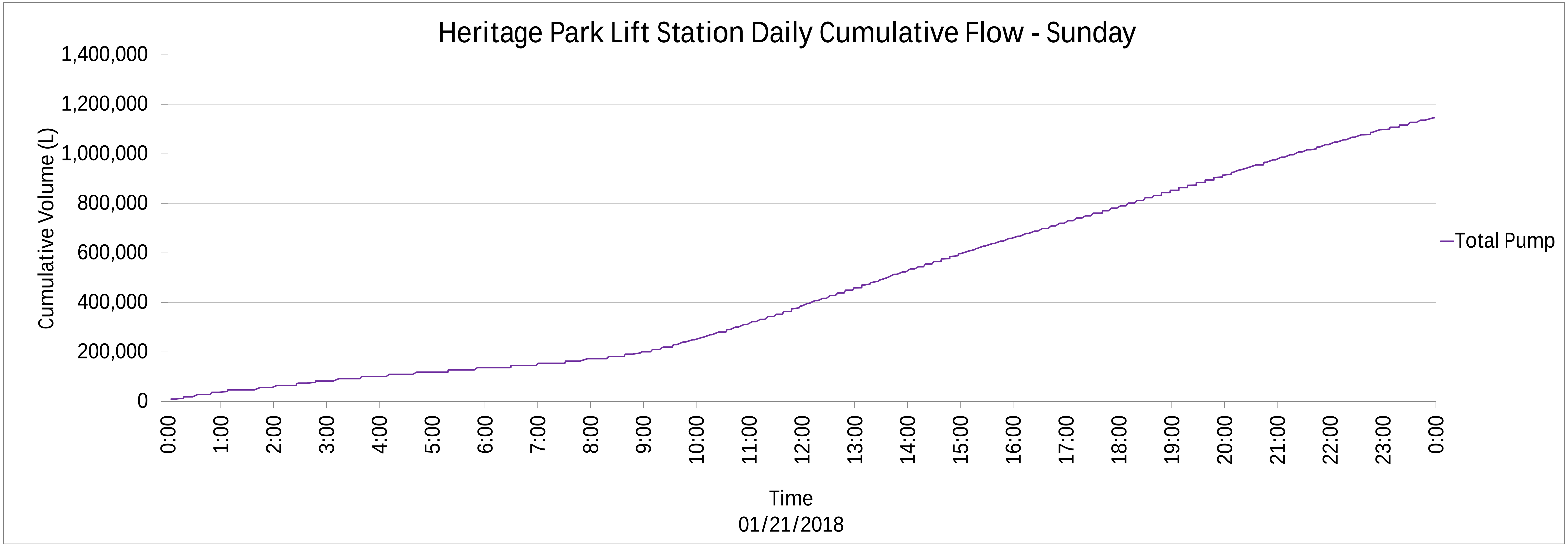

Heritage Park Lift Station Pump Capacity - Tuesday
Pump Capacity (L/s)
160
140
120
100
80
60
40
20
0
0:00
1:00
2:00
3:00
4:00
5:00
6:00
7:00
8:00
9:00
10:00
11:00
12:00
13:00
14:00
15:00
16:00
17:00
18:00
19:00
20:00
21:00
22:00
23:00
0:00
Time
01/23/2018
Pump 1
Pump 2
Pump 3

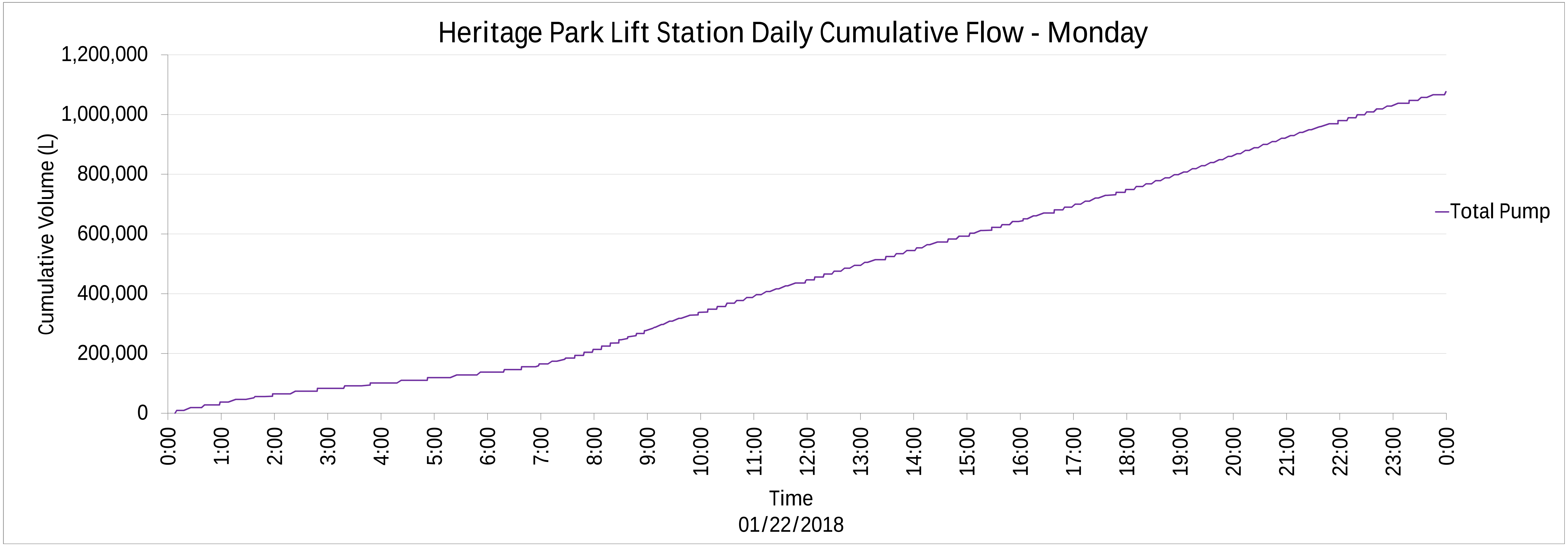

Heritage Park Lift Station Pump Capacity - Wednesday
Pump Capacity (L/s)
160
140
120
100
80
60
40
20
0
0:00
1:00
2:00
3:00
4:00
5:00
6:00
7:00
8:00
9:00
10:00
11:00
12:00
13:00
14:00
15:00
16:00
17:00
18:00
19:00
20:00
21:00
22:00
23:00
0:00
Time
01/24/2018
Pump 1
Pump 2
Pump 3

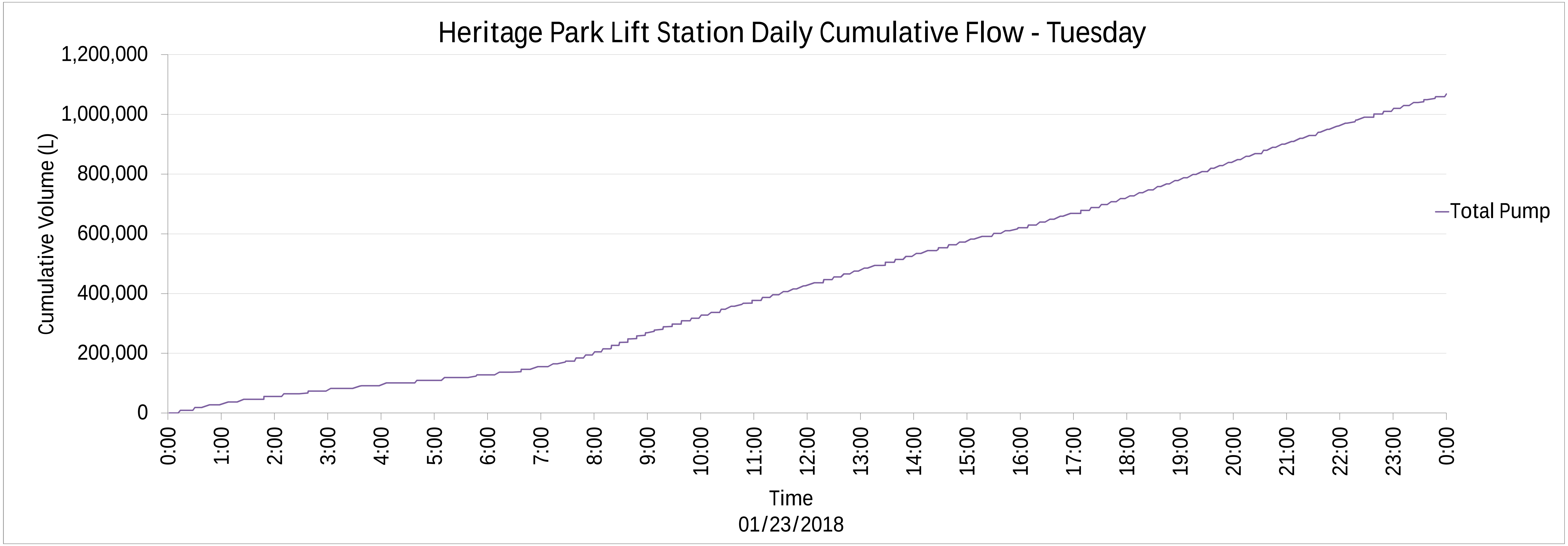

Heritage Park Lift Station Pump Capacity - Thursday
Pump Capacity (L/s)
160
140
120
100
80
60
40
20
0
0:00
1:00
2:00
3:00
4:00
5:00
6:00
7:00
8:00
9:00
10:00
11:00
12:00
13:00
14:00
15:00
16:00
17:00
18:00
19:00
20:00
21:00
22:00
23:00
0:00
Time
01/25/2018
Pump 1
Pump 2
Pump 3

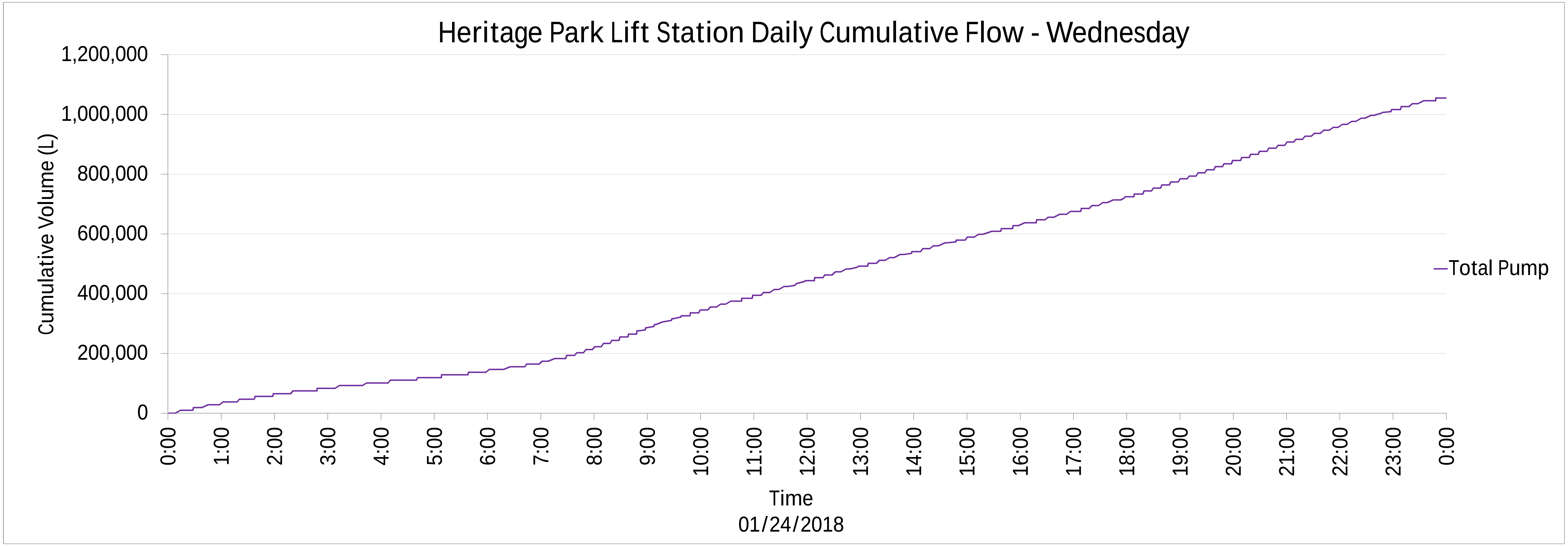

Heritage Park Lift Station Pump Capacity - Friday
Pump Capacity (L/s)
160
140
120
100
80
60
40
20
0
0:00
1:00
2:00
3:00
4:00
5:00
6:00
7:00
8:00
9:00
10:00
11:00
12:00
13:00
14:00
15:00
16:00
17:00
18:00
19:00
20:00
21:00
22:00
23:00
0:00
Time
01/26/2018
Pump 1
Pump 2
Pump 3

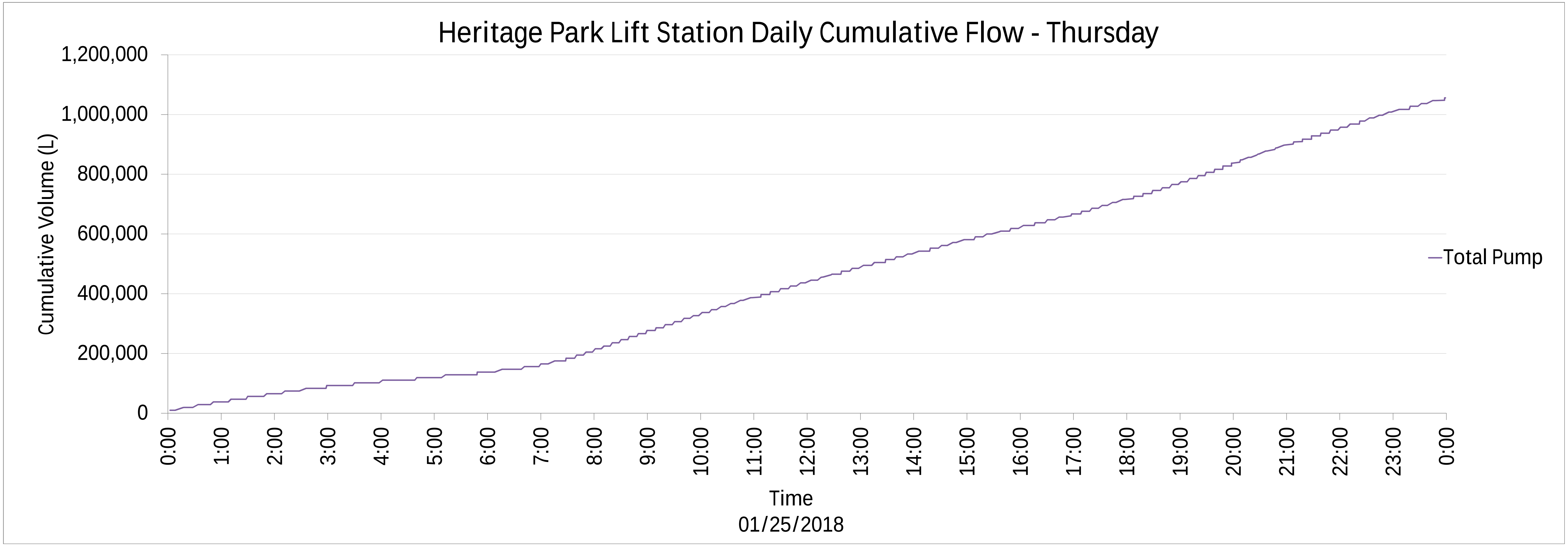

Heritage Park Lift Station Pump Capacity - Saturday
Pump Capacity (L/s)
160
140
120
100
80
60
40
20
0
0:00
1:00
2:00
3:00
4:00
5:00
6:00
7:00
8:00
9:00
10:00
11:00
12:00
13:00
14:00
15:00
16:00
17:00
18:00
19:00
20:00
21:00
22:00
23:00
0:00
Time
01/27/2018
Pump 1
Pump 2
Pump 3

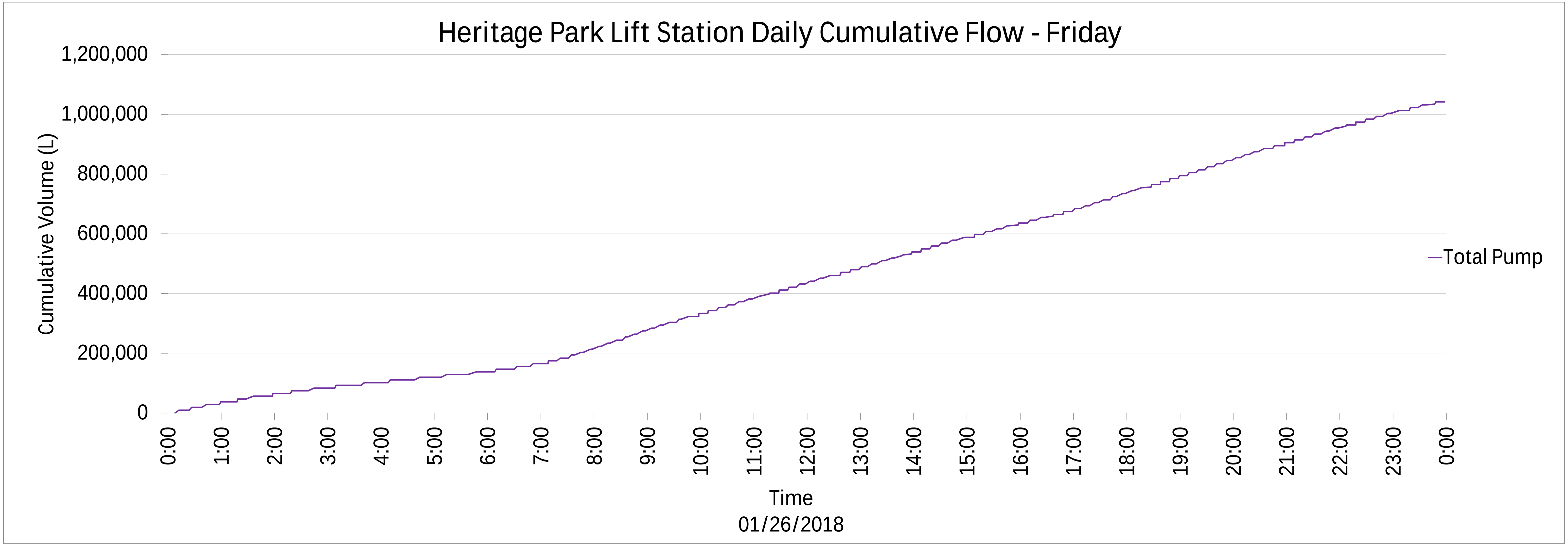

Heritage Park Lift Station Daily Cumulative Flow - Sunday
Cumulative Volume (L)
1,400,000
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
0:00
1:00
2:00
3:00
4:00
5:00
6:00
7:00
8:00
9:00
10:00
11:00
12:00
13:00
14:00
15:00
16:00
17:00
18:00
19:00
20:00
21:00
22:00
23:00
0:00
Time
01/21/2018
Total Pump

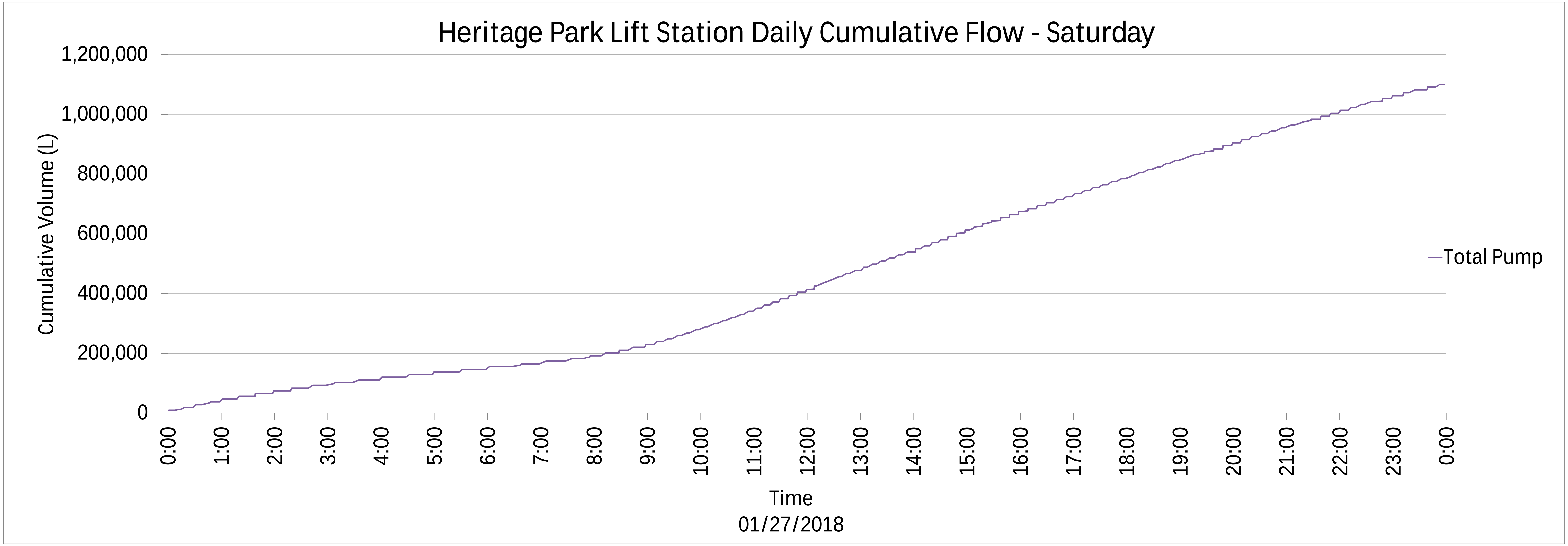

Heritage Park Lift Station Daily Cumulative Flow - Monday
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
Cumulative Volume (L)
0:00
1:00
2:00
3:00
4:00
5:00
6:00
7:00
8:00
9:00
10:00
11:00
12:00
13:00
14:00
15:00
16:00
17:00
18:00
19:00
20:00
21:00
22:00
23:00
0:00
Time
01/22/2018
Total Pump


Heritage Park Lift Station Daily Cumulative Flow - Tuesday
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
Cumulative Volume (L)
0:00
1:00
2:00
3:00
4:00
5:00
6:00
7:00
8:00
9:00
10:00
11:00
12:00
13:00
14:00
15:00
16:00
17:00
18:00
19:00
20:00
21:00
22:00
23:00
0:00
Time
01/23/2018
Total Pump


Heritage Park Lift Station Daily Cumulative Flow - Wednesday
Cumulative Volume (L)
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
0:00
1:00
2:00
3:00
4:00
5:00
6:00
7:00
8:00
9:00
10:00
11:00
12:00
13:00
14:00
15:00
16:00
17:00
18:00
19:00
20:00
21:00
22:00
23:00
0:00
Time
01/24/2018
Total Pump


Heritage Park Lift Station Daily Cumulative Flow - Thursday
Cumulative Volume (L)
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
0:00
1:00
2:00
3:00
4:00
5:00
6:00
7:00
8:00
9:00
10:00
11:00
12:00
13:00
14:00
15:00
16:00
17:00
18:00
19:00
20:00
21:00
22:00
23:00
0:00
Time
01/25/2018
Total Pump


Heritage Park Lift Station Daily Cumulative Flow - Friday
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
Cumulative Volume (L)
0:00
1:00
2:00
3:00
4:00
5:00
6:00
7:00
8:00
9:00
10:00
11:00
12:00
13:00
14:00
15:00
16:00
17:00
18:00
19:00
20:00
21:00
22:00
23:00
0:00
Time
01/26/2018
Total Pump


Heritage Park Lift Station Daily Cumulative Flow - Saturday
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
Cumulative Volume (L)
0:00
1:00
2:00
3:00
4:00
5:00
6:00
7:00
8:00
9:00
10:00
11:00
12:00
13:00
14:00
15:00
16:00
17:00
18:00
19:00
20:00
21:00
22:00
23:00
0:00
Time
01/27/2018
Total Pump

Appendix B

Disposal of Chlorinated Water Memorandum

AECOM
99 Commerce Drive 204 477 5381 tel
Winnipeg, MB, Canada R3P 0Y7 204 284 2040 fax
www.aecom.com

To: File

Date: May 2, 2018
Project #: 60549028 (500)
From: Mike Gaudreau, P. Eng.

cc: Armand Delaurier, City of Winnipeg
Marv McDonald and Adam Braun, AECOM

Memorandum

Subject: **High Risk River Crossings - Disposal of Chlorinated Water**

Two feeder mains (FMs) will be inspected as a part of the High Risk River Crossing (HRRC) Phase Two project. The cleaning and inspection operations, and subsequent flushing and disinfection operations of these pipelines will require disposal of chlorinated water either to a Waste Water Sewer (WWS) or Combined Sewer (CS), or a natural water body. Flushing and disinfection operations will be completed in accordance with CW 2125 and AWWA C651.

During the cleaning and inspection processes proposed for the inspection program, a majority of scale and debris will be removed via pigging. Typically, the river crossings are in the range of 200-250 metres in length and will require 3-5 cleaning passes (Table 1).

Table 1 – Water Main Characteristics

Site	Diameter (mm)	Approximate Length (m)	Approximate Water Volume (m^3)
Charleswood - Assiniboia Feeder Main	600	225	70
Kildonan - Redwood Feeder Main	600	225	70

The flushing component typically requires a high flow corresponding to a minimum flushing velocity of 0.76 m/s to remove debris from the FM and is completed using water from the City’s distribution system, which has a relatively low chlorine concentration level (maximum of 1.5 mg/l). Due to the proceeding pigging operations, it is not expected that an extended flushing period will be required.

Disinfection requires significantly higher chlorine levels (10 mg/l to 75 mg/l); however, the flow rates may be significantly lower than those required during flushing.

Due to the chlorine content in the water, discharge to a river body directly, or through a Land Drainage Sewer (LDS) is not permitted without de-chlorination. Options for de-chlorination of the water used for flushing and disinfection of the WMs/FMs are:

1. Discharge to a WWS/CS for treatment at a Pollution Control Center (PCC).
2. De-chlorination at source and discharge to river or LDS using Vita-D-Chlor^{TM} Tablets or solution.
3. Detention until residuals are within safe limits.

Option 1 requires a careful review of the receiving WWS/CS to ensure basement flooding or combined sewer overflows do not occur. As a conservative approach, a limiting flow rate of half the full flow capacity of the receiving sewer was adopted for this project. Maximum discharge rates to nearby WWS/CS manholes have been provided for all FM crossings on the construction drawings.

For Option 2, a review of the chlorine residuals in the system as it relates to the amount of required Vita-D-Chlor^{TM} Tablets for de-chlorination must be completed. Based on Vita-D-Chlor^{TM} Taby Mat and Sock literature, a flow rate of 30 l/s (500 gpm) with a concentration of 2 mg/l requires four tablets placed in a Taby Mat or Sock for de-chlorination. The tablets typically last between 15-20 minutes, and therefore, the contractor must monitor and replace the tablets during the de-chlorination process. The Vita-D-Chlor^{TM} Tablets come in packages containing up to 140 tablets.

For Option 3, the chlorinated water would be transferred and stored in a tank. Chlorine residuals would then be monitored until the 0.01 mg/L threshold is reached at which point the water could be discharged to an adjacent water course or LDS.

The contractor will be required to submit a detailed plan for dealing with chlorinated water used for flushing and disinfection of the cleaned and inspected pipelines for review prior to undertaking the work. A review of the plan will be undertaken based on proposed flushing flow rates, permissible discharge rates, and de-chlorination procedures.

Feeder Main Flushing

FM flushing requires a minimum velocity of 0.76 m/s within the pipe. This results in a minimum flushing flow rate of:

- 215 l/s for a 600 mm FM

Since water within the City's distribution system can have chlorine residuals up to 1.5 mg/l, the flushed water will require de-chlorination.

The allowable flow rates for the receiving WWS/CS (previously discussed) are much lower than the minimum flushing flow rate. Consequently, this will require the contractor to store a large amount of water and utilize a controlled discharge to the WWS/CS, potentially making Option 1 impractical.

Based on the low concentration of chlorine residuals in the distribution system, Option 2 (de-chlorination) may be a more attractive option. For a one hour of flushing operation at the minimum flushing rate and a chlorine residual concentration of 2mg/l, the required amount of Vita-D-Chlor ^{TM} Tablets amounts to roughly the following:

- 600 mm WM: 115 tablets for de-chlorination

It is not expected that this duration of flushing will be required.

Water Main/Feeder Main Disinfection

The flow rates required for disinfection can be much lower than those required for flushing, but the concentration of chlorine flushed out of the system can range from 10 mg/l to 75 mg/l. Flushing of the system, which occurs after a 24 hour retention period, requires chlorine residuals greater than 10 mg/l and is completed using water from the distribution system.

Option 1 in this case may be an attractive option if the contractor can maintain flow rates below the maximum allowable flow rates in the receiving WWS/CS.

Implementation of Option 2 may be less preferable, as the initial concentration can be very high and would require a large amount of Vita-D-Chlor™ Tablets.

Mike Gaudreau, P. Eng.
Municipal Engineer
Conveyance
MG/pab

Appendix C

Risk Assessment Technical Memorandum

AECOM
99 Commerce Drive 204 477 5381 tel
Winnipeg, MB, Canada R3P 0Y7 204 284 2040 fax
www.aecom.com

To: Armand Delaurier, C.E.T., City of Winnipeg

Date: June 14, 2018
Project #: 60549028 (500)
From: Nathan Kehler, P. Eng. and
Adam Braun, P. Eng.

cc: Chris Macey, P. Eng. and Marv McDonald, C.E.T., AECOM

Technical Memorandum

Subject: **High Risk Crossings - Phase 2 - 2018 Inspection Program Risk Management**

1. Introduction

This memo is intended to address risk management for the upcoming 2018 High Risk River Crossing (HRRC) 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 HRRC Phase One program undertaken between 2012 and 2015 which saved the City upwards of $30 million dollars in replacement costs through inspection, condition assessment, and development of an asset management program focused on re-inspection and rehabilitation.

The work undertaken to clean and inspect the river crossings does pose numerous logistical challenges and risks, 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 2018 involves cleaning and inspection of five pipelines consisting of two sewer crossings (Sites 4 and 5) and three water crossings (Sites 1, 2, and 6). External inspection, not requiring cleaning or work within the pipe is proposed for Site 3. Risk assessments have been completed for the following pipelines:

- Site 1: Kildonan-Redwood Feeder Main (600 mm - Steel)
- Site 2: Charleswood-Assiniboia Feeder Main (600 mm - Steel)
- Site 3: St. Vital Bridge Force Main (500 mm - Steel)
- Site 4: Newton Ave Force Main (350 mm - HDPE)
- Site 5: Heritage Park Force Main (250 mm - PVC)
- Site 6: Fort Garry – St. Vital Feeder Main (600 mm – Cast Iron)

The scope of this technical memorandum is 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 conventional sewer flushing and pigging. Inspections will involve a number of different technologies and approaches, including inline electromagnetic Remote Field Technology (RFT), CCTV/leak detection (Sahara), SONAR, and physical sampling. Rationale for the use of each respective inspection technology and a site by site approach can be found in our draft Technology Selection memo (February, 2018). The following inspections are proposed as part of this program.

- Site 1: Kildonan-Redwood Feeder Main – Inline RFT inspection
- Site 2: Charleswood-Assiniboia Feeder Main – Inline RFT inspection
- Site 3: St. Vital Bridge Force Main – External inspection
- Site 4: Newton Ave Force Main – SONAR inspection
- Site 5: Heritage Park Force Main – External sampling
- Site 6: Fort Garry – St. Vital Feeder Main – Sahara inspection

Inspection of all pressure pipeline crossings will be accompanied by a low head leakage test.

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 can be found in Section 3.

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

Risk Profile – Low

Mitigation Strategies

2.1.1 Sewer Sites

For sewer force main 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 periodic system disruptions are required, risks will be mitigated by a well-planned and conservatively scheduled work plan, such that the system can be returned to service in an appropriate timeframe or bypasses installed. No major system disruptions are anticipated for the remaining assets in the inspection program.

A hydraulic technical memorandum on the Newton Ave Force Main site was issued on February 2, 2018. Based on our analysis to date the required modifications to the Newton Ave Force Main crossings can be successfully completed without adversely affecting operation of the wastewater collection system. Modifications made to the gate chambers between the Linden and Hawthorne force mains will allow for isolation of the force main to be inspected with minimal disruption to these systems. These are discussed more fully later in this document.

Hydraulic modeling for the Heritage Park Force Main has also been completed in order to determine shut down windows for pipe removal and sampling should it be required. This memorandum has identified a safe dry weather flow disruption window of 5 to 7 hours to conduct pipe inspection, testing and sampling work proposed at this site.

Most sewer pipeline work will need to be completed during periods of dry weather 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.

2.1.2 Water Sites

The largest risks associated with major water crossings 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. We do not anticipate the proposed feeder main 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.

AECOM provided valve closures and proposed shutdowns to the City's Water Planning & Project Delivery (WPPD) Branch for assessment and preliminary modeling comments were provided by the WPPD Branch on February 12, 2018 via email, attached in Appendix A. Key findings reported by the City are:

- The proposed isolation of the Kildonan-Redwood Feeder Main (Site 1), Charleswood/Assiniboia Feeder Main (Site 2), and the Fort Garry-St. Vital Feeder Main (Site 6) individually do not pose a significant risk to the operation of the regional water system as a whole. Note, some pressure drops and potential for discolored water were reported.
- The Kildonan-Redwood Feeder Main (Site 1) and Charleswood/Assiniboia Feeder Main (Site 2) may be isolated concurrently.
- The Kildonan-Redwood Feeder Main (Site 1) and Fort Garry-St. Vital Feeder Main (Site 6) may not be isolated concurrently.

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

Risk Profile – Low

Mitigation Strategies

2.2.1 Physical Damage to the Pipeline

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 25 mm less than the pipe ID.

Cleaning in wastewater crossings will be limited to traditional sewer flushing and pigging using soft to medium foam cleaning pigs (Figure 2). Cleaning of ferrous metal pressure mains will include a progressive pigging operation including the use of medium foam bristle pigs (see Figure 3) for the removal of tuberculation. Aggressive cleaning via scraper pigs and/or high velocity jetting equipment will be avoided.

2.2.2 Overpressure

In addition to the deployment of inline inspection tools and physical sampling, AECOM is proposing the use of low head leakage test on two sewer forcemains, namely Newton and Heritage Park. The low head pressure test will be undertaken at pressures either at or slightly above the normal operating pressure of the main. This will allow for confirmation of current hydrostatic integrity of the crossing without the risk of damaging the pipeline by rupturing a joint or aggravating existing corrosion related defects.

The RFP for this project indicated that the City of Winnipeg will be undertaking baseline pressure tests on Sites 1 and 2. To date, AECOM has not been provided with these results.

This type of test was undertaken as part of the 2015 inspection program (HRRC Phase 1) to confirm the hydrostatic integrity of the inspected water mains and successful in detecting a leak on the Maryland Water Main crossing. The City’s Water Services Division (WSD) was then able to locate the leak using their leak correlator unit, so a repair could be done.

While, the leakage test will typically be completed at pressures slightly above normal operating pressure (say 5 psi), where valve integrity is in question, and we anticipate the potential for valve bypassing, the test will include a second test at a pressure slight lower pressure than the system to confirm any valve leakage. A stable pressure would indicate the isolated section is water tight, while an increase in pressure would be indicative of bypassing valves.

2.3 Obstructing Pipelines during Cleaning Processes

Risk Profile – Low - Medium

Mitigation Strategies

Cleaning and inspection tasks will result in deployment of full diameter cleaning pigs through the pipelines. Pipeline pigging will be completed in a progressive manner, starting with soft, undersized pigs and progressing to firmer, full sized products as required to achieve the desired level of cleaning. More aggressive pigs are not deployed until previous pigging attempts are proven successful.

While, typically cleaning pigs are deployed and advanced by differential pressure flow through the pipeline, the use of untethered pigging in municipal applications can be problematic. In water systems, water from the pigged section needs to be isolated from the system, which is difficult in larger pipelines.

Tethered pigging has been successfully utilized in Winnipeg (HRRC Phase 1) to clean both gravity sewer siphons and water main crossings, however, pulling large pipe pigs by tether can result in significant forces and failure of pigs, as discussed further below. It is recommended that cleaning as part of this project be completed using tethered pigging, utilizing properly designed pigs and tethers.

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 (force mains 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.

The formation of tuberculation is a function of water quality, and interior coatings. Based on our experience during the inspection works in 2012 and 2015 and other projects on the City's regional water infrastructure we are expecting moderate tuberculation within the water mains. Both the Charleswood-Assiniboia and Kildonan-Redwood Feeder Mains are reported to have been manufactured with a coal tar enamel interior coating which should assist in reducing tuberculation buildup. Figure 1 depicts tuberculation within a retrieved sample from the St. James Water Main in 2015. Note this pipeline was manufactured without any form of interior lining.

Tuberculation will be removed to a level that's required for inspection through the use of flushing, and deployment of foam and coated bristle pigs (Figure 3).

Figure 1 - Tuberculation within the St. James Water Main

Case Study:

During the 2015 inspection program (HRRC Phase 1) the tether for a foam bristle pig deployed within the Goulet-Doucet water main crossing broke and resulted in the need for external intervention to retrieve the cleaning pig. Inspection of the tether found that the nylon pull rope was damaged during the manufacturing process and thus broke during the cleaning operation. Figure 2, depicts the cleaning pig with an intact nylon pull rope and the broken tow rope from the pig lost within the Goulet-Doucet water main crossing.

After failure of the nylon pull ropes, all of the cleaning pigs were modified with steel cables and plates, see Figure 3. In order to avoid the issues encountered on the Goulet-Doucet water main, a requirement for load rated (steel or synthetic) cables and steel support plates will be included in the contract documents.

Figure 2 - Foam Pig with Nylon Pull Rope (left) and Broken Pull Rope (right)

Figure 3 - Modified Cleaning Pig with Steel Cable Pull Ropes

2.4 Obstructing Pipelines during Inspection Processes

Risk Profile – Low

Mitigation Strategies

Deployment of inspection tools will be undertaken by an experienced pipeline inspection contractor. None of the inspection technologies/platforms selected for these inspections require a tight to wall pipeline fit. Both SONAR and Sahara inspection tools are considerably undersized from the pipeline internal diameter and the advanced electromagnetic inspection tools identified for use on this project are typically deployed with flexible sensor arrays and are undersized by 25 mm or more from the interior diameter. Thus, the 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.

To further reduce risk associated with hard debris obstructions, 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. Inspection contractors typically build their own gauge pigs specifically for their own inspection equipment and we therefore do not expect any issues with obstructions during the inspection process. Figure 4 depicts gauge pig deployment in the Goulet-Doucet Water Main crossing in 2015.

Figure 4 - Gauge Pig Deployment

2.5 Structural Damages/Buoyancy Effects on Pipelines

Risk Profile – Low

Mitigation Strategies

Four of the six pipelines to be inspected under this program are dredged into the channel at shallow depths, the remaining two are either encased within a tunnel (Site 6) or aerial (Site 3). Flotation risk for each of the buried pipelines has been assessed for a dewatered state, see Table 1. Overall, the pipelines exhibit factors of safety above unity (1) and are thus at a low risk of floatation during inspection, should they become dewatered. However, a requirement to maintain the pipeline in a full, non-dewatered state will be included in the cleaning and inspection contracts as an additional means of reducing floatation potential. 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 during both submission reviews and the work itself.

Table 1 - Flotation Factor of Safety - Dewatered

Site	Crossing	Factor of Safety Against Floatation (Dewatered)
1	Kildonan-Redwood Feeder Main	2.81
2	Charleswood/Assiniboia Feeder Main	2.24
3	St. Vital Bridge Force Main (Aerial)	N/A
4	Newton Ave Force Main (HDPE Siphon Only)	1.65
5	Heritage Park Force Main	7.60
6	Fort Garry-St. Vital Feeder Main (Tunneled)	N/A

2.6 System Operational Status

Risk Profile – Low-Medium

Mitigation Strategies

The City's regional (and distribution) water system valves are not typically operated on a regular basis and thus a risk exists of discovering inoperable or bypassing valves while isolating and dewatering the identified feeder main crossings. In order to avoid unforeseen delays in construction and inspection, a full list of required valve closures was provided to the City via email on December 21, 2017 with the understanding that Operations personnel would test the operational status of the valves in advance of the work. Should valves be found to be inoperable or bypassing, there are two options: closure of additional valves or replacement. The former will result in larger impacts on the distribution system as additional offtakes are affected, and the latter could result in significant delays to the inspection works. Large diameter butterfly valves are not always stocked in North America and delivery times can be up to 20 weeks. Further procurement of ancillary components (Victaulic couplings, dismantling joints, etc.) could take 3 to 4 weeks.

AECOM has provided both primary and secondary valve operations for the proposed shutdowns. Primary valve closures were indicated as being necessary to complete the work. In addition, secondary valve closures were recommended in order to provide “double blocking” on the system, which is considered good practice with regards to safety. Provided that all valves are functional and not bypassing, secondary valve closures are not critical to being able to complete the work at all locations except Charleswood-Assiniboia (Site 2). It is our understanding that all of the identified valves have been checked by WSD and were found to be in working order (email from Armand Delaurier, February 13, 2018).

Additionally, AECOM has recommended the City undertake trial shutdown's of each site to confirm both valve operation and ability to depressurize the identified crossings (i.e. confirm that valves are not bypassing). A trail shutdown of each site will confirm the City's ability isolate and dewater the identified crossings. Confirmation of valve operation alone does not necessarily confirm if system valves are seating and sealing correctly. If desired, we recommend that the trial shutdowns are scheduled to be completed overnight when the effects of discoloured water are minimized.

3. Site Specific Risk Reviews

The following is a review of site specific risks and their related mitigation measures.

3.1 Site 1 – Kildonan-Redwood Feeder Main

3.1.1 Background and Proposed Works

The Kildonan-Redwood Feeder Main is a critical component in the City's regional water system as one of two Red River crossings between the McPhillips and MacLean Pumping Stations. The crossing has 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 5). This site will offer some unique access challenges for cleaning and inspection using inline RFT tools.

Due to the configuration of the main, the access to the pipe will be established from within existing chambers on each side of the river, both requiring disassembly of existing chamber piping. For the west pipeline access, several mains will require isolation including two local 250 mm water mains on Redwood, a 400 mm offtake main heading south, and the 600 mm feeder main 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, providing access directly into the feeder main’s west drop shaft.

For east side access, the feeder main will require isolation at a valve chamber immediately east of the offtake chamber, and installation of a new 300 mm offtake valve outside of the chamber. The existing 600 mm cross, spool piece, and slip coupler along with ancillary piping and valves 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.

Upon completion of the inspection works, the existing side-outlet elbow will be reinstalled on the west side. New 600 mm fittings and spool piece will be installed to replace the 600 mm expansion joint and cross removed on the east side. Removable concrete panels will be replaced and re-sealed, and any excavations will be backfilled.

In addition to the crossing inspection works, AECOM is also proposing to remove abandoned piping from the east crossing chamber, remove an existing hydrant that is currently fed from within the chamber, and modify the 300 mm water main offtake as required.

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:

- 1 day for isolation and dewatering
- 5 days for disassembly and removal of chamber piping
- 2 days for cleaning operations
- 1 day for inspections
- 7 days for installation of proposed chamber piping
- 6 days for flushing, disinfection, sampling and health tests
- 2 days to return site to service

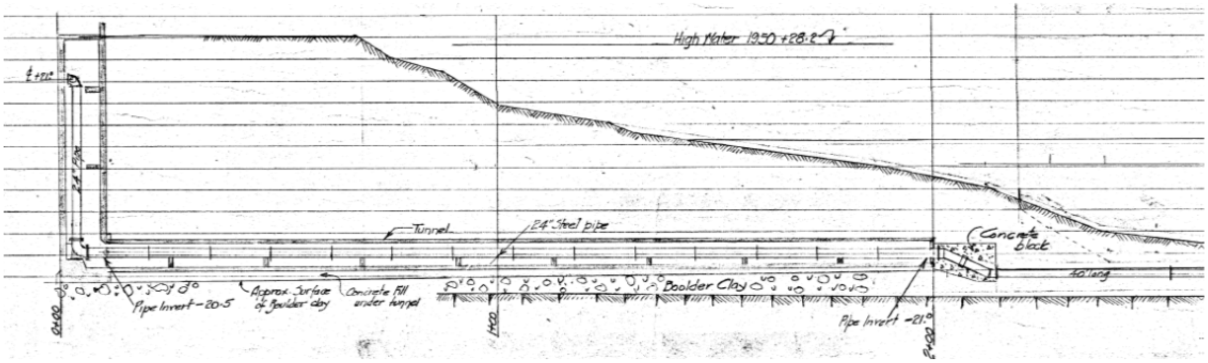


Figure 5: Kildonan-Redwood Feeder Main - Profile

3.1.2 Risk Mitigation

The site specific risks for this location are generally limited to the chamber modifications required to access the pipeline. Some of these risks include:

- Special design fittings: Some of the fittings that are being removed, specifically the 600 mm side outlet 90deg bend on the west side are unique and would have extensive lead times for procurement should they be damaged.

- Reassembly of existing chamber components:
  - Pipe alignment and lay lengths. Challenges can be encountered if the original assembly was poorly executed without recommended joint gaps or misaligned pipes.
  - Matching and procuring old style Victaulic components can pose issues.
- Presence of existing Victaulic jointed valves with non-standard lay lengths.

To mitigate these risks AECOM is proposing the following:

- Inspection of the existing chamber to identify Victaulic components (completed during HRRC Phase 1).
- Inclusion of specification requirements for the inspection of necessary Victaulic components by a qualified representative and Contractor prior to procurement of components.
- Design of piping modifications to eliminate or account for potential misalignment of piping and reduce risks associated with obscure components.
- Ensure all components are on site and pre-fit prior to cutting into the pipeline.

3.2 Site 2 – Charleswood-Assiniboia Feeder Main

3.2.1 Background and Proposed Works

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

It is expected that the out of service window for this crossing will be in the order of 3 weeks to complete system modifications (1 week), cleaning and inspection (1 week), and returning pipe to service (1 week), similar to the Kildonan-Redwood feeder main work outlined above. However, unlike the Kildonan-Redwood crossings, proposed system modifications are located outside of the existing valve chambers and carry considerably less risk in terms of assembly and schedule.

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. Accessing the pipe from inside the chamber 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 launch wyes on either end of the water main 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. Figure 6, depicts a typical launch assembly.

Figure 6 - WM Launch Assembly - Goulet-Doucet WM Inspection (2015)

Both primary and secondary valve closures are required to safely complete the work in order to eliminate thrust forces imparted to the chambers. Secondary closures on the water main offtakes have been suggested as a precaution, but are not critical to complete the work, provided that the primary valves are functioning and not bypassing.

3.2.2 Risk Mitigation

Site specific risk mitigation will be similar to that described for the Kildonan-Redwood Feeder Main crossing, including: Design to reduce construction risk, requirement to confirm dimensions, and pre-fitment of components prior to construction.

3.3 Site 3 – St. Vital Bridge Force Main

3.3.1 Background and Proposed Work

The 500 mm St. Vital Bridge Force Main crossing is unique in this inspection program in that it is the only aerial crossing. The 500 mm steel force main which conveys waste water flows from the Baltimore Road Pumping Station is supported from the underside of the St. Vital Bridge and crosses the Red River between Churchill Drive and Kingston Row. The pipeline is coated with 50 mm of polyurethane insulation and galvanized steel cladding.

As discussed in AECOM’s draft Technology Selection memo (February 16, 2018), an inline inspection of the force main is not practical. Thus an external inspection of the pipeline is proposed utilizing a two phase approach as described below:

Phase 1 of the inspection consisted of a visual survey of the full length of the pipeline utilizing a truck-mounted under-bridge crane (UBC). The UBC is owned and operated by the Public Works Department (PWD), and usage was coordinated and paid for by the Water and Waste Department (WWD). This inspection was completed by AECOM personnel on February 8, 2018. UBC inspections have a number of inherent safety risks, and specialty training is required in order to enter the crane basket, including fall arrest and self-rescue techniques. As such, only trained personnel were selected to perform the inspection in coordination with trained UBC operators.

The purpose of the inspection is to identify segments of the pipe that appear to be at a higher risk for exterior corrosion, such as those under deck drains or bridge deck joints where salt-laden runoff is likely to cause localized corrosion. The visual inspection also identified other areas of potential concern such as discoloration or defects in the pipe cladding, visible indications of leakage, pipe support points, bends, and the air release assembly at the south end of the bridge.

Phase 2 of the inspection will be performed as part of the tendered pipeline modifications and inspection contracts. The areas at high risk for external corrosion identified in Phase 1 will have the cladding and insulation removed from the pipe by the support contractor, utilizing shoring assembled on the river bank, in advance of the inspections. The inspections contractor will then perform an exterior inspection of the exposed areas utilizing an external inspection technology. AECOM is proposing to expose and inspect approximately 3 to 4 locations along the pipeline. As the pipeline is not buried, there is no restriction to performing further phased inspections in the future, should this be deemed necessary. If river conditions do not permit use of shoring, a UBC may be utilized.

Upon completion of the inspection, the force main’s external insulation and cladding will be restored by the support contractor.

If the UBC is utilized, specialized safety training is required in order to complete the modification and inspection work using a UBC. Contractors completing the work will have to demonstrate that their crew members have all the necessary training. AECOM assumes that WWD will again coordinate the use of PWD’s UBC for Phase 2.

Following completion of the inspection, new insulation and cladding will be installed to replace what was removed.

3.3.2 Risk Mitigation

With the proposed procedure the inherent risk to the pipeline is minimal. However, it is also conceivable that existing through-wall corrosion defects may be uncovered or disturbed during removal of the pipe cladding and insulation, resulting in the creation of an active leak. As a contingency, the contractor will be required to have access to repair clamps which may be installed prior to completing a permanent welded repair. It is our understanding from past experience that the Baltimore Road Pumping Station can be shut down for up to 8 hours in order to facilitate repairs, should they be required.[1]

3.4 Site 4 – Newton Avenue Force Main

3.4.1 Background and Proposed Work

The 350 mm HDPE Newton Avenue force main crossing services the Hawthorne Pumping Station and runs in parallel to a 350 mm steel force main servicing the Linden Pumping Station. Modifications made during the 2014 inspection program permit cross connection between the two force mains and the following modifications within the upstream valve chamber will be undertaken to permit isolation and inspection of the 350 HDPE crossing. A blind flange will be installed on the tee in the east chamber allowing the Linden and Hawthorn pump station force mains to remain in operation, utilizing the 350 mm steel crossing for the duration of the work. Removal of the existing knife gate valve will permit insertion of cleaning and inspection tools.

---

[1] *UMA Engineering, Trial Program to Monitor Wastewater Sewer Pipeline River Crossings for Leaks in Compliance with Revised Environmental Act License No. 2669E (Draft), April, 2007.*

Removal of an existing 90 degree bend and drop pipe in the downstream manhole will also need to be undertaken to permit inspection. These modifications have no impact on the operation of the steel force main pipe.

The piping modifications will require brief shutdowns of the Hawthorne Pumping Station to permit the above noted chamber piping modifications. 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 dated June 13, 2018. Results of this modeling concluded:

- Shutdowns of pump station will only occur during dry weather flow
- Shutdowns of pump station is limited to 12 hours, and will commence during nighttime periods to take advantage of low diurnal flow patterns. The noted duration above is expected to be conservative, as the piping has only recently been assembled.
- Once modifications are completed, both pump stations can operate indefinitely on one force main, without adversely affecting the system operation. This eliminates time constraints for placing the 350 mm HDPE crossing back into service, and further acts to reduce the risks associated with the cleaning and inspection work.

3.4.2 Risk Mitigation

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 that the time required to complete the piping modifications will be approximately 4 hours per shutdown. Once the piping modifications have been completed the system can operate on one crossing pipe indefinitely, reducing risk during the cleaning and inspection work.

Cleaning pigs will be launched from the upstream valve chamber and pushed through to the downstream manhole. Initial cleaning using conventional sewer flushing equipment will be recommended. It should be noted that SONAR inspections are routinely used to assess the level of debris buildup in pipelines and the inspection probe is not sized to a close tolerance of the pipe ID. As such there is less risk of the equipment becoming stuck in the line than comparable electromagnetic inspection equipment.

Cleaning and Inspection Procedure:

- Complete upstream valve chamber piping modifications
- Complete downstream discharge manhole modifications
- Force main cleaning
- Inspection
- Restore downstream discharge manhole to existing configuration
- 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.
- All fittings and piping will be on-site and prepped for assembly prior to pump station shutdowns

- Operating both Linden and Hawthorne pumping stations through a single force main as per the original configuration has been demonstrated both through modeling and experience in 2014 to not adversely affect pump station operation

3.5 Site 5 – Heritage Park Force Main

The Heritage Park force main is a 250 mm PVC pressure pipe installed in 1989 and modified in 2015 to accommodate construction of a new bridge on Ness Avenue over Sturgeon Creek. Waste water flow is pumped in a westerly direction from the Heritage Sewer Pumping Station, discharging into a manhole at the intersection of Ness Ave. and School Road.

As discussed in AECOM's draft Technology Selection memo (June, 2018) an inline inspection of the force main is not recommended due to the material utilized in its construction and site specific conditions. By sampling and testing a portion of the original 1989 force main and completion of a low head pressure test AECOM will be able to assess the condition of the force main as a whole without the need for an invasive inline inspection.

In order to further corroborate the condition of the pipe, a sample of pipe material will be collected and sent for physical testing. AECOM has evaluated locations for retrieval of a sample of the original PVC force main including retrieval of abandoned portions of the main and concluded that an excavation adjacent to the pumping station is the least disruptive location in terms of impact on the site.

- The presence of bridge embankments and working in close proximity to existing force main bends makes retrieval of the abandoned force main impractical.
- Sample retrieval from the west side of the creek will cause damage to the recently completed landscaping and path construction.
- The presence of other utilities makes excavations along Ness Ave. impractical.

In discussions with Steve Ferry of PSILab Inc. in Longmont, Colorado he has indicated that a 350 to 400 mm sample is required to complete the anticipating testing regime. The pipe will be located using soft-dig methods to prevent damage to the pipe during excavation. Once the pipe has been located, the excavation will be completed and shoring installed. Prior to collecting the pipe sample, the contractor will be required to have all necessary equipment and replacement parts on hand. A sample of pipe approximately 1000 mm long will be collected allowing for redundancy and additional testing if required and a new section of pipe installed with two couplers. We anticipate working time to complete the sample collection will be approximately two hours.

Analysis and modeling of the pump station run times and upstream storage capacity has been completed and suggests the pumping station can be removed from service for 5-7 hours during dry weather conditions. This is ample time to complete the proposed sampling, if well planned. A temporary bypass can be accommodated using the existing station piping by disconnecting the 200 mm bypass pipe within the pump station and connecting a temporary hose at this location. The bypass hose can be run up to grade through the existing hatch in the north east corner of the station floor, and out the station through an existing window above grade. From there the hose can be run across the bridge to discharge in the downstream manhole. As there is ample time to complete the proposed works, AECOM has not completed detailed hydraulics on the bypass option at this time.

.

Following collection of the pipe sample, a low head leakage test will be performed on the pipeline to confirm the hydrostatic integrity of the crossing. We anticipate the pressure test operation to take approximately one hour to complete.

As in-line inspection of this force main is not being recommended at this time, cleaning and pigging operations are not required.

3.5.1 Risk Mitigation

Key Risk Mitigation Items:

- No in-line inspection means there is no need for extensive pipeline modifications or potential to cause a blockage in the line during cleaning.
- Analysis of pump station run times to determine shutdown windows.
- Minor and easily reversible modifications required to install a temporary bypass line, if required.
- Excavations only in non-critical areas away from public infrastructure.
- All fittings and piping will be on-site and prepped for assembly prior to removing pipe sample.

3.6 Site 6 – Fort Garry – St. Vital Feeder Main

3.6.1 Background and Proposed Work

The Fort Garry – St. Vital Feeder Main is a 600 mm grey cast iron pipeline crossing the Red River between the Fort Garry Bridges on Bishop Grandin Boulevard. The feeder main crossing was installed in conjunction with the Branch II Aqueduct and is installed within tunnel shafts and a tunnel in the underlying limestone bedrock crossing beneath the river. Subsequent to installation, the tunnel shafts and tunnel were filled with concrete, permanently encasing the pipelines.

The crossing consists of a vertical shaft on either side of the river with a horizontal tunneled crossing between them. Inspection of the line will require traversing through a series of 90 degree bends.

As discussed in AECOM’s draft Technology Selection memo (June, 2018) the environment in which the feeder main has been installed precludes the formation of exterior corrosion and thus the use of a visual inspection technology, such as Sahara, may be sufficient for us to infer the condition of the crossing as a whole. In addition to providing a visual assessment of the pipe wall and leak detection, Sahara will also permit assessment of debris buildup within the siphon. Due to the profile of the crossings, assessment of debris buildup is prudent prior to tendering a cleaning program as would be required prior to deployment of an inline RFT tool.

The Sahara tool is tethered, and small enough in diameter that it can be launched through a 2” port on the feeder main. Therefore, the likelihood of the tool becoming stuck in the pipeline is minimal. To complete the inspection, pipeline modifications would be limited to the installation of a new port in the west side valve chamber on the “river” side of the butterfly valve.

Visual results from the inspection will be used to assess interior corrosion on the pipeline. Should the Sahara inspection identify areas of concern warranting a more detailed inspection, deployment of an inline RFT tool can be undertaken in a manner similar to that of the other ferrous metal feeder main crossings.

Based on a review of site drawings, the existing valve chambers at the site are not adequate for launching of inline RFT tools and cleaning devices due to presence of butterfly valves. There are several options for the development of pipeline access for deployment of an inline RFT tool:

- Removal of the existing 90 degree bends within the Branch II Aqueduct tunnel shaft and installation of a new 600 mm tee on top of the feeder main drop pipe, however, this will require structural modification of the east and west tunnel shafts
- Reconfiguration of the existing valve chambers for provision of a launch assembly. This would likely require replacement of the existing butterfly valves and associated piping and may require reconstruction of the existing chambers themselves.
- Installation of launch assemblies exterior to the existing chambers and removal of the existing butterfly valves to facilitate inspection.

As access modifications for deployment of an inline RFT tool are not currently anticipated, plans for RFT inspection are not being developed further at this stage. Should deployment of an inline RFT tool be required, access modifications will be assessed, developed, and could be issued as a separate contract or as a change order to the awarded support contract.

3.6.2 Risk Mitigation

The deployment of Sahara vs. an inline RFT tool is the largest form of risk mitigation being undertaken at this site due to the significantly smaller impact to the pipeline and the City's regional water system in addition to its significantly lower deployment risk profile. Similar to all other sites requiring system modifications, confirmation and pre-fitting of materials prior to cutting into the pipe will be strictly enforced.

4. Conclusion

River crossing inspections need to be undertaken with an abundance of caution and careful planning by all parties, including the owner, Engineer, and Contractors. The risk assessment contained herein addresses the key issues and mitigation measures required to successfully complete this program.

We trust this information meets your requirements on this matter. Should you have any queries or require further information or clarification, please do not hesitate to contact either of the writers or Marv McDonald of this office.

N Kehler

Adam Braun. P. Eng.
Municipal Engineer
Conveyance
NJK/ADB/MGM/gms

Nathan Kehler, P. Eng.
Municipal Engineer
Conveyance

Appendix A

From:	Lucky, Ryan <ryanlucky@winnipeg.ca>
Sent:	Monday, February 12, 2018 11:46 AM
To:	McDonald, Marvin; Delaurier, Armand
Cc:	Braun, Adam
Subject:	RE: Water Modeling

Marv/Adam/Armand

The results of my modeling are below. The shutdowns should not occur between May-September long to minimize impacts and avoid potential conflicts with our water main cleaning program. I will be forwarding this information to Water Services as it is up to them as to if any shutdowns can proceed.

Kildonan – Redwood
Minimal pressure impact. Discolored water impact would be only potentially local from the secondary closures.

Charleswood-Assiniboine
The model predicts peak hour pressure drops of up to 2 psi. The St Charles and St James – Brooklands – Weston wards would potentially be impacted by discolored water. Grace Hospital falls within the potentially impacted area and would have to be notified prior to shutdown.

I don't see any compounded impacts or redundancy concerns with having the Kildonan – Redwood and Charleswood-Assiniboine FM shutdowns overlap. I agree with your recommendation that no more than 1 pipe string be open at any one time.

FG SV
Sahara Tool
I will need confirmation from Pure on what the required velocity will be. I expect that achieving 1m/s will be challenging and if it can be done the impacts would be quite significant. This will require some more discussion on the City side once I have modeled the system with the minimum velocity supplied by Pure.
Shutdown
This shutdown cannot overlap with Kildonan – Redwood as the east side of the City would be very vulnerable (negative pressures) if MacLean pumping station had any issues. I did the modeling assuming the secondary valves at St Mary's would not be closed to minimize the impact. If double blocking is required by the contractor I can re-run the model at that time. The model predicts peak hour pressure drops of up to 2.5 PSI. The pressure drops are fairly widespread and could be higher than the model predicts due to closed valves. The St Vital, South Winnipeg – St Norbert, St Boniface and River Heights – Fort Garry (Fort Garry Portion) wards could potentially be impacted by discolored water.

Regards,

Ryan

From: McDonald, Marvin [mailto:Marvin.McDonald@aecom.com]
Sent: Thursday, February 08, 2018 8:17 AM
To: Delaurier, Armand
Cc: Lucky, Ryan; Braun, Adam
Subject: RE: Water Modeling

Armand/Ryan

From our perspective, we are needing to assess construction timing effects. We will be taking the 2 crossings (Charleswood-Assiniboine, Kildonan-Redwood) out of service for extended periods.
We will be inspecting FGSV in conjunction with Branch I Pure mobilization sometime this spring

Charleswood Assiniboine)

- Installation of launch wyes – assume 1 week, may be in advance of inspection
- Pipe cleaning and inspection- assume 1 week (likely more like 2-3 days, but past experience has been reruns often required)
- Disinfection and testing – assume 1 week (usually about 4 days)

Kildonan –Redwood

- Disassembly of piping in advance of inspection – 3-5 days
- Pipe cleaning and inspection- assume 1 week (likely more like 2-3 days, but past experience has been reruns often required)
- Re-assembly and reconfiguration of chamber pipe- assume 1 week
- Disinfection and testing – assume 1 week (usually about 4 days)

FGSV

- Installation of a larger (2-3” port) to launce Sahara tool. This may be done live, but also may need a short shutdown to facilitate a saddle weld (few hours)
- Inspection – one day
- Disinfection not required (components disinfected prior to entry)
- We need to ensure flow directionality and velocity range to facilitate inspection (AECOM to provide the V range)

It is assumed we will not be permitted to shutdowns May long to September long, but please confirm.
Is it permissible that these 2 inspections overlap? Can they occur at same time? We would recommend that no more than one pipe string be open (cut, not be able to return to service within a few hours) at any one time, but with disinfection, testing etc, both could be out of service
Our schedules (to be confirmed) will need to be considered with other infrastructure projects. A majority of above would be September through mid October, although there may be opportunity to complete one of Charleswood in May.

All of the above will probably trigger red water, pressure effects. From our perspective, we just need the windows assured. The timing windows and answer to shutdown overlap are needed ASAP (end February) so we can incorporate into tenders, and would be preferable to have high level understanding for planned risk assessment workshop(last week February tentative?)

Marv

From: Delaurier, Armand [mailto:ADelaurier1@winnipeg.ca]
Sent: February-07-18 3:04 PM
To: McDonald, Marvin
Cc: Lucky, Ryan
Subject: Water Modeling

Marv,
As per Ryan’s questions to me, can you clarify what you’re looking for?

Also, can you indicate when you need this modeling sent to you by? We'll do our best depending on what you need & when.
Armand

From: Lucky, Ryan
Sent: Wednesday, February 07, 2018 2:53 PM
To: Delaurier, Armand
Subject: RE: HRRC Phase Two - Action Required!

Hi Armand,

I would like to clarify what exactly AECOM wants in terms of modeling. Given the timeline I would like to minimize the work required. Do they require pressure impacts, redundancy/operational considerations and predicted discolored water impacts? Adding the discolored water analysis would add delay in getting them the modeling results as I probably have most of the pressure impacts in my files. I could provide general comments (low, moderate, high, etc.) on the amounts of expected discolored water without modeling.

I am in tomorrow but off on Friday. If the more simplified approach is acceptable, I could provide the information by end of the day Monday. If full discolored water analysis is required I would get them the information later in the week assuming there are no operational situations that require immediate assistance.

Regards,

Ryan

Appendix D

Geotechnical Reports

AECOM
99 Commerce Drive 204 477 5381 tel
Winnipeg, MB, Canada R3P 0Y7 204 284 2040 fax
www.aecom.com

To: Marvin McDonald

Date: September 13, 2018
Project #: 60549028 (432.9)
From: Alexander Hill, P. Geo. (B.C.), FGS

cc: Ryan Harras and Adam Braun, AECOM

Technical Memorandum

Subject: **High Risk River Crossings – Phase 2 - Geotechnical Assessment for Site 5 and 6**

1. Introduction

1.1 General

The City of Winnipeg (City) has retained AECOM Canada Ltd (AECOM) to provide consulting services related to the condition assessment of High Risk Sewer and Water River Crossings contained within the Phase 2 assessment program. As part of the stipulated condition assessment, geotechnical review is required for two of the high risk crossing sites with the objective of identifying the potential risk of slope instability impacting the serviceability of buried sewer and water systems. The findings of this assessment will assist the City in evaluating the probability of failure, and how best to manage these assets. These two sites include; Heritage Park sewer force main Crossing (Site 5) and Fort Garry/St. Vital Feeder Main Crossing (Site 6).

The geotechnical component of the condition assessment includes a desktop study and review of the available background information along with completion of a visual field inspection of each site. The findings and conclusions derived from the review phase will be used as part of the preliminary slope stability assessment. This Technical Memorandum (TM) documents and presents the findings of the preliminary geotechnical assessment for Site 5 (Heritage Park) and 6 (Fort Garry/St. Vital).

1.2 Background

In previous assignments, AECOM was engaged by the City in 2012 to undertake an advanced river crossing inspection program consisting of nineteen (19) of the City’s most critical river pipeline crossings. Of these nineteen (19) crossing locations, five (5) were selected for geotechnical assessment based upon the observed conditions and the importance of the potentially impacted assets. Several of the sites in the 2012 program involve assets that are in the current program, including:

- St. Vital Bridge Force Main
- Charleswood Assiniboine Feeder Main
- Kildonan-Redwood Feeder Main
- Newton Avenue Force Main

Visual characterizations were previously completed on these crossings, and one of the crossings, the Kildonan-Redwood crossing was selected for further geotechnical assessment. The remaining sites were assessed to be of low risk of slope instabilities engaging the City utilities.

Geotechnical assessment for Kildonan-Redwood Feeder Main, including the conclusions and recommendations relating to each site can be found within the following report:

- AECOM Canada Ltd (March 24, 2016) Technical Memorandum- *Geotechnical Slope Stability Analysis High Risk Water and Wastewater River Crossings*. Ref. 60270487.

Two additional sites were added to the current program including:

- Heritage Park Force Main crossing of Sturgeon Creek (Site 5).
- Fort Garry/ St. Vital Feeder Main crossing of the Red River (Site 6).

The following reports and studies should be referenced in conjunction with this Technical Memorandum (TM):

Site 5 (Heritage Park)

- TREK Geotechnical (October 2014) *Geotechnical Investigation Ness Avenue at Sturgeon Creek Culvert Replacement, Preliminary Design*.

Site 6 (Fort Garry/St. Vital)

- AECOM Canada Ltd (December 12, 2013) Technical Memorandum- *Preliminary Geotechnical Assessment Fort Garry Interceptor Sewer Crossing at the Red River*.
- AECOM Canada Ltd (May 23, 2012)- Technical Memorandum; *Test hole adjacent to Interceptor, Fort Garry to St. Vital Interceptor, East Bank of Red River at Bishop Grandin Boulevard*.
- Klohn Leonoff Consultants Ltd (April 5, 1976) - Report on Sub-Soil Investigation for; *Fort Garry-St. Vital Corridor, Winnipeg, Manitoba*. Ref. W-983.

The following sources of information were referenced in review and evaluation of each high risk crossing site:

- As-built records;
- Aerial photography;
- Historic reports;
- Geological survey maps;
- City of Winnipeg operation staff; and
- Anecdotal information (where available).

1.3 High Risk River Crossing Locations

1.3.1 Site 5: Heritage Park

- Key Asset: 250 mm PVC Force Main

Site 5 is located directly north of Ness Avenue within the Heritage Park area of Winnipeg. The approximate UTM coordinates of the site are; 623,045 m East 5,527,640 m North, and the location of the site is shown in Figure 1 below.

Figure 1 - Site 5 - General Site Location

The Sturgeon Creek crossing at Ness Avenue currently consists of a high level bridge structure with an under-bridge pedestrian crossing. The Sturgeon Creek Greenway trail currently runs parallel to the creek along its western bank. Sturgeon Creek flows south towards the Assiniboine River, with the bridge crossing located just slightly north of a bend in the creek. The bend in the creek generally denotes areas of active erosion (due to higher velocity flows) and deposition. Recent bridge replacement works (including embankment reconstruction) were completed in 2016 to replace a former box culvert with the current three-span bridge. Embankment stabilization work extended approximately 100 m south and 10 m north of the bridge crossing within the study area.

The existing force main is situated approximately 40 m north of the bridge crossing at Ness Avenue, and crosses the creek from the existing lift station to the west bank of the creek as shown in Figure 2. The approximate minimum invert of the 250 mm force main is 228.0 m which is approximately 1.7 to 1.8 m below the base of the creek channel. The profile of the force main is shown on Tetra Tech Drawing P-3465-15-017 included within Appendix A of this TM.

Trek Geotechnical undertook a subsurface geotechnical investigation between August 20 and 22, 2014 to determine the subsurface ground and groundwater conditions north and south of the former culvert crossing location. The findings of this investigation are discussed in further detail within Section 3.0 of this TM. A total of twelve (12) test holes had been drilled at the site as part of Trek's investigation, with two (2) test holes drilled north of Ness Avenue within the east and west bank of Sturgeon Creek.

Figure 2 - Site 5 Location of 250 mm PVC Force Main

1.3.2 Site 6: Fort Garry/St. Vital

- Primary Key Asset: 600 mm Feeder Main & 1,650 mm Branch II Aqueduct.
- Secondary Key Asset: 700 and 800 mm HDPE Sewer Interceptor Pipes.

Site 6 is located along the Red River at the Bishop Grandin Bridge crossing in the south of Winnipeg. The crossing consists of two bridges which form part of the Bishop Grandin Boulevard. The general location of the site is shown in Figure 3.

AECOM has previously undertaken a geotechnical investigation along the eastern riverbank slopes in 2013 to assess the potential risks of slope instability with respect to the 800 mm Interceptor Sewer. It was concluded that slope conditions were significantly impacted when assessed under short term conditions (i.e., rapid drawdown) which could potentially result in a slope failure engaging the existing interceptor sewer within the eastern riverbank slopes. The report recommended placement of stone rip-rap in-conjunction with slope regrading to mitigate the adverse effects of rapid drawdown on the bank stability. This work was completed in spring of 2014, along with repairs to the 800 mm interceptor on the eastern bank. Records of this work are included in Appendix B.

Figure 3 - Site 6 - General Site Location

The existing 600 mm Feeder Main and BRII Aqueduct cross the river at an approximate elevation of 210.3 m (690 feet) below the river channel and rise to an approximate elevation of between 226.59 and 226.92 m within the riverbank slopes adjacent the river crossing. As-built records indicate that the Feeder Main and Aqueduct were installed within bedrock below the river channel. Elsewhere, the Feeder Main and Aqueduct rise within the riverbank slopes away from the channel.

In addition to the Feeder Main and Aqueduct, the much shallower 700 and 800 mm sewer interceptor pipes cross the river channel from the east at an approximate inferred invert elevation of between 218.0 and 219.5 m. The interceptor sewers in turn rise significantly within the slope to an approximate elevation of 224.4 m. The approximate locations of the buried sewer interceptor are shown on the as-built records contained within Appendix B.

2. Field Inspection

2.1 General

Field Inspection of Site 5 and 6 was undertaken on July 7, 2017 by AECOM personnel to document and photograph site conditions (topography, evidence of instabilities, vegetation, etc.). The inspection was also performed to visually evaluate the river/creek bank slopes to support the subsequent preliminary slope stability analysis. The findings of the field inspection have been incorporated into the subsurface ground model as described in Section 4.

Photographs taken during the field inspection are included in Appendix C.

2.2 Site Surveys

Topographic surveys were not included as part of the geotechnical field program, and as such, all subsequent geotechnical analyses have been based on previous topographic surveys, LIDAR information and previous studies conducted within the area (specifically along the sewer/water system). The positions of known sewer and water systems have been inferred from as-built records, and incorporated into the geotechnical analysis.

3. River/Creek Bank Characterization

The system of bank characterization as detailed in the City of Winnipeg Waterway Authority's *Riverbank Characterization Study* (May 2000) has been adopted for characterizing the subject river/creek bank slope within the two project sites.

3.1 Site Reconnaissance

Photographs taken during the course of the field inspection visit are presented as Appendix C enclosed within this Technical Memorandum (TM). A summary of the observations noted during the site reconnaissance is included within Appendix D.

3.2 Site 5 - Heritage Park

The site is located slightly upstream of the creek bend, yet the profile of the creek bank is mostly unaffected north of the bridge crossing, where south of the bridge the creek begins to meander. The creek bank slopes are subject to the effects of both erosion and deposition closer to the bridge crossing as the banks begin to adjust their shape and profile prior to the creek bend further downstream (as illustrated in Figure 1). As a consequence, along the eastern creek bank erosional processes will be dominant, and along the western bank deposition will mostly occur. As per the Riverbank Characterization Study (May 2000), the site is classified as *Transition Banks*. Transition banks are located in areas leading into convex or concave sections of the river (and include straight river stretches). These areas are typically characterized by both shallow and deep seated failures. However, given that part of the eastern and western creek bank slopes near to the existing bridge crossing were recently stabilized, these slopes are classified as “Altered Banks”.

No significant evidence of slope movement was identified during the field inspection, but small shallow retrogressive failures may result in deeper seated failures if erosion is not kept in check.

3.2.1 Creek Bank Slopes

3.2.1.1 Eastern Creek Bank Slopes

- The east creek bank slope appears to be at a profile of between 3 to 4(H):1(V). The profile of the slope appears to steepen closer to the bridge crossing.
- No evidence of global slope instability was recorded within the eastern creek bank, however the slope of the creek bank is moderately steep based upon visual assessment.
- The slope profile becomes flatter further north away from the bridge crossing.
- An erosional scarp at the toe of the creek bank slope was observed during the inspection. The erosional feature measured approximately 300 mm in height, and was concentrated to a small area near the bridge crossing.
- Stone rip-rap was present along the eastern creek bank up to an approximate distance of 5 m north of the bridge crossing. The rip-rap generally appeared in good condition and was free of debris and silt.
- No indications of slope bulging, soil creep or tension cracking were visible at the time of field inspection.
- No animal burrows or infestations were noted within the creek bank slope.

3.2.1.2 Western Creek Bank Slopes

- The west creek bank slope was estimated at a profile of 3 to 4(H):1(V) between the existing pathway and the crest of the creek bank slope. Generally below the crest of the creek bank slope (down to the toe), the profile was approximately 3(H):1(V).
- Soil desiccation and soil cracking were visible in localized areas between the pavement and the toe of the creek bank slope. Areas of desiccation cracking appeared to be mostly confined to areas around the bridge crossing location. Soil desiccation is likely attributed to seasonal moisture changes and not to slope instability.
- Erosion at the toe of the creek bank slope (within the study area) was noted directly adjacent to the bridge crossing location at the time of the field inspection. The erosion was characterised by a 300 mm high scarp extending to a height of 450 mm further north of the bridge crossing. No further signs of global slope movement were noted up slope of the toe scarp.
- Stone rip-rap was present along the western creek bank up to an approximate distance of 5 m north of the bridge crossing. The rip-rap generally appeared in good condition and was free of debris and silt.
- No indications of slope bulging, soil creep or tension cracking were visible at the time of field inspection.
- No evidence of animal burrows or infestations was noted within the creek bank slope.

3.2.2 Existing Structures

3.2.2.1 Eastern Creek Bank

- An existing lift station is located along the eastern creek bank of the site, and is set back approximately 10 to 15 m from the crest of the creek bank slope. No indication of movement or distortion to the structure was noted at the time of the field inspection. Minor ground movement below the concrete apron of the building was evident at the time of the inspection, but this is likely owing to ground settlement.
- Hydro power poles are present north of the lift station, and appeared to be generally vertical in inclination and in good condition.
- The existing pavement structure adjacent the lift station did not show signs of cracking or movement.

3.2.2.2 Western Creek Bank

- The fence line of the adjacent property appeared to be in reasonable condition, and in places the fence line leaned slightly towards the creek.
- The pavement surface (recently constructed in 2016) did not indicate any signs of distress or movement.
- The pavement structure appeared to incorporate drainage pipes (220 mm OD PVC pipe) sloped from the west to the east so as to discharge water down the creek bank slope from the adjacent structure.
- A geotechnical standpipe installation was still present (as part of the TREK 2014 geotechnical investigation program) within the western creek bank slope.

3.2.3 Vegetation

3.2.3.1 Eastern Creek Bank

- The eastern bank of the creek is generally characterised by a well-maintained grassed lawn area between the existing lift station and the creek bank slope. Several trees were also noted in and around the lift station. No indication of tree movement was visible (i.e., trees were generally vertical).

3.2.3.2 Western Creek Bank

- Several mature trees were present within the confines of the adjacent property; elsewhere the vegetation cover consisted of maintained grass lawn.
- Vegetation cover becomes sparse to the east of the existing pavement (at a distance of approximately 50 m north of the bridge crossing).
- Localized dense patches of brush/shrubs were identified between areas of little to no vegetation cover between the edge of the pavement and the creek bank slope.

3.3 Site 6 - Fort Garry/St. Vital

The Fort Garry/St. Vital Feeder Main is located in a large alluvial flood plain that extends just south of Bishop Grandin Boulevard to approximately 1 km north of the project site. The alluvial soils that form the flood plain are comprised mainly of beds of clay, silt, sand, and gravel, which were deposited either directly on glacial till or on a layer of lacustrine clay. The alluvial deposits are exposed over the full height of the subject riverbank throughout the study area.

Based on site observations obtained through the field inspection, the riverbanks within the study area are classified as an erosion controlled bank in accordance with the criteria described in the City of Winnipeg Waterway Authority's *Riverbank Characterization Study* (May 2000). However, given that the lower slopes of the eastern riverbank were recently stabilized (between the Fort Garry Bridges), these slopes are classified as "Altered Banks". Remedial works have been performed to alter the condition of the banks (i.e., control toe erosion), yet the riverbanks within the subject site are largely classified as "Erosion Controlled Bank". This classification is attributed where riverbank loss is primarily the result of erosion along the edge of the river at summer water levels. Bank failures are typically localized and occur by toppling of over steepened riverbank slopes created as a result of excessive toe erosion.

Shallow failures or sloughing of the bank face often follow floods or heavy precipitation which can saturate the bank and reduce the strength of the soil. There is no evidence of deep seated or rotational failures along either the eastern or western riverbank slopes at the crossing location, but the rate of bank loss may be accelerated following heavy precipitation or rapid drawdown events. For the most part, the lower portions of the riverbank slopes are protected with stone rip-rap which will positively contribute to reducing erosion to the near surface soils during a heavy rainfall event. Unprotected slopes may experience some loss of material through progressive erosional processes, but through proactive monitoring this risk can be managed.

3.3.1 River Bank Slopes

3.3.1.1 Eastern Riverbank Slopes

- Between the two bridges (within the study area) the east riverbank visually appeared to be at an approximate slope profile of 3 to 4(H):1(V), with the remainder of the slope non-visible below the water line. No visual indication of global slope instability was observed within the riverbank above the water line at this location.
- Based on visual inspection and review of the as-built records, stone rip-rap was placed along the lower riverbank slopes to mitigate the effects of erosion. At the time of inspection, the top of the rip-rap was only slightly exposed above the water line, but otherwise the rip-rap was generally not visible for inspection.
- Some evidence of minor erosion was noted within the riverbank slopes between the two bridges (within the study area) above the existing rip-rap coverage. The erosion appeared to be concentrated to small areas of the slope at the water’s edge, and was not visible further up-slope towards the sidewalk pavement.
- No evidence of animal burrows or infestations was noted within the river bank slopes. Animal burrows were frequently observed along the ground surface east of the sidewalk pavement.
- Stone rip-rap placed around the bridge piers was not noted to extend beyond the limits of the bridge by more than several meters. Considerably less rip-rap was present around the northern bridge pier as compared to the south bridge pier. Some loss of rip-rap around the bridge abutments has exposed the underlying alluvial soils.
- Minor slope erosion was observed within the soft alluvial soils exposed above the river water level to the south of the south bridge pier (i.e., outside of the limits of the study area) in locations were there was an absence of stone rip-rap. Slope erosion was more pronounced to the north of the north bridge pier (also outside of the limits of the study area) where erosional scarps were observed. These scarps measured approximately 350 to 500 mm in vertical height and were concentrated to lower areas of the riverbank slopes.
- Washed-up trees and other debris were identified along the riverbank slopes adjacent to the north bridge pier. It is likely that this position along the riverbank slope indicates the river’s recent high water level.
- Ground elevations above the riverbank (between the two highway approach embankments) generally sloped west towards the sidewalk pavement and riverbank. Surface water from the approach embankments generally appeared to shed towards low spots located in between the bridges and then westwards towards the river.

3.3.1.2 Western Riverbank Slopes

- West of the asphalt pavement (orientated north to south), the ground surface between Bishop Grandin Boulevard gently falls east towards the bridge abutments. The slope profile changes at a point almost in line with the bridge abutments within the study area, sloping sharply towards the sidewalk pavement. The slope has an approximate profile of 3.5 to 4(H):1(V), then flattens out closer towards the sidewalk further east.

- The approach embankments are slightly higher in elevation than the ground between the east and westbound lanes of the highway, resulting in a general low point. This low point however is generally consistent in elevation to adjacent ground with the exception of the approach embankments.
- The crest of the riverbank slope was visible several meters from the edge of the sidewalk, and the surface of the riverbank was visible for approximately 4.5 to 6 m until intercepting the water's edge further downslope. The surface of the riverbank slope was generally covered in shrubs and bushes; and the profile of the riverbank slope was estimated at 1.5 to 2(H):1(V).
- Stone rip-rap was present surrounding the two bridge abutments and for an approximate length of 20 m between the two highway bridges along the shoreline within the study area. The rip-rap was generally large (greater than 600 mm) and in places appeared to be ravelling and moving down slope into the river. Some loss of rip-rap around the bridge abutments has exposed the underlying alluvial soils.
- Erosion has resulted in gullying and material loss in and around the rip-rap as a consequence of surface water flow from the culverts west of the riverbank. Gullies measuring a depth of up to 400 mm were recorded.
- Erosional scarps were noted near to the toe of the riverbank slope between the two bridges and measured approximately 100 mm in vertical height. Erosional scarps at the toe were most prevalent in areas without armouring.
- Tension cracking was noted near the slope crest where the slope profile was steeper than approximately 2(H):1(V). Currently, the tension cracking was noted to be isolated and was not a general characteristic of the slope as a whole. No indications of relative vertical movement (slumping or rotational movement) were evident within the areas of tension cracking.
- No evidence of animal burrows or infestations was noted within the creek bank slope.

3.3.2 Existing Structures

3.3.2.1 Eastern Riverbank

- Several structures were observed along or near to the eastern riverbank slope within the study area. These include:
    - Two (2) Bridge Structures- including bridge superstructure and substructures (abutments and piers);
    - Sidewalk Pavement;
    - Valve Chamber;
    - 350 mm Diameter Culverts (3);
    - Hydro Tower (1); and
    - Geotechnical Instrument (1) - Groundwater Monitoring Well.

- The ground immediately surrounding the hydro tower appeared to be undermined due to a combination of animal burrows and over-steepened side slopes. The base of the concrete footings were exposed along the underside of the tower base locations. The foundation fill used to elevate the towers was sloped at an approximate profile of 2(H):1(V), and showed signs of slope bulging near the toe. The towers are removed from the riverbank slopes in the study area and are deemed not to have any direct impact upon riverbank stability.
- The existing sidewalk pavement showed signs of distress in some locations within the study area adjacent to the riverbank crest. Cracks within the asphalt surface were orientated in a north south direction running parallel to the riverbank crest. These cracks appear to have been patched/repaired and no vertical or horizontal displacement was noted as a result of the cracking.

- Corrugated Steel Pipe (CSP) culverts have been installed at the foot of the bridge abutments and are surrounded by grouted stone rip-rap. Elsewhere above the bridge abutments and above the bridge piers, the stone rip-rap was placed in a non-grouted form. Soil erosion was more evident in areas directly opposite the rip-rap.

3.3.2.2 Western River Bank

- The following structures were observed within and adjacent to the study area:
    - Two (2) Bridge Structures- including bridge superstructure and substructures (abutments and piers);
    - Lift station (and associated valve chambers);
    - Monitoring station(s);
    - Drainage Culverts (4);
    - Hydro Towers (2); and
    - Sidewalk Pavement.
- All structures outlined above visually appeared in good condition.

3.3.3 Vegetation

3.3.3.1 Eastern Riverbank

- The riverbank slope between the two bridges was generally covered with bushes and light vegetation growth from the riverbank toe to the edge of the sidewalk pavement.
- The slopes above the sidewalk pavement largely consisted of maintained lawns and were devoid of any large trees or denser vegetation growth.
- Large mature trees were observed directly adjacent to the study area (north and south of the bridges) along the riverbank crest. These larger trees generally followed the alignment of the slope crest, and appeared to be leaning toward the river.

3.3.3.2 Western River bank

- Several large mature trees were identified in clusters near the toe and crest of the slope, elsewhere the vegetation cover consisted of shrubs and bushes. West of the pavement structure (above the riverbank crest), the vegetation comprised of maintained grass lawn.
- There is no indication of significant vegetation movement to suggest slope instability within the study area.

4. Stability Assessment

4.1 General

The primary objective of the preliminary slope stability analysis is to assess the existing stability of the river/creek bank slopes, and to determine if prevailing slope conditions place the buried sewer/water systems at risk. Preliminary slope stability analyses have been undertaken for Site 5 (Heritage Park) and Site 6 (Fort Garry/St. Vital) crossings.

4.2 Methodology

4.2.1 Stability Analysis

Two-dimensional slope stability models were developed using GeoStudio 2007 (Slope/W) based on the Limit Equilibrium method of analysis. The riverbank geometries were established based on recent LIDAR survey provided by the City, as-built record drawings and available information contained within the geotechnical engineering reports.

The soil stratigraphy for the stability models was derived from geological maps and available test hole information contained within the geotechnical engineering reports. Assumptions were necessary to facilitate the analysis where local or detailed information was limited. The pipe location at each crossing was taken from the Record Drawings, and the pipe profiles within the slope stability models were inferred where necessary.

River elevations were based on information sourced from the City of Winnipeg's online database (http://www.winnipeg.ca/publicworks/pwddata/riverlevels/ accessed December 2017 to January 2018). River elevations were adjusted to reflect high and low water events as shown in Table 5. Creek water levels for Sturgeon Creek were based on the representative levels referenced within the Trek Geotechnical Report (October 2014).

Upon establishing a slope stability model for each location, the assessment was performed using Morgenstern-Price's general method of slices, which satisfies both moment and horizontal force equilibrium. 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 a more complex analysis method.

As part of the analysis, the following slip surfaces were considered of interest and are presented graphically as Figure 4. 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 riverbank and would likely compromise the global stability of riverbank. Only slip surfaces with a depth of 0.5m or greater have been assessed in this case.
- **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/or 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 localized to the toe of the slope and 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.

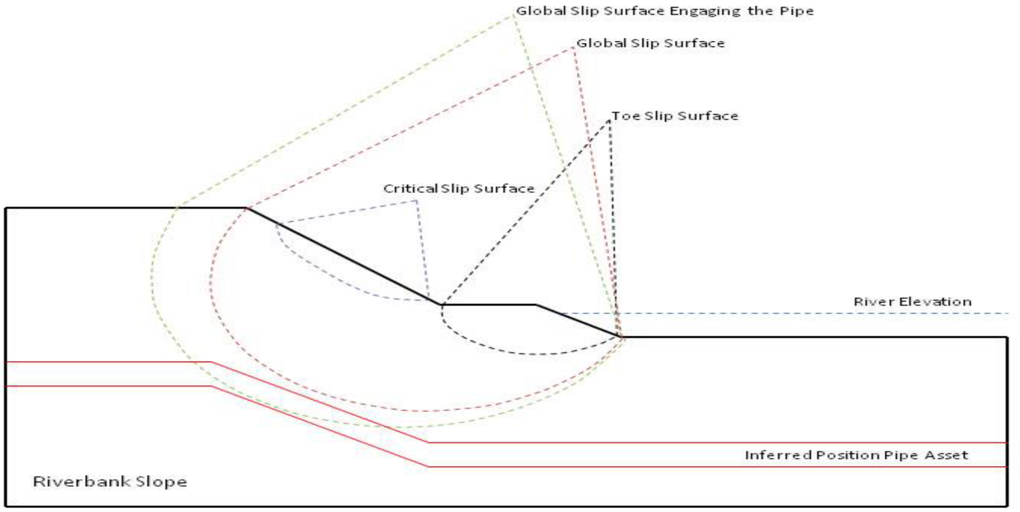


Figure 4 - Assessed Slip Surfaces

4.2.2 Slope Stability Cases

The following loading conditions have been considered as part of the preliminary slope stability analysis, and are outlined below.

- Long-Term Design Condition; and
- Short-Term Design Condition (Rapid Drawdown).

An acceptable FS can be defined between 1.3 and 1.5 depending on whether short-term or long-term conditions are being considered, and based on other factors including but not limited to associated impact of instability, risk management approach and related cost to improve the stability. For purposes of this TM and consistent with acceptable design practice, river/creek stability is assessed under the following design conditions and the corresponding target factor of safety (FS) against slope instability.

- Long Term at FS >= 1.50.
- Short Term rapid drawdown at FS >= 1.30.

Rapid drawdown is a state in which the creek or river level against the bank falls rapidly below its normal level, but the piezometeric conditions within the bank slope are elevated above normal steady state parameters. The application of this condition is further described in Section 4.2.4 of this TM.

4.2.3 Soil Parameters

Soil strength parameters used in the stability analyses are presented in Table 2 and Table 4 for Site 5 and 6 respectively. Soil parameters were selected based upon review of the geotechnical laboratory data contained within the available geotechnical reports for each site, combined with local knowledge and prior experience.

4.2.3.1 Site 5: Heritage Park

In order to develop the slope stability models, subsurface ground and groundwater conditions were referenced from available information sources (as outlined in Section 1.2 of this TM) as summarised below:

East Creek Bank

- **TH14-05**: TREK Geotechnical (October 2014) *Geotechnical Investigation Ness Avenue at Sturgeon Creek Culvert Replacement, Preliminary Design* (see Appendix E).

West Creek Bank

- **TH14-04**: TREK Geotechnical (October 2014) *Geotechnical Investigation Ness Avenue at Sturgeon Creek Culvert Replacement, Preliminary Design* (see Appendix E).

Further information regarding the subsurface ground conditions are shown on the as-built record **Drawings Ref. B243-15-006** and **P-3465-15-017** as shown in Appendix E.

A summary of the adopted subsurface ground conditions is outlined in Table 1 below.

Table 1 - Summary of Ground Conditions at Site 5 (Heritage Park)

East Creek Bank (TH14-05)			West Creek Bank (TH14-04)		
Stratum	**Elevation to Base (m)**	**Thickness (m)**	**Stratum**	**Elevation to Base (m)**	**Thickness (m)**
Alluvial Clay	231.1	1.00	**Alluvial Clay**	229.4	1.50
Lacustrine Clay	226.5	4.60	**Lacustrine Clay**	226.7	2.70
Glacial Till	225.6	0.90	**Glacial Till**	225.6 (1)	1.10
Bedrock-Calcareous Mudstone	220.4 (1)	5.20			

Notes: (1) - Depth to base not proven.

Soil Parameters

Fully softened (near residual) shear strength values were assigned to the Alluvial and Lacustrine Clays. The depth of bedrock was referenced from as-built records. The bedrock was treated as an *impenetrable layer* within the analyses, and therefore was not assigned a shear strength value.

The following soil parameters were adopted as part of the slope stability analysis at Site 5.

Table 2 - Site 5 - Heritage Park- Soil Strength Parameters for Stability Analysis

Stratum	Moist Bulk Unit Weight (kN/m^3)	Internal Angle of Friction (Degrees)	Cohesion (kPa)
Clay Fill	18	20	0.0
Alluvial Clay	18	18	2.5
Lacustrine Clay	18	14	5.0
Glacial Till	21	30	10.0

4.2.3.2 Site 6: Fort Garry/St. Vital

In order to develop the slope stability models, subsurface ground and groundwater conditions were referenced from available information sources (as outlined in Section 1.2 of this TM) as summarised below:

East Riverbank

- **TH13-01** and **02**: AECOM (December 12, 2013), *Preliminary Geotechnical Assessment Fort Garry Interceptor Sewer Crossing at the Red River* (see Appendix F).
- **H-1001, H-1002** and **TH 4, TH402 and TH 403**: Klohn Leonoff Consultants Ltd (April 12, 1976), *Report on Sub-Soils Investigation for Fort Garry- St. Vital Corridor, Winnipeg, Manitoba* (see Appendix F).

West Riverbank

- **H-1003**, **H-1004, TH 401**: Klohn Leonoff Consultants Ltd (April 12, 1976), *Report on Sub-Soils Investigation for Fort Garry- St. Vital Corridor, Winnipeg, Manitoba* (see Appendix F).

Further information regarding the subsurface ground conditions are shown on the as-built record **Drawings Ref. B-5092-205** and **B-5092-206** as shown in Appendix F.

A summary of the adopted subsurface ground conditions is outlined in Table 3 below.

Table 3 - Summary of Ground Conditions at Site 6 (Fort Garry/St. Vital)

East River Bank			West River Bank		
Stratum	**Elevation to Base (m)**	**Thickness (m)**	**Stratum**	**Elevation to Base (m)**	**Thickness (m)**
Alluvial Clay	223.10	6.00	**Alluvial Clay**	223.33	4.11
Glacio-Lacustrine Clay	217.93- 219.57	3.90-13.72	**Glacio-Lacustrine Clay**	217.32- 217.93	11.59- 12.50
Glacial Till	216.41 (1)	1.60- 2.00	**Glacial Till**	216.41- 217.17	0.76- 1.52
Bedrock	213.36 (2)	3.05	**Bedrock**	211.01- 214.58	5.40- 6.16

Notes: (1) - Proven only in TH 1001 & 1002; (2) - Depth to base not proven.

Soil Parameters

Fully softened shear strength values were assigned to the Alluvial and Lacustrine Clays. The depth of bedrock was referenced from as-built records. The bedrock was treated as an *impenetrable layer* within the analyses, and therefore was not assigned a shear strength value.

The following soil parameters were adopted as part of the slope stability analysis at the site.

Table 4 - Site 6 - Fort Garry/St. Vital- Soil Strength Parameters for Stability Analysis

Stratum	Moist Bulk Unit Weight (kN/m^3)	Internal Angle of Friction (Degrees)	Cohesion (kPa)
Alluvial Clay*	18	18	5.0
Lacustrine Clay	18	14	5.0
Glacial Till	21	30	10.0

Notes: *- Inclusive of Upper and Lower Alluvium Clay.

4.2.4 River/Creek Water Levels

River levels for the Red River used within the slope stability analysis for Site 6 are summarized in Table 5 below. River levels adopted as part of the analysis have been referenced from the City of Winnipeg's on-line database (http://www.winnipeg.ca/publicworks/pwddata/riverlevels/) and from previous geotechnical reports associated with each site. Creek levels for Sturgeon Creek (Site 5 - Heritage Park) have been referenced from previous studies performed at the site and these are outlined in Table 5 below.

Table 5 - Summary of River/Creek Water Levels

Water Course	Site Reference	Normal Winter Water Level (NWWL) (m)	Normal Summer Water Level (NSWL) (m)	Rapid Drawdown-Meters (m) (1)	Reference
Sturgeon Creek	5 - Heritage Park	231.2	232.0*	Elevated phreatic surface of 0.8 m.	• As-built Records. • TREK Geotechnical Report (October 10, 2014)
Red River	6 - Fort Garry/St. Vital	222.02	223.74	Elevated phreatic surface of 0.72m.	• AECOM Canada Ltd (December 12, 2013)- Preliminary Geotechnical Assessment

Notes:*- Assumed; (1) - Based on a reduction in the phreatic surface from summer to winter water levels.

Groundwater conditions applied to the models for each slope stability analysis are reflective the river/creek levels as outlined in Table 5.

5. Slope Stability Results

The results of the preliminary slope stability analysis performed as part of this study as described in Section 4 are outlined below.

5.1 Current Riverbank Stability

Based upon the established subsurface ground model (incorporating all available topographic information) taken from along the pipe alignment at each site, and assessment was completed in terms of both localized stability and the probability of global failure engaging the sewer/water pipe. The Factors of Safety (FS) derived from this assessment are presented in Table 6 and Table 7 for Site 5 and 6 respectively.

Table 6 - Site 5 - Current Slope Stability of Creek Bank Slope along Pipe Alignment

River Conditions	Global Slip Stability (GS)		Global Stability Engaging the Pipe (GS+P)		Toe Slip Surface (TS)		File Output Reference	
	East	West	East	West	East	West	East	West
Normal Winter Water Level (NWWL)	1.6	1.9	1.6	1.9	1.6	1.9	G-01	G-02
Normal Summer Water Level (NSWL)	1.7	2.1	1.7	2.1	1.7	1.7	G-03	G-04
Rapid Drawdown	1.5	1.8	1.5	1.8	1.5	1.4	G-05	G-06

Table 7 - Site 6 - Current Slope Stability of River Bank Slope along Pipe Alignment

River Conditions	Global Slip Stability (GS)		Global Stability Engaging the Pipe (GS+P)		Toe Slip Surface (TS)		File Output Reference	
	East	West	East	West	East	West	East	West
Normal Winter Water Level (NWWL)	1.5	1.4	1.5*	1.4*	1.5	1.4	G-07	G-08
Normal Summer Water Level (NSWL)	1.7	1.5	1.7*	1.5*	1.7	1.5	G-09	G-10
Rapid Drawdown	1.5	1.3	1.5	1.3*	1.5	1.3	G-11	G-12

Notes: *- Intercepts the 700 and 800 mm HDPE Interceptor Sewer.

6. Conclusions and Recommendations

6.1 Conclusions

Based on the results of the preliminary slope stability assessment for Sites 5 and 6, the following general conclusions are drawn:

- Site 5 (Heritage Park): The 250 mm force main is not currently at risk due to slope instability within the creek bank slopes based upon known available information. Current global slope stability FS values are greater than 1.5 and 1.3 for long and short term conditions respectively.
- Site 6 (FGSV): The 600 mm feeder main and aqueduct are not currently at risk of slope instability whereby a failure surface with FS less than 1.5 would engage the feeder main or Aqueduct. However, the 700 mm and 800 mm HDPE interceptor sewers are at risk of being engaged by a failure surface with an FS less than 1.5 (between 1.40 and 1.45) during periods of low water level (i.e., during the winter months).

6.2 Recommendations

The findings of the slope stability analysis indicate the following:

Site 5 - Heritage Park

1. Periodic monitoring of the creek bank slopes and the immediate area is suggested as part of regular maintenance, but specifically should be undertaken following heavy rainfall and flooding events.
2. Areas of erosion within the creek banks should be of specific interest, and should be actively documented to ensure that material is not lost from within the slopes resulting in a progressive reduction of slope stability.
3. Excavation work within the creek bank slopes or immediate area should be assessed for its potential impacts upon existing slope stability prior to commencement.

Site 6 - Fort Garry/St. Vital

1. Under current observable conditions, the eastern river bank slope has a FS for both long- and short-term conditions greater than the required design criteria. No further action is required unless the slope conditions deteriorate or significantly different hydraulic conditions (river level) are experienced.
2. The FS values for the western riverbank slope is marginally less than the required long-term slope stability design criteria under winter river level conditions. Whilst the existing FS values do not meet current industry accepted design standards, the risk of immediate slope failure is considered low. A progressive reduction in the FS of the riverbank slope through erosion should be monitored regularly to mitigate the risk of reduction in slope stability through erosion.
3. Consideration of slope improvements within the western riverbank should be assessed on a cost/benefit basis. Should slope improvements be considered, upgrades in the form of regrading and/or placement of stone rip-rap would likely only provide a five to ten percent improvement in slope stability as compared to existing levels. Unless deemed critical, periodic visual inspection should be sufficient in the short term until such time that existing slope stability falls below a FS of about 1.3. Should the need for slope improvement to be required in the short term, consideration may be given to slope regrading and placement of stone rip-rap.

6.3 Limitations of Slope Stability Analyses

Slope stability analysis has been performed for each site based upon in some cases limited topographical information (i.e., LIDAR data and as-built record information), limited geotechnical information (specifically the west bank of Site 6), limited pipe invert/condition information and positional information. The results should therefore be viewed as preliminary with the exception of the east riverbank for Site 6 which builds further on geotechnical analysis performed by AECOM in 2013.

The primary objective of the stability assessment was to establish the level of risk presented to the buried water and sewer systems as a result of slope instability. The risk of slope instability provides an indication of the failure surface with the lowest Factor of Safety (FS) likely to engage the pipe.

7. Closing

The findings and conclusions contained within this report were based on the results of as-built records in conjunction with information contained within previous studies with an extrapolation of soil and groundwater conditions. If conditions are encountered that appear to be different from those shown within the existing documentation and described in this report, or if assumptions stated herein are not in keeping with the design, this office should be notified in order that the recommendations can be review and justified, if necessary.

Soil conditions by their nature can be highly variable across a site. If conditions are encountered that appear to be different from this identified within this report, or if the assumptions stated herein are not in keeping with the design, this office should be notified in order to review and adjust (if necessary) the material contained within report.

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

Respectfully submitted,
AECOM Canada Ltd.

Prepared by:

for Alexander Hill, P.Geo. (B.C), FGS
Geotechnical Engineering

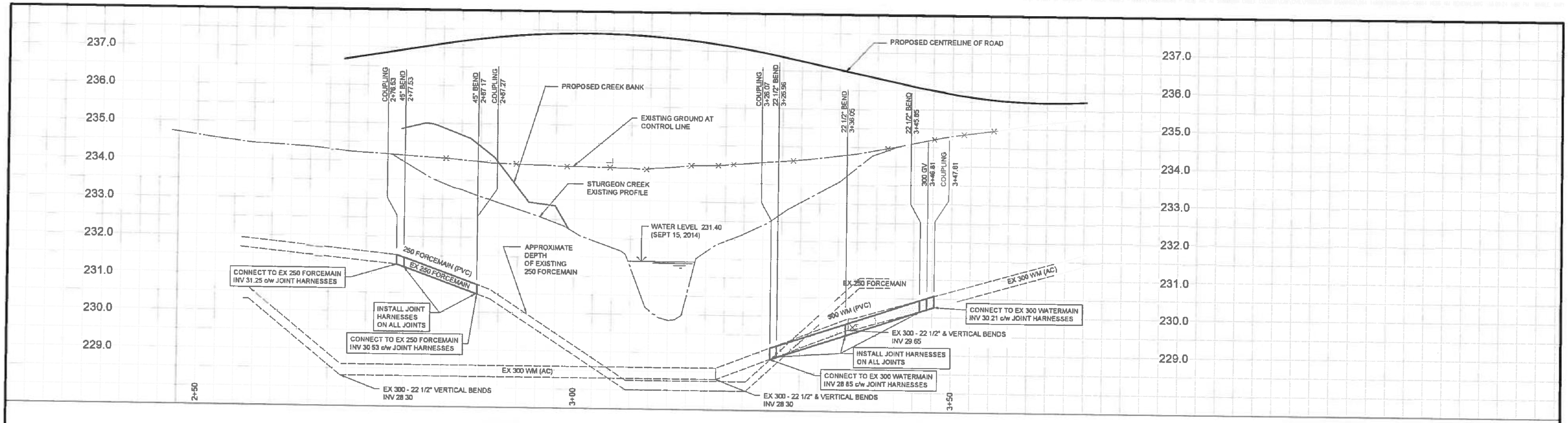
Reviewed by:

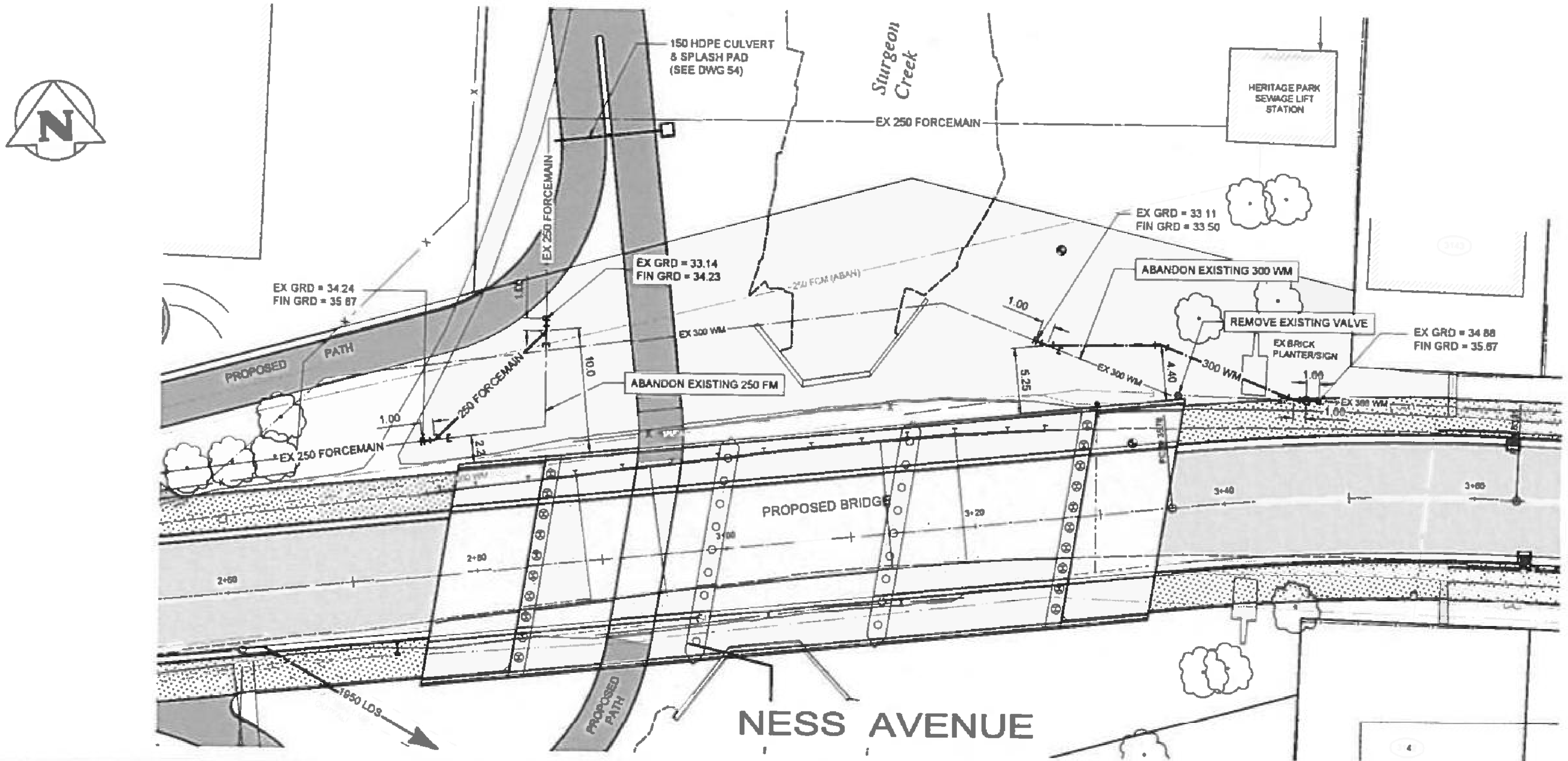
Elliott E Drumright

Elliott Drumright, PhD, P.E
Associate Geotechnical Engineer

Appendix A

Site 5 – Heritage Park: As-built Records





CONSTRUCTION NOTES

1. LOCATION OF ALL SEWER AND WATER LINES TO BE CONFIRMED IN THE FIELD
2. SEWER AND WATER SERVICES SHOWN ON DRAWINGS ARE APPROXIMATE ONLY
3. ALL FITTINGS ON WATERMAINS AND FORCEMAINS REQUIRE RESTRAINING DEVICES

GENERAL NOTES

1. CHAINAGE IS ALONG CONTROL LINE OF NESS AVENUE.
2. SEE SPECIFICATIONS FOR DETAILED TEST HOLE INFORMATION.

APEGM
Certificate of Authorization
TETRA TECH WEI Inc.
No. 5313 Date: April 30, 2016

150 WM	WATERMAIN	150 WM	→	SWALE	→	—	RIGHT-OF-WAY	
400 FM	FEEDERMAIN	400 FM	○	MANHOLE	●		GAS	
300 LDS	LAND DRAINAGE SEWER	300 LDS	□	CATCHBASIN	■		HYDRO	
250 WWS	WASTEWATER SEWER	250 WWS	⊞	HYDRO BOX			MTS	
300 CS	COMBINED SEWER	300 CS	•	HYDRO POLE			DETECTABLE WARNING TILE	▭
200 FCM	FORCEMAIN	200 FCM	•–•	LIGHT STANDARD			ELEVATION	32.123
	JUNCTIONS		←	ANCHOR			SIDEWALK	
⌖	HYDRANT	✦	○	TREE			CURB	
●	VALVE	●	⌖	SURVEY BAR			ASPHALT	
♂	CURB STOP		▣	SIGNAL CONTROLLER			CONCRETE	
EXISTING	LEGEND-PLAN	PROPOSED	EXISTING	LEGEND-PLAN	PROPOSED	EXISTING	LEGEND-PLAN	PROPOSED

LOCATION APPROVED
UNDERGROUND STRUCTURES

SUPV. U/G STRUCTURES COMMITTEE DATE

NOTE:
LOCATION OF UNDERGROUND STRUCTURES AS SHOWN ARE BASED ON THE BEST INFORMATION AVAILABLE. BUT NO GUARANTEE IS GIVEN THAT ALL EXISTING UTILITIES ARE SHOWN OR THAT THE GIVEN LOCATIONS ARE EXACT. CONFIRMATION OF EXISTENCE AND EXACT LOCATION OF ALL SERVICES MUST BE OBTAINED FROM THE INDIVIDUAL UTILITIES BEFORE PROCEEDING WITH CONSTRUCTION.

B.M.
ELEV.

NO.	REVISIONS	DATE	BY
0	ISSUED FOR TENDER	15.09.24	KJM

TETRA TECH

DESIGNED BY GWC
CHECKED BY KJM
DRAWN BY GMD
APPROVED BY EFS
HOR. SCALE: 1 : 250
VERTICAL: 1 : 50
DATE 15.09.24
ACCEPTED BY DATE
ORIGINAL DRAWING SIGNED BY: D.N. BURMEY, P.ENG. 15.09.24
D.N. BURMEY, P.ENG.
BRIDGE PROJECTS ENGINEER

K.J. McRAE
ORIGINAL SIGNED
15.09.24

CONSULTANT DRAWING NO.
1400070800-DWG-C0064

Winnipeg

THE CITY OF WINNIPEG
PUBLIC WORKS DEPARTMENT
ENGINEERING DIVISION

NESS AVENUE AT STURGEON CREEK
BRIDGE CONSTRUCTION

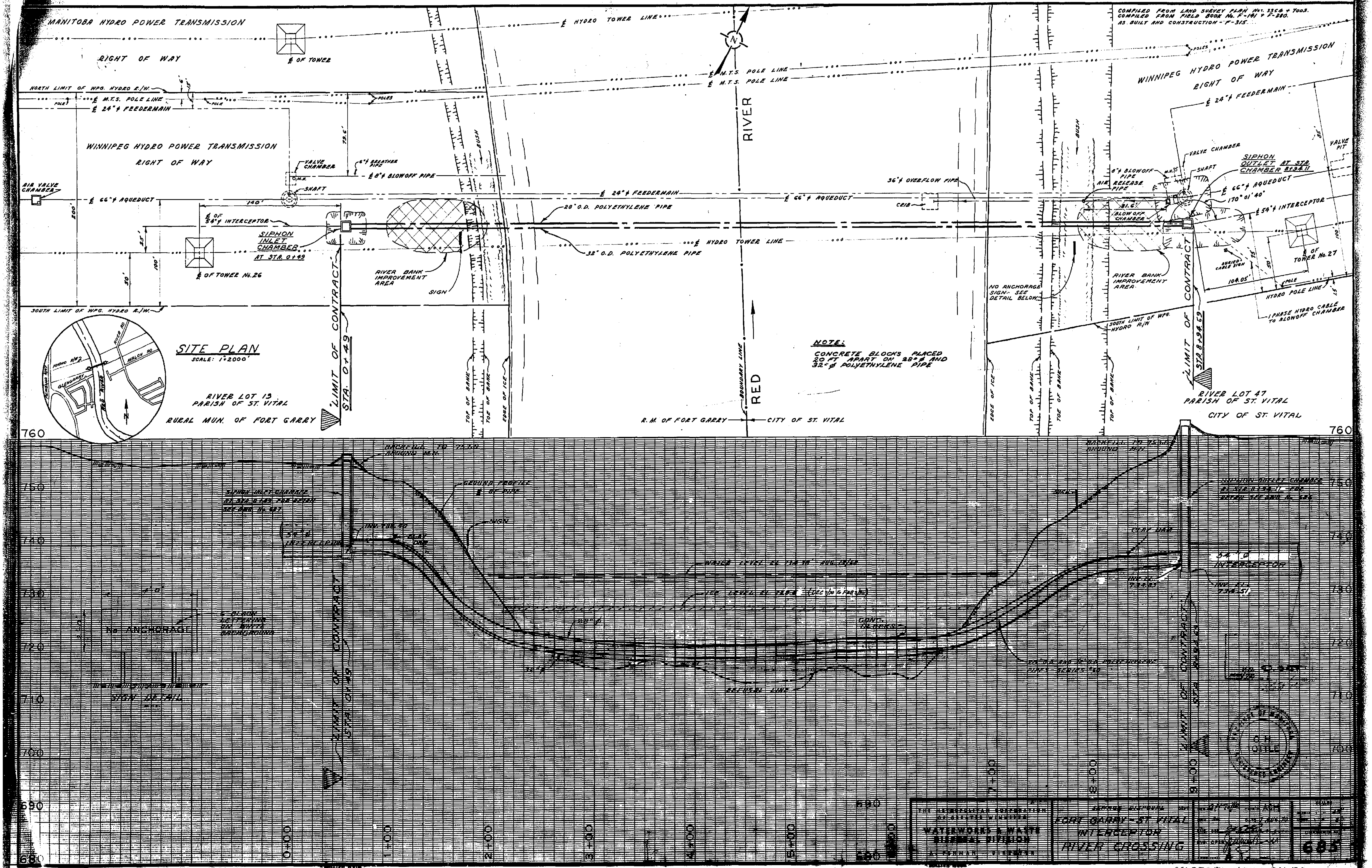
300 WATERMAIN &
250 FORCEMAIN RELOCATION
ON NESS AVENUE

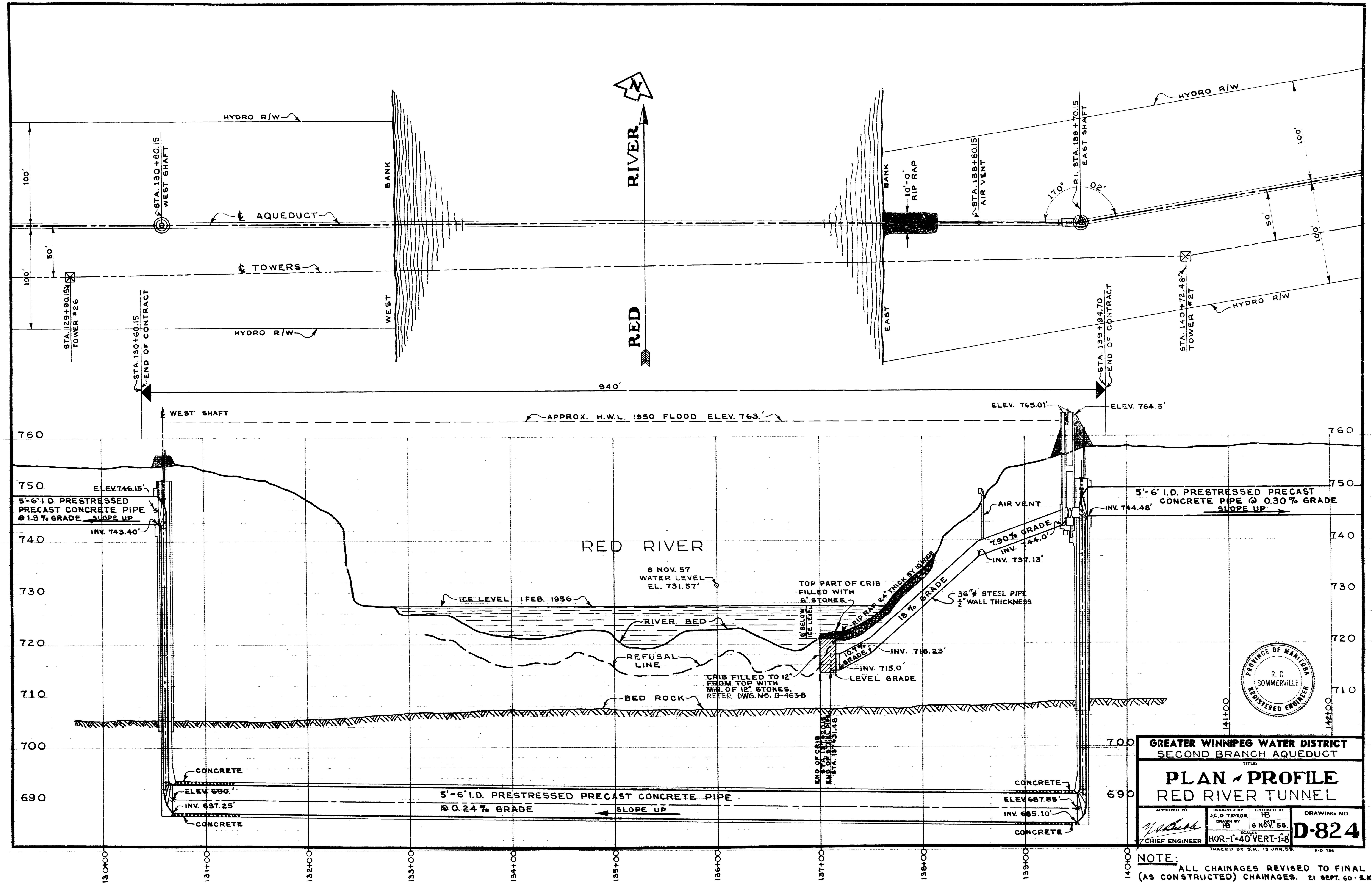
CITY DRAWING NUMBER
P-3465-15-017
SHEET 64 OF 64

64

Appendix B

Site 6 – Fort Garry/St. Vital: As-built Records


MANITOBA HYDRO POWER TRANSMISSION
RIGHT OF WAY
℄ OF TOWER
℄ HYDRO TOWER LINE
COMPILED FROM LAND SURVEY PLAN No. 33C4 + 7003.
COMPILED FROM FIELD BOOK No. F-141 + F-380.
AS BUILT AND CONSTRUCTION - F-315
POLES
WINNIPEG HYDRO POWER TRANSMISSION
RIGHT OF WAY
℄ 24"⌀ FEEDERMAIN
NORTH LIMIT OF WPG. HYDRO R/W.
℄ M.T.S. POLE LINE
℄ 24"⌀ FEEDERMAIN
WINNIPEG HYDRO POWER TRANSMISSION
RIGHT OF WAY
VALVE CHAMBER
6"⌀ BREATHER PIPE
℄ 8"⌀ BLOWOFF PIPE
SHAFT
AIR VALVE CHAMBER
℄ 66"⌀ AQUEDUCT
140'
℄ OF 54"⌀ INTERCEPTOR
SIPHON INLET CHAMBER AT STA. 0+49
℄ OF TOWER No. 26
RIVER BANK IMPROVEMENT AREA
SIGN
BUSH
28" O.D. POLYETHYLENE PIPE
32" O.D. POLYETHYLENE PIPE
℄ 24"⌀ FEEDERMAIN
℄ 66"⌀ AQUEDUCT
℄ HYDRO TOWER LINE
RIVER
RED
BOUNDARY LINE
36"⌀ OVERFLOW PIPE
CRIB
SOUTH LIMIT OF WPG. HYDRO R/W.
LIMIT OF CONTRACT STA. 0+49
TOP OF BANK
TOE OF BANK
EDGE OF ICE
NOTE:
CONCRETE BLOCKS PLACED 20 FT APART ON 28"⌀ AND 32"⌀ POLYETHYLENE PIPE
NO ANCHORAGE SIGN - SEE DETAIL BELOW
6"⌀ BLOWOFF PIPE
AIR RELEASE PIPE
BLOWOFF CHAMBER
VALVE CHAMBER
SHAFT
SIPHON OUTLET CHAMBER AT STA. 8+94.11
℄ 66"⌀ AQUEDUCT
170° 01' 40"
℄ 54"⌀ INTERCEPTOR
℄ OF TOWER No. 27
VALVE PIT
RIVER BANK IMPROVEMENT AREA
LIMIT OF CONTRACT STA. 8+94.69
HYDRO POLE LINE
POLE
104.05'
1 PHASE HYDRO CABLE TO BLOWOFF CHAMBER
SOUTH LIMIT OF WPG. HYDRO R/W
SITE PLAN
SCALE: 1"=2000'
RED RIVER
RIVER LOT 13
PARISH OF ST. VITAL
RURAL MUN. OF FORT GARRY
R.M. OF FORT GARRY
CITY OF ST. VITAL
RIVER LOT 47
PARISH OF ST. VITAL
CITY OF ST. VITAL
760
750
740
730
720
710
700
690
680
GROUND PROFILE ℄ OF PIPE
SIGN
SIPHON INLET CHAMBER AT STA. 0+49 FOR DETAIL SEE DWG. No. 687
54"⌀ INTERCEPTOR
CLAY DIKE
WATER LEVEL
ICE LEVEL
CONC. BLOCKS
54"⌀ INTERCEPTOR
No ANCHORAGE
SIGN DETAIL
SIPHON OUTLET CHAMBER AT STA. 8+94.11 FOR DETAIL SEE DWG. No. 686
LIMIT OF CONTRACT STA. 0+49
LIMIT OF CONTRACT STA. 8+94.69
0+00
1+00
2+00
3+00
4+00
5+00
6+00
7+00
8+00
9+00
THE METROPOLITAN CORPORATION OF GREATER WINNIPEG
WATERWORKS & WASTE DISPOSAL DIVISION
SEWAGE DISPOSAL
FORT GARRY - ST VITAL
INTERCEPTOR
RIVER CROSSING
685
MICROFILMED NOV 76



PROVINCE OF MANITOBA
R. C. SOMMERVILLE
REGISTERED ENGINEER

GREATER WINNIPEG WATER DISTRICT
SECOND BRANCH AQUEDUCT

TITLE:

PLAN & PROFILE
RED RIVER TUNNEL

APPROVED BY	DESIGNED BY	CHECKED BY	DRAWING NO.
Y. Bubb CHIEF ENGINEER	J.C.D. TAYLOR	HB	D-824
	DRAWN BY HB	DATE 6 NOV. 58.	
	SCALES HOR.-1"=40' VERT.-1"=8'		

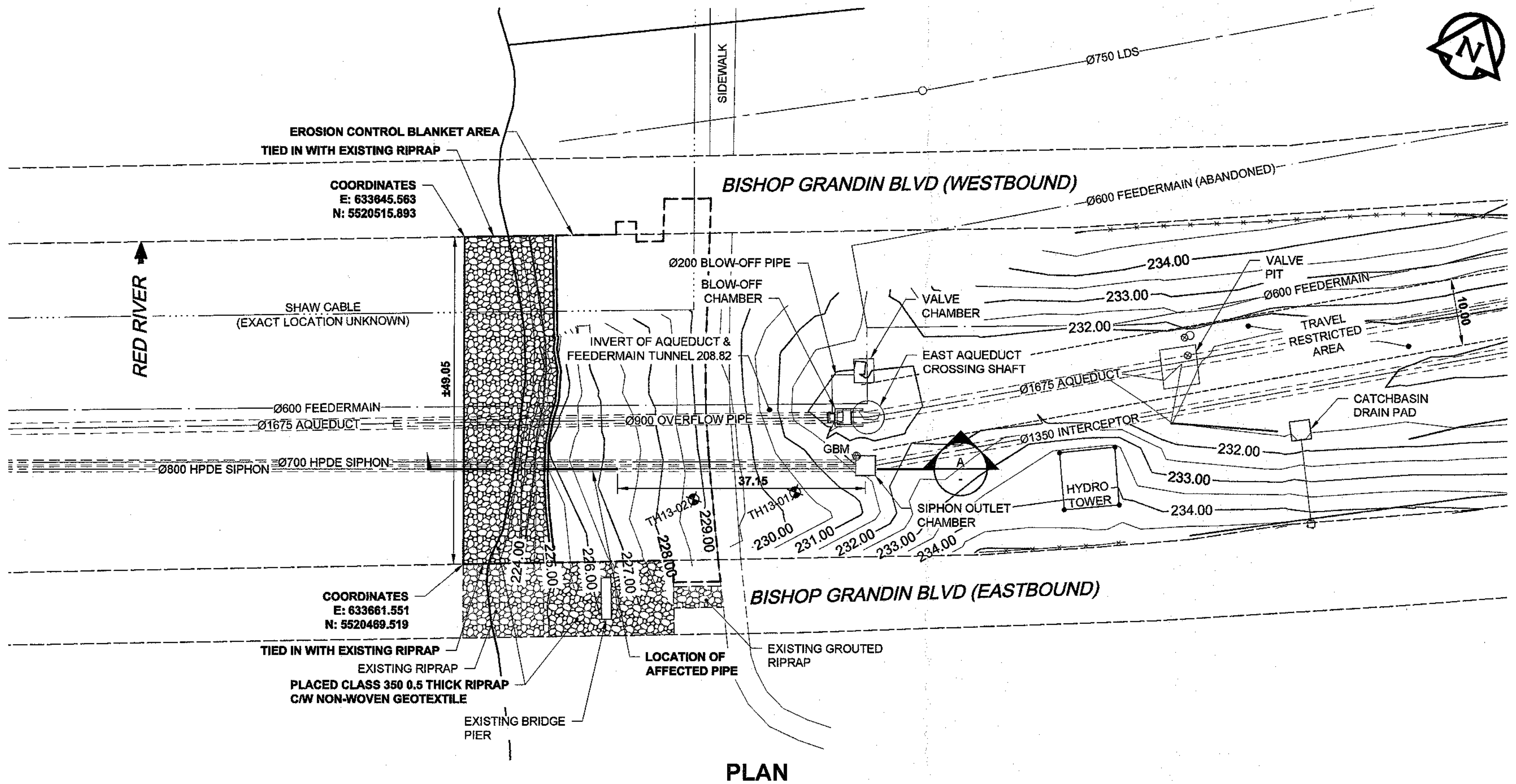
TRACED BY S.K. 13 JAN. 59.

H-O 134

NOTE:
ALL CHAINAGES REVISED TO FINAL (AS CONSTRUCTED) CHAINAGES. 21 SEPT. 60 - S.K.

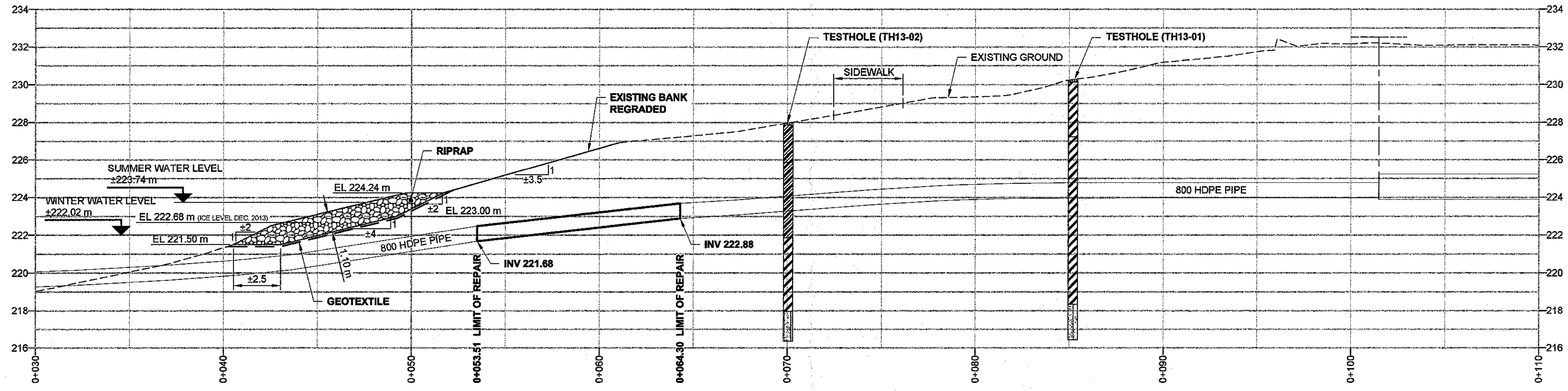
D SIZE 22" x 34" (559mm x 864mm)

PLOT: 6/13/2016 4:29:39 PM



PLAN

1:500



A SECTION

1:150

LEGEND

- - - - - - EXISTING GROUND PROFILE
— — — — GEOTEXTILE
———— NEW GROUND PROFILE
▨ EXISTING GROUND TO BE EXCAVATED
▨ RIP RAP CLASS 350
▩ TOPSOIL / ORGANICS
▨ CLAY AND SILT
▨ CLAY
▩ TILL

METRIC
WHOLE NUMBERS INDICATE MILLIMETRES
DECIMALIZED NUMBERS INDICATE METRES

APEGM
Certificate of Authorization
AECOM Canada Ltd.
No. 4671 Date: June 14/16

RECORD DRAWING

BID OPPORTUNITY NO. 1040-2013

EXISTING	LEGEND - PLAN	NEW	EXISTING	LEGEND - PLAN	NEW	EXISTING	LEGEND - PROFILE	NEW
150 WM	WATERMAIN	150 WM	♂	CURB STOP	●	150 WM	WATERMAIN	150 WM
-○-	HYDRANT	◆	—▷—	REDUCER	—▶—	┼	HYDRANT	┼
⊗	VALVE	●	—✕—	COUPLING	—✕—	X	VALVE	X
300 LDS	LAND DRAINAGE SEWER	300 LDS	⊃	ANODE	⊃	300 LDS	LAND DRAINAGE SEWER	300 LDS
250 WWS	WASTE WATER SEWER	250 WWS	— — —	HYDRO		250 WWS	WASTE WATER SEWER	250 WWS
○	MANHOLE	●	— · · —	MTS		— · —	PAVEMENT CROWN	— · —
□	CATCH BASIN	■	— ··· —	GAS		—◇—	N/W PROPERTY LINE	
△	CURB INLET	▲	⊕	TESTHOLE		—○—	S/E PROPERTY LINE	
⊏⊐	CULVERT	⊏⊐	•—•	LAMP STANDARD		—□—	N/W GUTTER	
—⊐	PIPE ABANDONMENTS	—⊐	⊙	TREE		— ○ —	S/E GUTTER	
⌖	SURVEY BAR							

LOCATION APPROVED
UNDERGROUND STRUCTURES

SUPR. U/G STRUCTURES COMMITTEE DATE

NOTE:
LOCATION OF UNDERGROUND STRUCTURES AS SHOWN ARE BASED ON THE BEST INFORMATION AVAILABLE. BUT NO GUARANTEE IS GIVEN THAT ALL EXISTING UTILITIES ARE SHOWN OR THAT THE GIVEN LOCATIONS ARE EXACT. CONFIRMATION OF EXISTENCE AND EXACT LOCATION OF ALL SERVICES MUST BE OBTAINED FROM THE INDIVIDUAL UTILITIES BEFORE PROCEEDING WITH CONSTRUCTION.

NO.	REVISIONS	YY/MM/DD	BY
1	ISSUED FOR RECORD	16/05/12	KMB/KLC
0	ISSUED FOR CONSTRUCTION	13/12/13	CM

AECOM

DESIGNED BY	AJH	CHECKED BY	
DRAWN BY	CM	APPROVED BY	
HOR. SCALE	AS NOTED	RELEASED FOR CONSTRUCTION	
VERT. SCALE	AS NOTED		
DATE		DATE	

PROFESSIONAL'S SEAL
Z. T. AL-HAYAZAI
Member 37236
June 14/16

CONSULTANT DRAWING NO.
60274906-01-B-1002-R0X.dwg

THE CITY OF WINNIPEG
WATER AND WASTE DEPARTMENT

FORT GARRY INTERCEPTOR
SIPHON REPAIR
GEOTECHNICAL
PLAN/PROFILE

SHEET 2 OF 2

CITY DRAWING NUMBER
LD-7277
REV 1

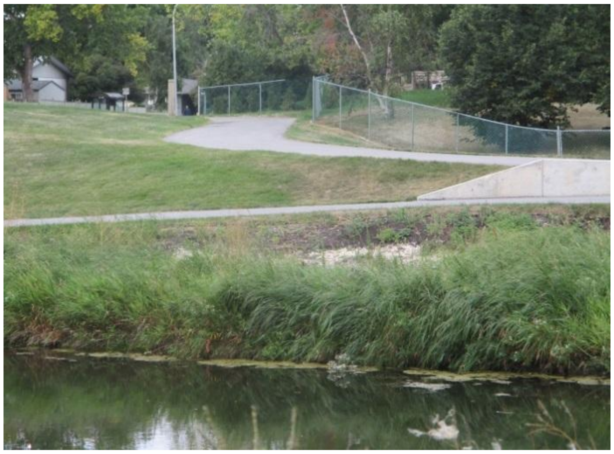
AECOM REVIEW | DRFT CHK

AECOM FILE NAME: 60274906-01-B-1002-R0X.dwg Saved By: cooperd

Appendix C

Field Inspection Photographs

			
Site 5- Eastern Creek Bank	Stone Rip-rap along bridge embankments (looking south)	Site 5- Eastern Creek Bank	Mature trees set-back from creek bank slope (looking east)
			
Site 5- Eastern Creek Bank	Stone Rip-rap along bridge embankments (looking southwest)	Site 5- Eastern Creek Bank	Slope conditions below Ness Avenue Bridge (looking south)
			
Site 5- Eastern Creek Bank	General view of creek and creek banks (looking north)	Site 5- Eastern Creek Bank	General view of creek and creek banks (looking north)

			
Site 5- Western Creek Bank	General view of western creek banks slopes (looking west).	Site 5- Eastern Creek Bank	Minor soil desiccation cracking within slope soils near toe (looking west)
			
Site 5- Eastern Creek Bank	Ness Avenue Bridge Crossing (looking south)	Site 5- Eastern Creek Bank	Eastern creek bank slope- slope crest (looking north)
			
Site 5- Western Creek Bank	Existing sidewalk pavement above western bank (looking west)	Site 5- Western Creek Bank	Existing sidewalk pavement above western bank (looking west)

			
Site 5- Western Creek Bank	Erosional scarps near water's edge along lower slope (looking west)	Site 5- Western Creek Bank	Sidewalk pavement (looking south)
			
Site 5- Western Creek Bank	Adjacent property (looking west)	Site 5- Western Creek Bank	Sidewalk pavement (looking North)
			
Site 5- Western Creek Bank	Sidewalk pavement Bridge Crossing (looking South)	Site 5- Western Creek Bank	Western slope surface and bridge crossing (looking south)

Site 5- Western Creek Bank

Soil desiccation cracking in exposed slope soils (looking northeast)

Site 6- Eastern River Bank

Ground surface between approach embankments (looking west)

Site 6- Eastern River Bank

Foundation fill for hydro-tower (looking southwest)

Site 6- Eastern River Bank

Exposed foundation footing for hydro-tower (looking south)

Site 6- Eastern River Bank

General view of ground surface between bridges (looking west)

Site 6- Eastern River Bank

Bridge approach fill and bridge abutment (looking north)

			
Site 6- Eastern River Bank	Sidewalk pavement with cracking (looking south)	Site 6- Eastern River Bank	Sidewalk pavement with cracking (looking north)
			
Site 6- Eastern River Bank	General view down slope between bridges (looking west)	Site 6- Eastern River Bank	South bridge (eastbound) abutment and standpipe (looking east)
			
Site 6- Eastern River Bank	Grouted rip-rap and sidewalk pavement (looking west)	Site 6- Eastern River Bank	Grouted rip-rap above south bridge pier (looking west)

Site 6- Eastern River Bank

Red River at base of south bridge pier (looking west)

Site 6- Eastern River Bank

Adjacent riverbank slopes south of south bridge (looking south)

Site 6- Eastern River Bank

General of riverbank slope between bridges (looking north)

Site 6- Eastern River Bank

North of north bridge with partially eroded slope soil (looking north)

Site 6- Eastern River Bank

Soil erosion in upper riverbank slope (looking southeast)

Site 6- Eastern River Bank

General view of drainage culvert below north bridge pier (looking east)

Site 6- Western River Bank	General view in between bridges (looking east)

Site 6- Western River Bank	General view in between bridges (looking east)

Site 6- Western River Bank	North bridge abutment and embankment (looking north)

Site 6- Western River Bank	South bridge abutment and embankment (looking south)

Site 6- Western River Bank	South bridge pier and drainage culvert (looking east)

Site 6- Western River Bank	Sidewalk pavement and north bridge abutment (looking north)

			
Site 6- Western River Bank	Up slope of south bridge pier with eroded soils (looking east)	Site 6- Western River Bank	Adjacent riverbank slopes south of south bridge (looking south)
			
Site 6- Western River Bank	Lower riverbank slope between bridges (looking north)	Site 6- Western River Bank	Lower riverbank slope between bridges (looking northwest)
			
Site 6- Western River Bank	Lower riverbank slope between bridges (looking north)	Site 6- Western River Bank	Random rip-rap near lower slope of riverbank slope (looking north)

Appendix D

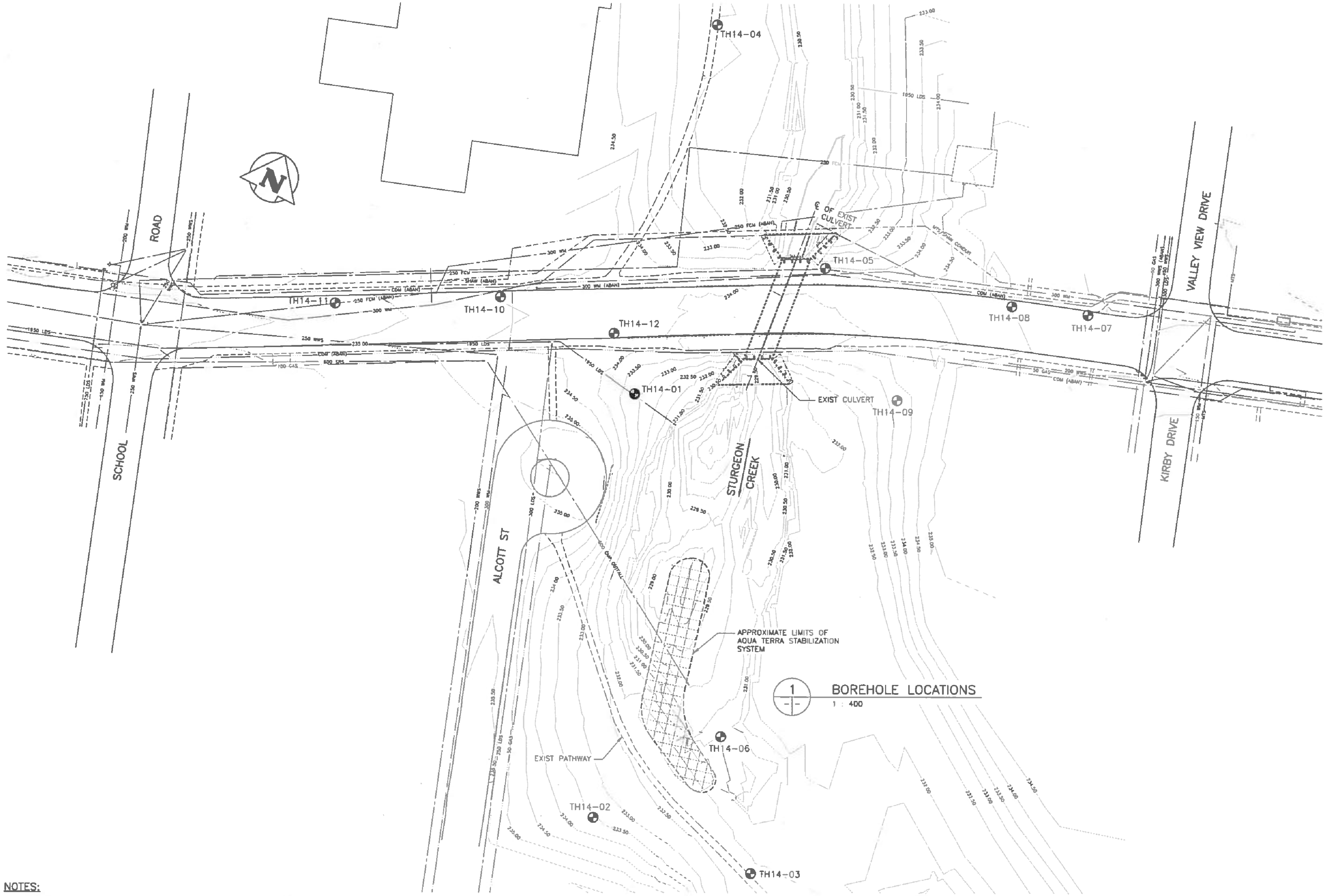
Site Reconnaissance Summary

APPENDIX D- SUMMARY OF SITE INSPECTIONS

				PIPE ASSET					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)	COMMENTS
CROSSING ID NUMBER	LOCATION	PIPE FUNCTION	RIVER	PIPE DIAMETER (mm)	PIPE MATERIAL	INSPECTION LENGTH (m) OF RIVERBANK	SIDE OF RIVER	NEIGHBOURING STREET	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	NOT EXIST		
5	Heritage Park	FORCE MAIN	STURGEON CREEK	250	PVC	100	EAST	VALLEY VIEW DRIVE			X	X		X			X		X	X			X		X	X		X		1	
							WEST	NESS AVENUE			X	X		X			X		X	X			X		X	X		X		1	
6	Fort Garry/St.Vital	FEEDER MAIN	RED	600	CAST-IRON	85	EAST	BISHOP GRANDIN HIGHWAY			X		X		X		X		X		X	X		X		X		X		1	
		AQUEDUCT		1650	PPC		WEST	BISHOP GRANDIN HIGHWAY			X	X		X		X			X	X			X		X	X		X		2	Oversteepened lower riverbank slopes (minor tension cracking)
		SIPHON		700	HDPE	85	EAST	BISHOP GRANDIN HIGHWAY			X		X		X		X		X		X	X		X		X		X		1	
				800	HDPE		WEST	BISHOP GRANDIN HIGHWAY			X	X		X		X			X	X			X		X	X		X		2	Oversteepened lower riverbank slopes (minor tension cracking)

Appendix E

Site 5 – Heritage Park: Existing Geotechnical Information



1 BOREHOLE LOCATIONS
1 : 400

NOTES:

1. TH REFERS TO BOREHOLE TEST LOGS. TEST HOLES WERE TAKEN BY TREK GEOTECHNICAL BETWEEN AUGUST 20 AND 22, 2014.
2. THE TEST HOLE LOGS PROVIDED FOR THIS PROJECT HAVE BEEN COMPILED FOR DESIGN PURPOSES ONLY. WHILE IT IS BELIEVED TO CORRECTLY REPRODUCE OR SUMMARIZE OBSERVATIONS MADE DURING TESTING, THE INFORMATION IS VALID ONLY FOR THE PRECISE LOCATIONS SHOWN AND IS NOT TO BE CONSTRUED AS GUARANTEEING THE PRESENCE, ABSENCE, OR EXTENT OF BOULDERS, HARD OR SOFT FORMATIONS, WATER TABLES, ARTESIAN CONDITIONS AND OTHER VARIABLES. IT IS THE RESPONSIBILITY OF OTHERS USING THIS INFORMATION TO ENSURE THAT IT IS ADEQUATE FOR THEIR PURPOSES OR TO SUPPLEMENT IT WITH ADDITIONAL INFORMATION.
3. STANDPIPE PIEZOMETERS WERE INSTALLED IN THE BEDROCK AND SILT LAYER TO ASSESS GROUNDWATER LEVELS IN TH14-01, TH14-04 AND TH14-06.
4. CONTOURS SHOWN ARE EXISTING.
5. LOCATION OF UTILITIES ARE APPROXIMATE ONLY. LOCATION TO BE CONFIRMED IN THE FIELD.

APEGM
Certificate of Authorization
TETRA TECH WEI Inc.
No. 5313 Date: April 30, 2015

B.M.
ELEV.

NO	REVISIONS	DATE	BY
0	ISSUED FOR TENDER	15.09.24	S.A.

TETRA TECH

DESIGNED BY D.M.	CHECKED BY S.A.
DRAWN BY G.I.	APPROVED BY E.F.S.
HOR. SCALE: AS NOTED VERTICAL:	ACCEPTED BY DATE ORIGINAL DRAWING SIGNED BY: D.N. BURNEY, P.ENG. 15.09.24 D.N. BURNEY, P.ENG. BRIDGE PROJECTS ENGINEER
DATE 15.09.24	

S.Y.I. AWAD
ORIGINAL SIGNED 15.09.24
31981

CONSULTANT DRAWING NO.
1400070800-DWG-S0006

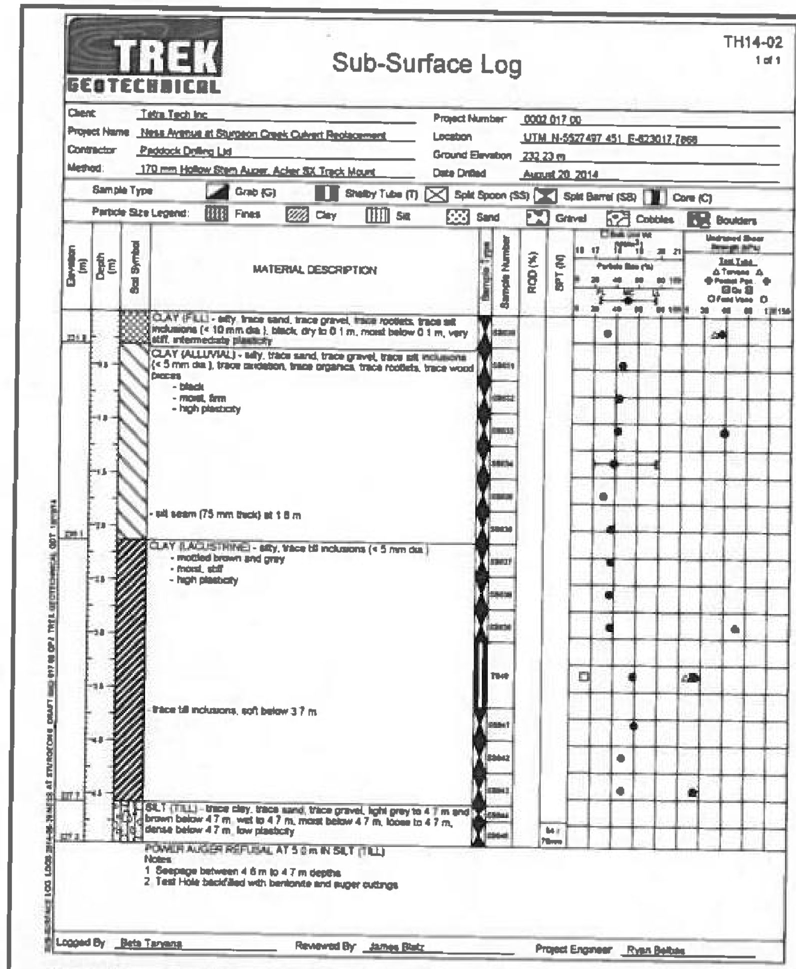
THE CITY OF WINNIPEG
PUBLIC WORKS DEPARTMENT
ENGINEERING DIVISION

NESS AVENUE AT STURGEON CREEK
BRIDGE CONSTRUCTION

BOREHOLE LOCATIONS

CITY DRAWING NUMBER
B243-15-006
SHEET 6 OF 64

6



Sub-Surface Log

TH14-02
1 of 1

Client: Tetra Tech Inc
Project Name: Ness Avenue at Sturgeon Creek Culvert Replacement
Contractor: Paddock Drilling Ltd
Method: 170 mm Hollow Stem Auger, Acker SX Track Mount
Project Number: 0002 017 00
Location: UTM N-5527497 451, E-623017 7868
Ground Elevation: 232 23 m
Date Drilled: August 20, 2014

Sample Type: Grab (G) | Shelby Tube (T) | Split Spoon (SS) | Split Barrel (SB) | Core (C)
Particle Size Legend: Fines | Clay | Silt | Sand | Gravel | Cobbles | Boulders

MATERIAL DESCRIPTION

CLAY (FILL) - silty, trace sand, trace gravel, trace rootlets, trace silt inclusions (< 10 mm dia), black, dry to 0 1 m, moist below 0 1 m, very stiff, intermediate plasticity

CLAY (ALLUVIAL) - silty, trace sand, trace gravel, trace silt inclusions (< 5 mm dia), trace oxidation, trace organics, trace rootlets, trace wood pieces
- black
- moist, firm
- high plasticity

- silt seam (75 mm thick) at 1 8 m

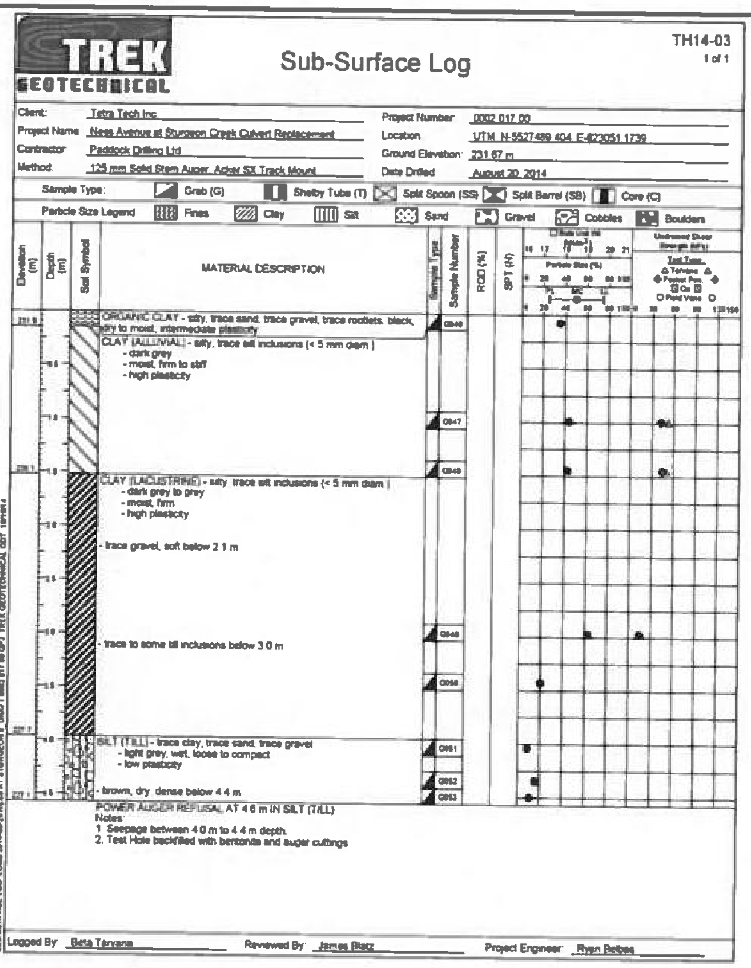
CLAY (LACUSTRINE) - silty, trace till inclusions (< 5 mm dia)
- mottled brown and grey
- moist, stiff
- high plasticity

- trace till inclusions, soft below 3 7 m

SILT (TILL) - trace clay, trace sand, trace gravel, light grey to 4 7 m and brown below 4 7 m, wet to 4 7 m, moist below 4 7 m, loose to 4 7 m, dense below 4 7 m, low plasticity

POWER AUGER REFUSAL AT 5 0 m IN SILT (TILL)
Notes:
1 Seepage between 4 6 m to 4 7 m depths
2 Test Hole backfilled with bentonite and auger cuttings

Logged By: Beta Taryana
Reviewed By: James Blatz
Project Engineer: Ryan Belbas



Sub-Surface Log

TH14-03
1 of 1

Client: Tetra Tech Inc
Project Name: Ness Avenue at Sturgeon Creek Culvert Replacement
Contractor: Paddock Drilling Ltd
Method: 125 mm Solid Stem Auger, Acker SX Track Mount
Project Number: 0002 017 00
Location: UTM N-5527489 404, E-623051 1739
Ground Elevation: 231 67 m
Date Drilled: August 20, 2014

Sample Type: Grab (G) | Shelby Tube (T) | Split Spoon (SS) | Split Barrel (SB) | Core (C)
Particle Size Legend: Fines | Clay | Silt | Sand | Gravel | Cobbles | Boulders

MATERIAL DESCRIPTION

ORGANIC CLAY - silty, trace sand, trace gravel, trace rootlets, black, dry to moist, intermediate plasticity

CLAY (ALLUVIAL) - silty, trace silt inclusions (< 5 mm diam)
- dark grey
- moist, firm to stiff
- high plasticity

CLAY (LACUSTRINE) - silty, trace silt inclusions (< 5 mm diam)
- dark grey to grey
- moist, firm
- high plasticity

- trace gravel, soft below 2 1 m

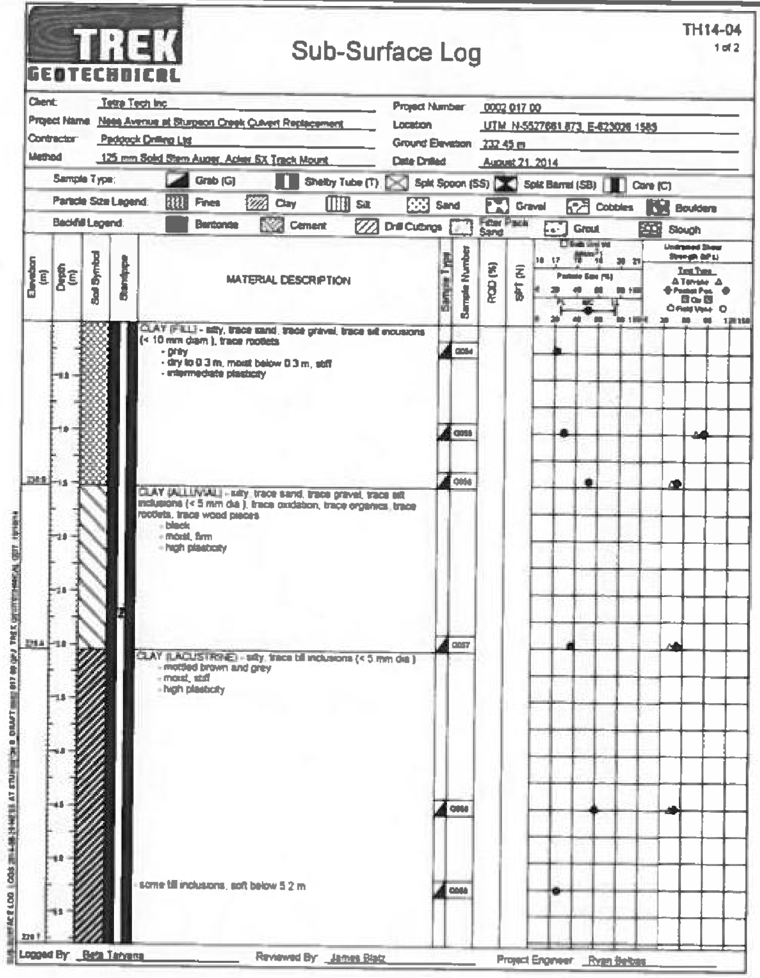
- trace to some till inclusions below 3 0 m

SILT (TILL) - trace clay, trace sand, trace gravel
- light grey, wet, loose to compact
- low plasticity

- brown, dry, dense below 4 4 m

POWER AUGER REFUSAL AT 4 6 m IN SILT (TILL)
Notes:
1 Seepage between 4 0 m to 4 4 m depth.
2. Test Hole backfilled with bentonite and auger cuttings

Logged By: Beta Taryana
Reviewed By: James Blatz
Project Engineer: Ryan Belbas



Sub-Surface Log

TH14-04
1 of 2

Client: Tetra Tech Inc
Project Name: Ness Avenue at Sturgeon Creek Culvert Replacement
Contractor: Paddock Drilling Ltd
Method: 125 mm Solid Stem Auger, Acker SX Track Mount
Project Number: 0002 017 00
Location: UTM N-5527681 873, E-623026 1585
Ground Elevation: 232 45 m
Date Drilled: August 21, 2014

Sample Type: Grab (G) | Shelby Tube (T) | Split Spoon (SS) | Split Barrel (SB) | Core (C)
Particle Size Legend: Fines | Clay | Silt | Sand | Gravel | Cobbles | Boulders
Backfill Legend: Bentonite | Cement | Drill Cuttings | Filter Pack Sand | Grout | Slough

MATERIAL DESCRIPTION

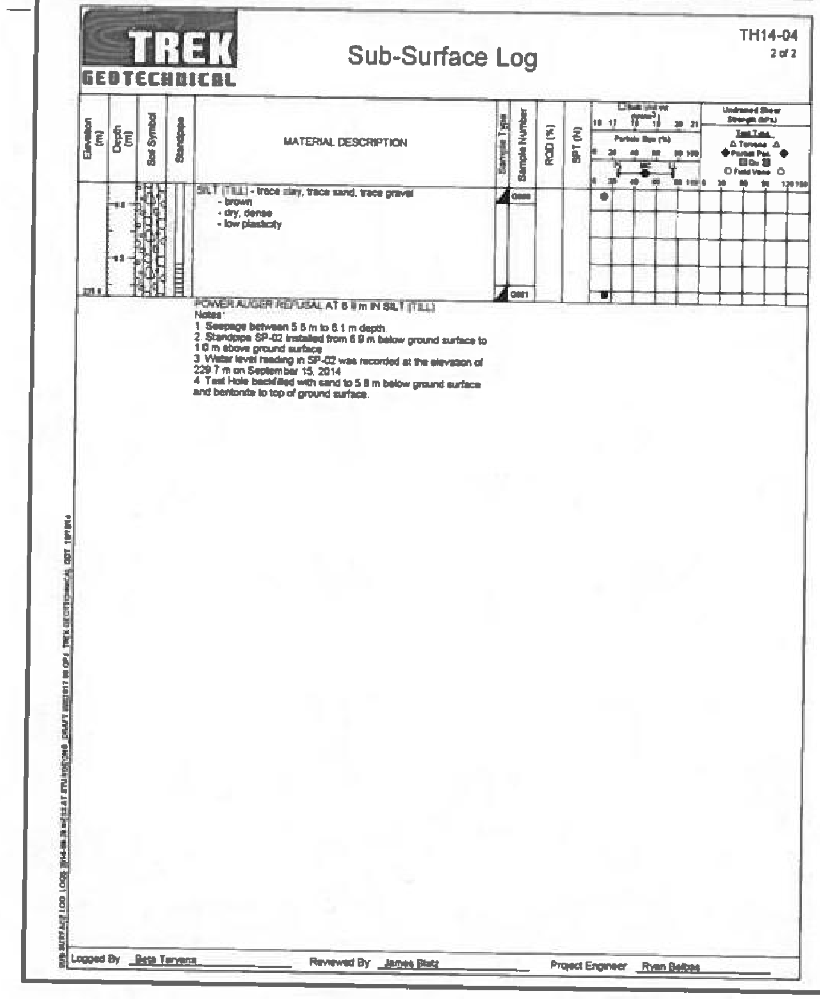
CLAY (FILL) - silty, trace sand, trace gravel, trace silt inclusions (< 10 mm diam), trace rootlets
- grey
- dry to 0 3 m, moist below 0 3 m, stiff
- intermediate plasticity

CLAY (ALLUVIAL) - silty, trace sand, trace gravel, trace silt inclusions (< 5 mm dia), trace oxidation, trace organics, trace rootlets, trace wood pieces
- black
- moist, firm
- high plasticity

CLAY (LACUSTRINE) - silty, trace till inclusions (< 5 mm dia)
- mottled brown and grey
- moist, stiff
- high plasticity

- some till inclusions, soft below 5 2 m

Logged By: Beta Taryana
Reviewed By: James Blatz
Project Engineer: Ryan Belbas



Sub-Surface Log

TH14-04
2 of 2

MATERIAL DESCRIPTION

SILT (TILL) - trace clay, trace sand, trace gravel
- brown
- dry, dense
- low plasticity

POWER AUGER REFUSAL AT 6 6 m IN SILT (TILL)
Notes:
1 Seepage between 5 6 m to 6 1 m depth.
2. Standpipe SP-02 installed from 6 9 m below ground surface to 1 0 m above ground surface
3 Water level reading in SP-02 was recorded at the elevation of 229.7 m on September 15, 2014
4 Test Hole backfilled with sand to 5 8 m below ground surface and bentonite to top of ground surface.

Logged By: Beta Taryana
Reviewed By: James Blatz
Project Engineer: Ryan Belbas

APEGM
Certificate of Authorization
TETRA TECH WEI Inc.
No. 5813 Date: April 30, 2016

B.M.
ELEV

NO	REVISIONS	DATE	BY
0	ISSUED FOR TENDER	15.09.24	S.A.

TETRA TECH

DESIGNED BY: D.M.
CHECKED BY: S.A.
DRAWN BY: G.I.
APPROVED BY: E.F.S.
HOR. SCALE: / VERTICAL: AS NOTED
DATE: 15.09.24
ACCEPTED BY / DATE: ORIGINAL DRAWING SIGNED BY: D.H. BURNEY, P.ENG. 15.09.24
D.H. BURNEY, P.ENG.
BRIDGE PROJECTS ENGINEER

S.Y.I. AWAD
ORIGINAL SIGNED
15.09.24
31981

CONSULTANT DRAWING NO.
1400070800-DWG-50008

THE CITY OF WINNIPEG
PUBLIC WORKS DEPARTMENT
ENGINEERING DIVISION

NESS AVENUE AT STURGEON CREEK
BRIDGE CONSTRUCTION

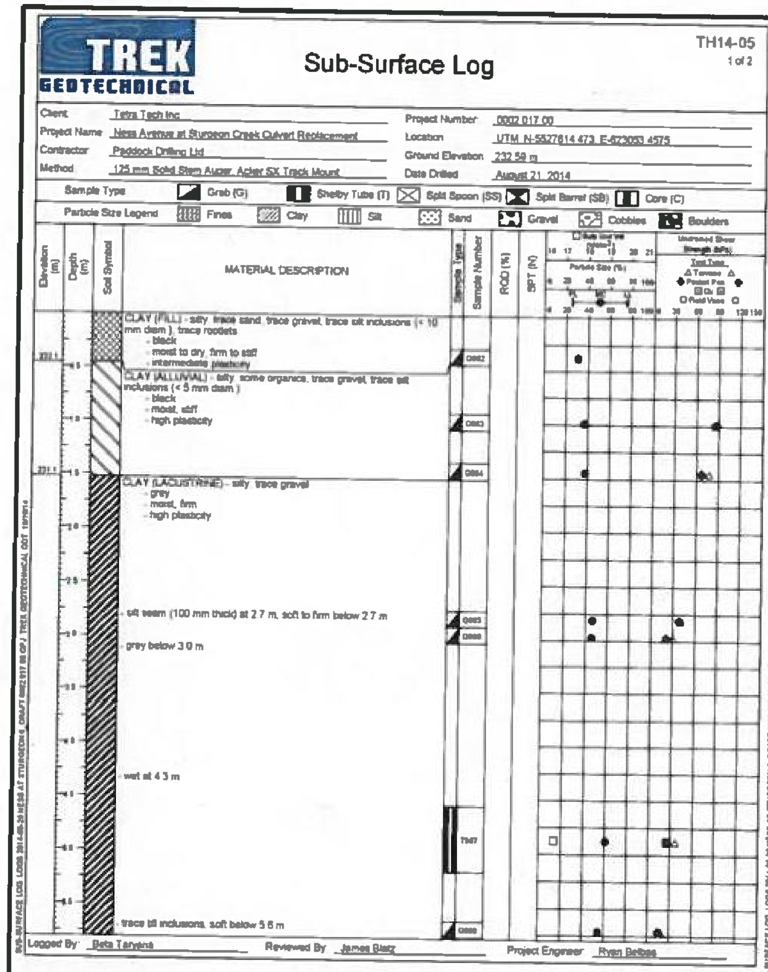
BOREHOLE LOGS
SHEET 2 OF 4

CITY DRAWING NUMBER
B243-15-008

SHEET 8 OF 64

8

X:\A-G\CITY OF WINNIPEG - PUBLIC WORKS - 0007\14000708.00 - NESS AVE AT STURGEON CREEK CULVERT\CAD\STRUCT\WORKING DRAWINGS\1400070800-DWG-S0009 BOREHOLE LOGS 3.DWG 15.09.23 2:08 PM TARYANA, BETA



Sub-Surface Log

TH14-05
1 of 2

Client: Tetra Tech Inc
Project Name: Ness Avenue at Sturgeon Creek Culvert Replacement
Contractor: Paddock Drilling Ltd
Method: 125 mm Solid Stem Auger, Acker SX Track Mount
Project Number: 0002 017 00
Location: UTM N-5527614.473, E-623053.4575
Ground Elevation: 232.59 m
Date Drilled: August 21, 2014

Sample Type: Grab (G), Shelby Tube (T), Split Spoon (SS), Split Barrel (SB), Core (C)
Particle Size Legend: Fines, Clay, Silt, Sand, Gravel, Cobbles, Boulders

MATERIAL DESCRIPTION

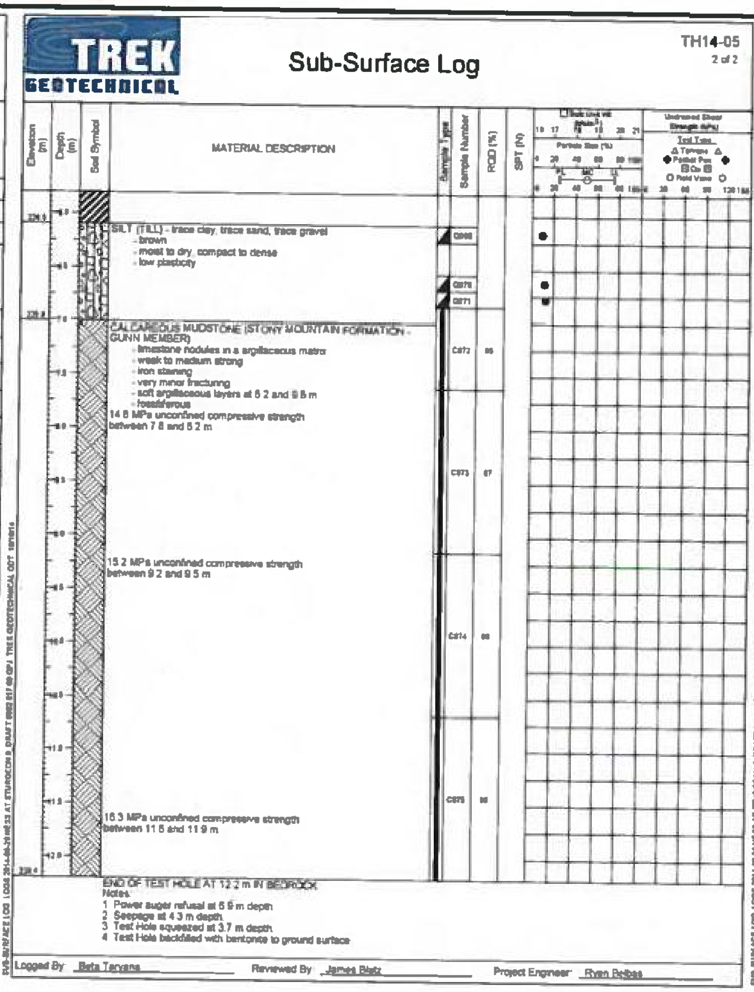
CLAY (FILL) - silty, trace sand, trace gravel, trace silt inclusions (< 10 mm diam.), trace rootlets
- black
- moist to dry, firm to stiff
- intermediate plasticity

CLAY (ALLUVIAL) - silty, some organics, trace gravel, trace silt inclusions (< 5 mm diam.)
- black
- moist, stiff
- high plasticity

CLAY (LACUSTRINE) - silty, trace gravel
- grey
- moist, firm
- high plasticity

- silt seam (100 mm thick) at 2.7 m, soft to firm below 2.7 m
- grey below 3.0 m
- wet at 4.3 m
- trace till inclusions, soft below 5.6 m

Logged By: Beta Taryana
Reviewed By: James Blatz
Project Engineer: Ryan Belbas



Sub-Surface Log

TH14-05
2 of 2

MATERIAL DESCRIPTION

SILT (TILL) - trace clay, trace sand, trace gravel
- brown
- moist to dry, compact to dense
- low plasticity

CALCAREOUS MUDSTONE (STONY MOUNTAIN FORMATION - GUNN MEMBER)
- limestone nodules in a argillaceous matrix
- weak to medium strong
- iron staining
- very minor fracturing
- soft argillaceous layers at 8.2 and 9.8 m
- fossiliferous

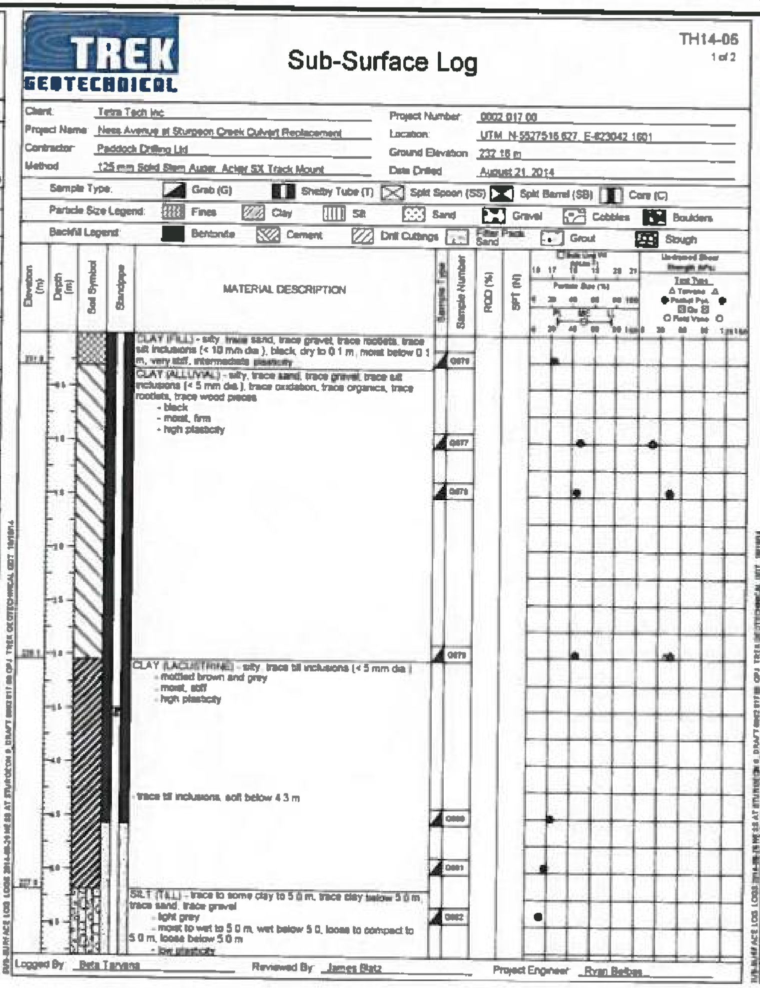
14.8 MPa unconfined compressive strength between 7.8 and 8.2 m

15.2 MPa unconfined compressive strength between 9.2 and 9.5 m

16.3 MPa unconfined compressive strength between 11.6 and 11.9 m

END OF TEST HOLE AT 12.2 m IN BEDROCK
Notes:
1. Power auger refusal at 6.9 m depth
2. Seepage at 4.3 m depth
3. Test Hole squeezed at 3.7 m depth
4. Test Hole backfilled with bentonite to ground surface

Logged By: Beta Taryana
Reviewed By: James Blatz
Project Engineer: Ryan Belbas



Sub-Surface Log

TH14-06
1 of 2

Client: Tetra Tech Inc
Project Name: Ness Avenue at Sturgeon Creek Culvert Replacement
Contractor: Paddock Drilling Ltd
Method: 125 mm Solid Stem Auger, Acker SX Track Mount
Project Number: 0002 017 00
Location: UTM N-5527516.627, E-623042.1601
Ground Elevation: 232.16 m
Date Drilled: August 21, 2014

Sample Type: Grab (G), Shelby Tube (T), Split Spoon (SS), Split Barrel (SB), Core (C)
Particle Size Legend: Fines, Clay, Silt, Sand, Gravel, Cobbles, Boulders
Backfill Legend: Bentonite, Cement, Drill Cuttings, Filter Pack Sand, Grout, Slough

MATERIAL DESCRIPTION

CLAY (FILL) - silty, trace sand, trace gravel, trace rootlets, trace silt inclusions (< 10 mm dia.), black, dry to 0.1 m, moist below 0.1 m, very stiff, intermediate plasticity

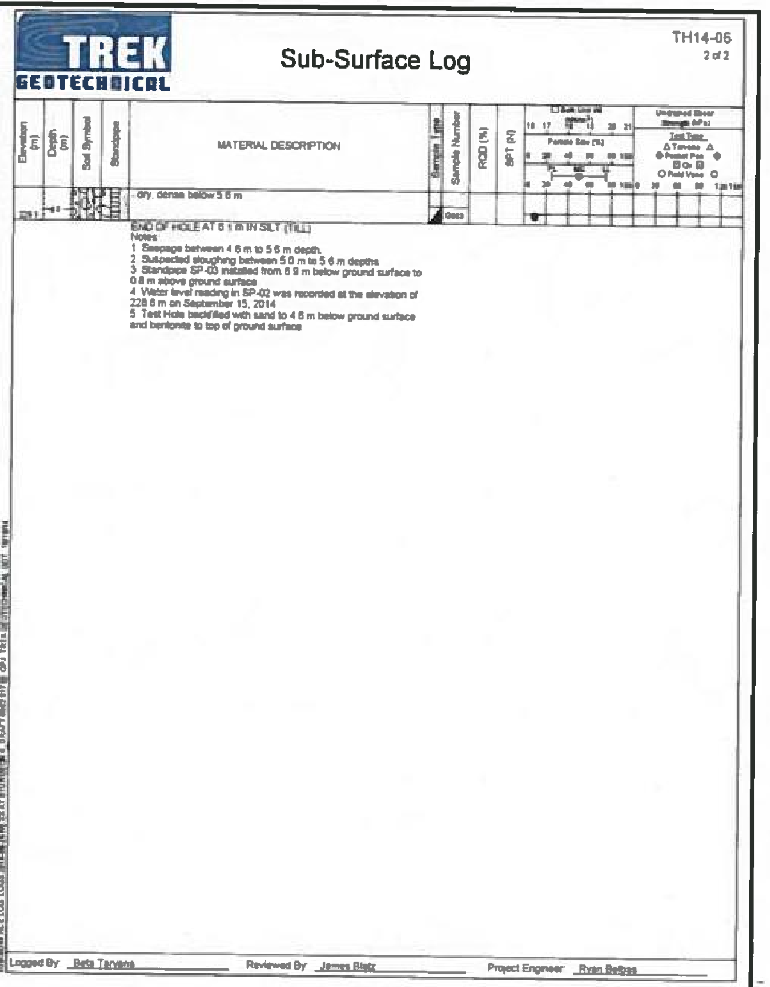
CLAY (ALLUVIAL) - silty, trace sand, trace gravel, trace silt inclusions (< 5 mm dia.), trace oxidation, trace organics, trace rootlets, trace wood pieces
- black
- moist, firm
- high plasticity

CLAY (LACUSTRINE) - silty, trace till inclusions (< 5 mm dia.)
- mottled brown and grey
- moist, stiff
- high plasticity

- trace till inclusions, soft below 4.3 m

SILT (TILL) - trace to some clay to 5.0 m, trace clay below 5.0 m, trace sand, trace gravel
- light grey
- moist to wet to 5.0 m, wet below 5.0, loose to compact to 5.0 m, loose below 5.0 m
- low plasticity

Logged By: Beta Taryana
Reviewed By: James Blatz
Project Engineer: Ryan Belbas



Sub-Surface Log

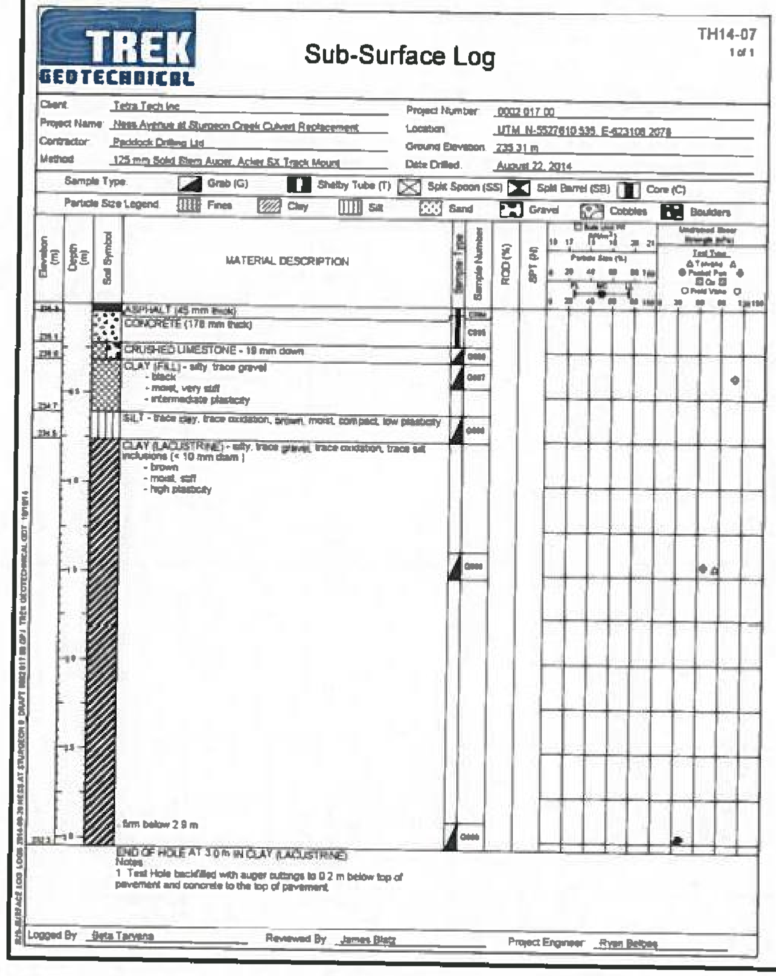
TH14-06
2 of 2

MATERIAL DESCRIPTION

- dry, dense below 5.6 m

END OF HOLE AT 6.1 m IN SILT (TILL)
Notes:
1. Seepage between 4.6 m to 5.6 m depth.
2. Suspected sloughing between 5.0 m to 5.6 m depths.
3. Standpipe SP-03 installed from 6.9 m below ground surface to 0.8 m above ground surface
4. Water level reading in SP-02 was recorded at the elevation of 228.6 m on September 15, 2014
5. Test Hole backfilled with sand to 4.6 m below ground surface and bentonite to top of ground surface

Logged By: Beta Taryana
Reviewed By: James Blatz
Project Engineer: Ryan Belbas



Sub-Surface Log

TH14-07
1 of 1

Client: Tetra Tech Inc
Project Name: Ness Avenue at Sturgeon Creek Culvert Replacement
Contractor: Paddock Drilling Ltd
Method: 125 mm Solid Stem Auger, Acker SX Track Mount
Project Number: 0002 017 00
Location: UTM N-5527610.535, E-623108.2078
Ground Elevation: 235.31 m
Date Drilled: August 22, 2014

Sample Type: Grab (G), Shelby Tube (T), Split Spoon (SS), Split Barrel (SB), Core (C)
Particle Size Legend: Fines, Clay, Silt, Sand, Gravel, Cobbles, Boulders

MATERIAL DESCRIPTION

ASPHALT (45 mm thick)

CONCRETE (178 mm thick)

CRUSHED LIMESTONE - 19 mm down

CLAY (FILL) - silty, trace gravel
- black
- moist, very stiff
- intermediate plasticity

SILT - trace clay, trace oxidation, brown, moist, compact, low plasticity

CLAY (LACUSTRINE) - silty, trace gravel, trace oxidation, trace silt inclusions (< 10 mm diam.)
- brown
- moist, stiff
- high plasticity

- firm below 2.9 m

END OF HOLE AT 3.0 m IN CLAY (LACUSTRINE)
Notes:
1. Test Hole backfilled with auger cuttings to 0.2 m below top of pavement and concrete to the top of pavement

Logged By: Beta Taryana
Reviewed By: James Blatz
Project Engineer: Ryan Belbas

APEGM
Certificate of Authorization
TETRA TECH WEI Inc.
No. 6313 Date: April 30, 2015

B.M.
ELEV.

NO	REVISIONS	DATE	BY
0	ISSUED FOR TENDER	15.09.24	S.A.

TETRA TECH

DESIGNED BY: D.M.
CHECKED BY: S.A.
DRAWN BY: G.I.
APPROVED BY: E.F.S.
HOR. SCALE / VERTICAL: AS NOTED
DATE: 15.09.24
ACCEPTED BY / DATE: ORIGINAL DRAWING SIGNED BY: D.N. BURVEY, P.ENG. 15.09.24
D.N. BURVEY, P.ENG.
BRIDGE PROJECTS ENGINEER

S.Y.I. AWAD
ORIGINAL SIGNED
15.09.24
31981

CONSULTANT DRAWING NO.: 1400070800-DWG-S0009

THE CITY OF WINNIPEG
PUBLIC WORKS DEPARTMENT
ENGINEERING DIVISION

NESS AVENUE AT STURGEON CREEK
BRIDGE CONSTRUCTION

BOREHOLE LOGS
SHEET 3 OF 4

CITY DRAWING NUMBER: B243-15-009
SHEET 9 OF 64

9

Appendix F

Site 6 – Fort Garry/St. Vital: Existing Geotechnical Information

TEST HOLE LOG

ELEV. COLLAR Tech: S. Gilchrist
ELEV. GROUND 751
CO-ORD. LOCATION 10(1)+ 25·– 100' Right

SAMPLE DATA
WEIGHT HAMMER
HEIGHT DROP

DEPTH / ELEV.	O.D / I.D.	BLOWS / FT	NO.
	3"Sy		1
10 / 741	3"Sy		2
	3"Sy		3
20 / 731	3"Sy		4
	3"Sy		5
30 / 721	3"Sy		6
	3"Sy		7
40 / 711	Bag		8

SYMBOL

DESCRIPTION OF MATERIAL

1 TOPSOIL

CLAY – silty
– medium plastic
– grey
– nuggetty to massive
– silt pockets to 1/8" diam
– stiff to very stiff

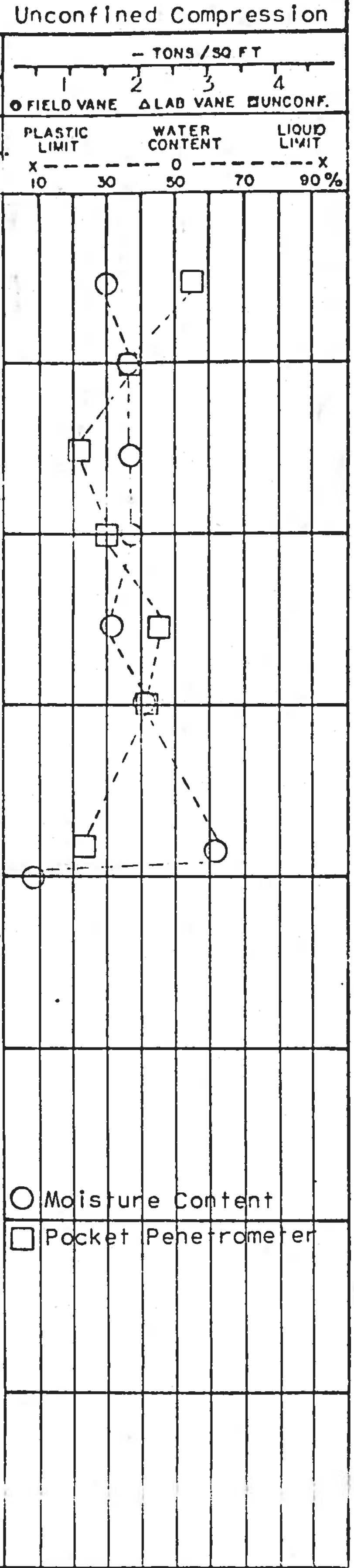
18

CLAY – highly plastic
– grey
– massive
– sand pockets <1/16" in diam to 20'
– silt pockets <1/16" in diam
– stiff to very stiff

38

40 TILL-LIKE – silty
– tan
– hard

NOTES:
1. Hole terminated at 40' on a boulder in till-like material.
2. About 1½' of water in hole when finished drilling.



Klohn Leonoff Consultants Ltd.
CIVIL & GEOTECHNICAL ENGINEERS

JOB No. W00983
PROJECT Transportation Corridor
LOCATION Ft Garry/St Vital, Manitoba
HOLE No. 401
DATE Feb 13/76 PLATE A-W-983-136

TEST HOLE LOG

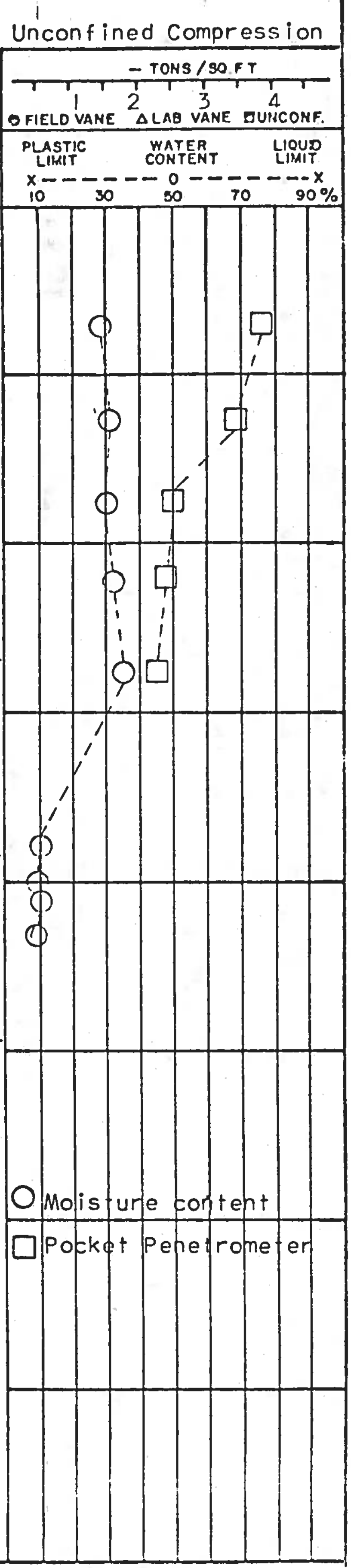
SAMPLE DATA
WEIGHT HAMMER
HEIGHT DROP

ELEV. COLLAR Tech: S. Gilchrist
ELEV. GROUND 753.5
CO-ORD. LOCATION 109 + 75 - 50' Right

DEPTH / ELEV	O.D. / I.D.	BLOWS / FT	NO.	SYMBOL	DESCRIPTION OF MATERIAL
					1 — TOPSOIL
					CLAY - silty, grey
					6
	3"Sy		1		CLAY - medium plastic - brown - massive - silt pockets to ¼" in diam - very stiff
10					
743.5	3"Sy		2		
	3"Sy		3		
20					
733.5	3"Sy		4		
	3"Sy		5		27
30					CLAY - highly plastic - grey - massive - silt pockets to 1/8" in diam - sand pockets to ½" in diam - very stiff
723.5					
	Bag		6		37
40	Bag		7		TILL-LIKE - silty - gravelly - tan - clay seams < 1/8" thick to 39' - soft to firm
713.5			8		
	Bag		9		43
					45
					GLACIAL TILL - silty - tan - hard
50					
703.5					

NOTES:

1. Hole terminated at 45' on a boulder in glacial till.
2. Slight seepage at 45' or elevation 708.5.



Klohn Leonoff Consultants Ltd.
CIVIL & GEOTECHNICAL ENGINEERS

JOB No. WG0985
PROJECT Transportation Corridor
LOCATION Ft Garry/St Vital, Manitob
HOLE No. 402
DATE Feb 16/75 PLATE A-W-983-137

TEST HOLE LOG

SAMPLE DATA	ELEV. COLLAR Tech: S. Gilchrist
WEIGHT HAMMER	ELEV. GROUND 758.5
HEIGHT DROP	CO-ORD. LOCATION 110 + 75 - 60' Right

DEPTH / ELEV	O.D / I.D.	BLOWS / FT	NO.
	3"Sy		1
10 748.5	3"Sy		2
	3"Sy		3
20 748.5	3"Sy		4
	3"Sy		5
30 728.5	3"Sy		6
40			7
718.5	Bag		
	Bag		8
	Bag		9
	Bag		10
	Bag		11
50 708.5			

DESCRIPTION OF MATERIAL

1 TOPSOIL

CLAY
- silty
- medium plastic
- brown
- massive
- very stiff

12

CLAY
- highly plastic
- mottled brown
- massive
- small pebbles to 1/8" in diam
- silt pockets to 1/16" in diam
- very stiff

22

CLAY
- highly plastic
- grey
- massive
- sand seams 1/16" thick to 25'
- silt pockets to 3/8" in diam
- small pebbles to ¼" in diam
- firm to very stiff

40

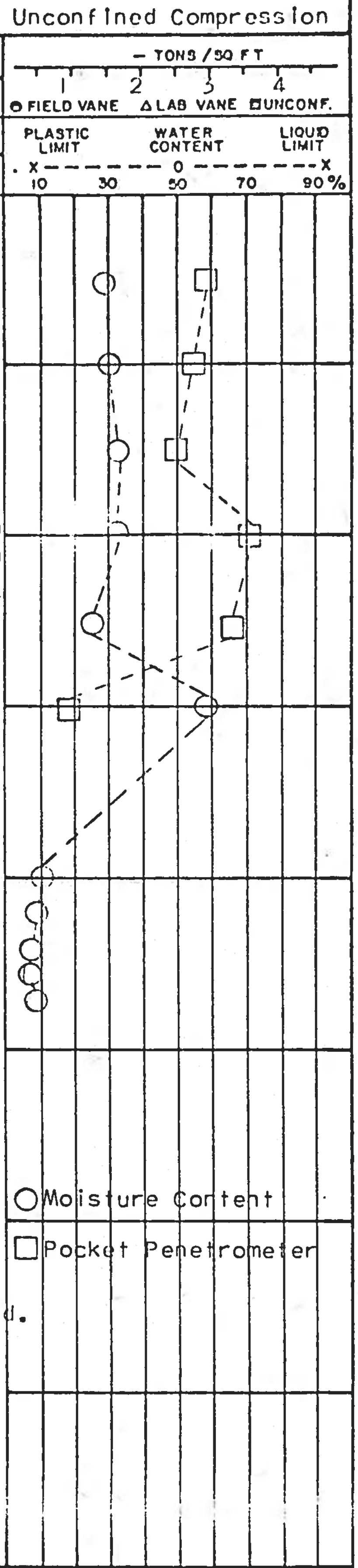
TILL-LIKE
- silty
- sandy
- gravelly
- tan
- soft

42

48

GLACIAL TILL
- silty
- tan
- hard to very hard

NOTES:
1. Hole terminated at 48' on a boulder in glacial till.
2. Water seepage from 47'.
3. About 6" of water in the bottom of the hole when drilling finished.



Klohn Leonoff Consultants Ltd.
CIVIL & GEOTECHNICAL ENGINEERS

JOB No. WG0983
PROJECT Transportation Corridor
LOCATION Ft Garry/St Vital, Manitob
HOLE No. 403
DATE Feb 16/75 PLATE A-W-983-138

TEST HOLE LOG

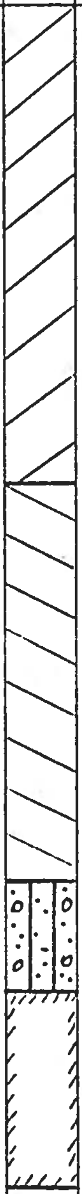
North Test hole
East Abutment

SAMPLE DATA
WEIGHT HAMMER 140
HEIGHT DROP 30"

Tech: J. A. Odermatt
ELEV. GROUND 760
CO-ORD. LOCATION 109 + 75; 120' left

- TONS / SQ FT: 1 2 3 4

PLASTIC LIMIT X — WATER CONTENT O — LIQUID LIMIT X: 10 30 50 70 90%



DEPTH / ELEV	O.D / I.D.	BLOWS / FT	NO.	DESCRIPTION OF MATERIAL
10				CLAY – mottled brown
750				
20				
740				24
30				CLAY COLOUR CHANGED TO GREY
730				
40				
720				45 GLACIAL TILL – soft to very stiff – grey
50	S.S.	35	1	50
710				LIMESTONE – hard – tight horizontal parting – whitish to cream – no water loss – smooth drilling – 21% to 70% recovery
60				60
700				
70				
690				

NOTES:
1. Hole terminated at 60'.
2. "B" casing to 50', couple of inches into rock.
3. Water at 24' the next morning or elevation 736.
4. Ford's Mayhew rig.
5. Coring 52'-55'4" - 70% recovery.
6. Coring 55'4"-60' - 21% recovery.
7. Possibly weathered to 52'.

○ Moisture Content
☐ Pocket Penetrometer Unconfined Compression Tons per sq ft.

Klohn Leonoff Consultants Ltd.
CIVIL & GEOTECHNICAL ENGINEERS

JOB No WG0083
PROJECT Transportation Corridor
LOCATION Ft Garry/St Vital, Manitoba
HOLE No. 1001
DATE Jan 20/76 PLATE A-W-083-140

South Test Hole
East Abutment

TEST HOLE LOG

SAMPLE DATA

WEIGHT HAMMER 140
HEIGHT DROP 30"

Tech: J. A. Odermatt
ELEV. GROUND 753
CO-ORD. LOCATION 109 + 40; 93' right

— TONS / SQ FT: 1 2 3 4

PLASTIC LIMIT X — WATER CONTENT O — LIQUID LIMIT X: 10 30 50 70 90%

DEPTH / ELEV	O D / I D	BLOWS / FT	NO.	DEPTH	DESCRIPTION OF MATERIAL
10 / 743					CLAY – mottled brown
20 / 733				20	CLAY COLOUR CHANGED TO GREY
30 / 723					
40 / 713	S.S.	30	1	37	GLACIAL TILL – soft to stiff 37-39' – stiff to very stiff 39-43'
				43	LIMESTONE – sound – very hard – white crystaline – no water loss – 25% to 80% recovery
50 / 703				53	
60 / 793					
70 / 783					

NOTES:

1. Hole terminated at 53'.
2. "B" casing to 43'.
3. No water loss in till and/or limestone.
4. Ford's Mayhew rig.
5. Coring 43'-48' – 25% recovery.
6. Coring 48'-53' – 80% recovery.
7. Possibly weathered rock 41'6" to 43'0".

○ Moisture Content
☐ Pocket Penetrometer Unconfined compression Tons per sq ft.

Klohn Leonoff Consultants Ltd.
CIVIL & GEOTECHNICAL ENGINEERS

JOB No. WG0983
PROJECT Transportation Corridor
LOCATION Ft Garry/St Vital, Manitoba
HOLE No. 1002
DATE Jan 21/76 PLATE A-W-983-J41

TEST HOLE LOG

North Test Hole
West Abutment

SAMPLE DATA
WEIGHT HAMMER 140
HEIGHT DROP 30"

Tech: J. A. Odermatt
ELEV. GROUND 756
CO-ORD. LOCATION 101 + 25; 120' left

— TONS / SQ FT
1 2 3 4

PLASTIC LIMIT x 10 · 30 · WATER CONTENT o 50 · 70 · LIQUID LIMIT x 90%

DEPTH ELEV	O.D I D	BLOWS FT	NO.	SYMBOL	DESCRIPTION OF MATERIAL
				SS	2 SNOW (snow dump area)
10 746					CLAY - mottled brown
20 736					22 CLAY COLOUR CHANGED TO GREY
30 726					
40 716	S.S.30/2"				41 GLACIAL TILL - firm to stiff - grey - till-like
					46 LIMESTONE - cream to rust 46-50' with some silty clay - sound, very hard, white crystaline with light horizontal partings - 45% to 75% recovery
50 706					55
60 996					
70 986					

NOTES:
1. Hole terminated at 55'.
2. Smooth drilling in rock.
3. No water loss.
4. Coring 46'-50'6" - 45% recovery.
5. Coring 50'6"-55' - 75% recovery.
6. Weathered rock 45'9" to 46'0".

○ Moisture Content

☐ Pocket Penetrometer Unconfined Compression Tons per sq ft.

Klohn Leonoff Consultants Ltd.
CIVIL & GEOTECHNICAL ENGINEERS

JOB No. WG0085
PROJECT Transportation Corridor
LOCATION Ft Garry/St Vital, Manitoba
HOLE No. 1003
DATE Jan 22/76 PLATE A-W-085-142

TEST HOLE LOG

South Test Hole
West Abutment

SAMPLE DATA
WEIGHT HAMMER 140
HEIGHT DROP 30"

Tech: J. A. Odermatt
ELEV. GROUND 756
CO-ORD. LOCATION 101 + 25; 111' right

– TONS / SQ FT
1 2 3 4
PLASTIC LIMIT · WATER CONTENT o LIQUID LIMIT x
10 30 50 70 90 %

DEPTH / ELEV	O.D / I.D	BLOWS / FT	NO.	DESCRIPTION OF MATERIAL
				CLAY – mottled brown
10 746				
20 736				22
				CLAY COLOUR CHANGED TO GREY
30 726				
40 716	S.S.	11/6"	1	41 GLACIAL TILL – firm to stiff – grey
	S.S.	30/½"	2	44½ LIMESTONE – sound, hard – brown to white – horizontal partings – 86% to 88% recovery
50 706				52
60 996				
70 986				

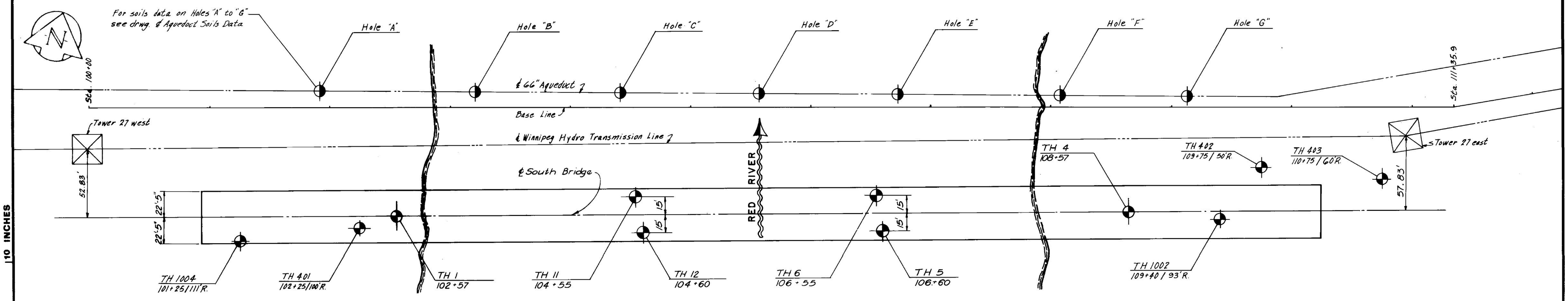
NOTES:
1. Hole terminated at 52'.
2. Coring 44'6"-48' – 86% recovery.
3. Coring 48'-52' – 88% recovery.

○ Moisture Content

☐ Pocket Penetrometer
Unconfined Compression
Tons per sq ft.

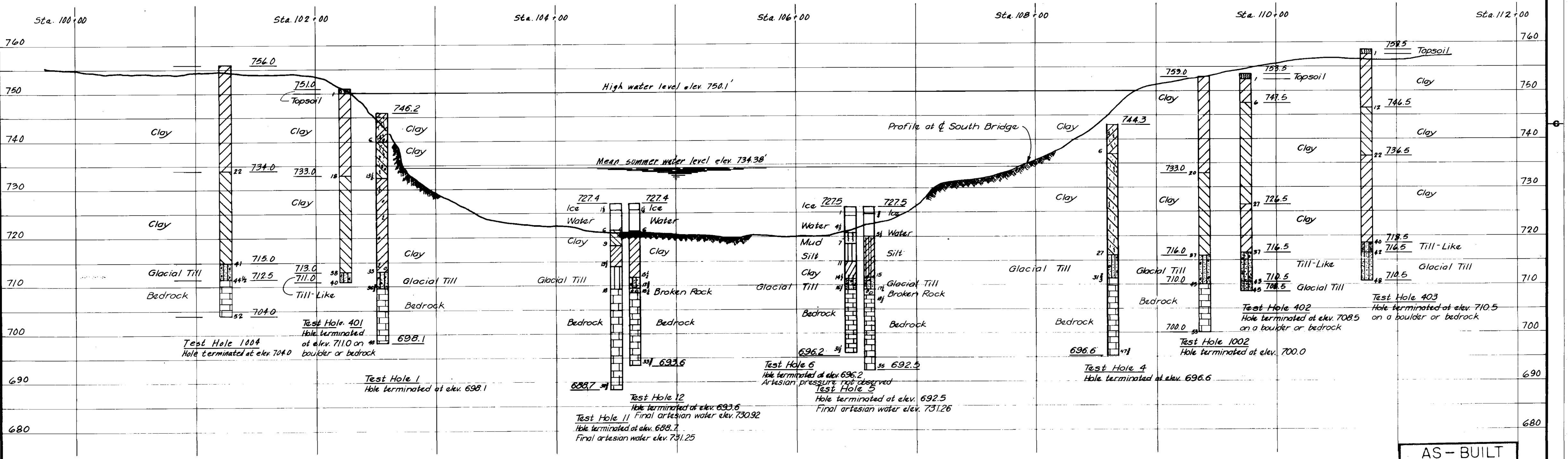
Klohn Leonoff Consultants Ltd.
CIVIL & GEOTECHNICAL ENGINEERS

JOB No. WG0983
PROJECT Transportation Corridor
LOCATION Ft Garry/St Vital, Manitoba
HOLE No. 1004
DATE Jan 23/76 PLATE A-W-083-J 43



TEST HOLE LOCATION PLAN

Scale : 1"= 40'-0"



TEST HOLE DATA

Scale : Horiz. 1"= 40'-0"
Vert. 1"= 10'-0"

Legend:

- ◐ Bridge test holes
- ◑ Aqueduct test holes for information only, taken from existing data (1960) see aqueduct soils data.

Notes:

1. Subsurface information shown on this drawing was obtained solely for use in establishing design controls for the project. This information is not guaranteed to be accurate or all-inclusive and it is not to be construed as part of the plans governing construction of the project. The Contractor is to satisfy himself as to actual conditions prevailing at the site.
2. Water levels measured at James Avenue Pumping Station.
3. The above drawing should be read in conjunction with Klohn Leonoff Consultants Ltd's Report W-1064.

AS-BUILT

DATE	F.B. NO.	PAGE
Nov. 16/79		

REVISIONS

D. L. MILLS March 24/77

THE CITY OF WINNIPEG
WORKS & OPERATIONS DEPARTMENT
STREETS & TRANSPORTATION DIVISION

W. L. WARDROP & ASSOCIATES LTD.
ENGINEERING CONSULTANTS
WINNIPEG · THUNDER BAY · REGINA · BARRIE · EDMONTON

APPROVED BY: DK Grimes DATE 25 Mar/77

DRAWN BY LMG
PRELIM. CHK. LWS Jan/77
DESIGN KWS DATE NOV.76
CHECK Jan/77

ROUTE 165

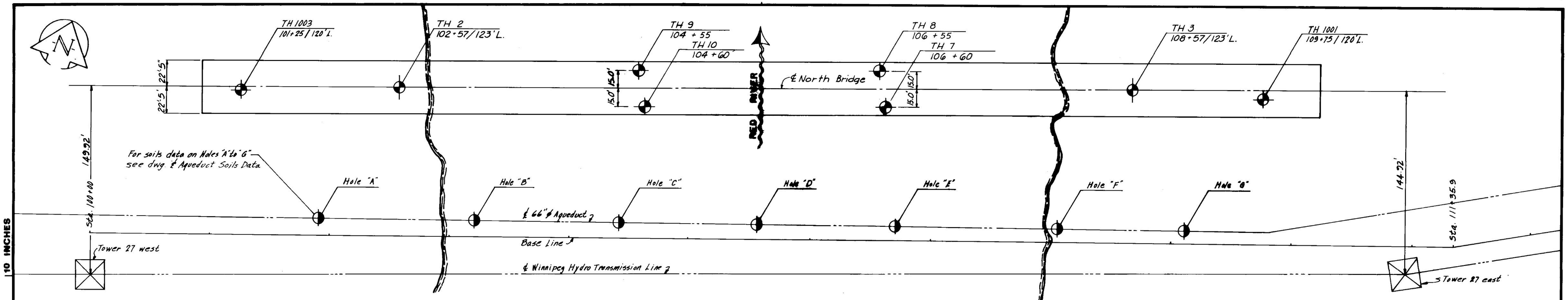
SOUTH BRIDGE SOILS DATA

APPROVED BY: Donald R Campbell 25/3/77
MANAGER OF STREETS AND TRAFFIC DATE

SCALE: AS SHOWN

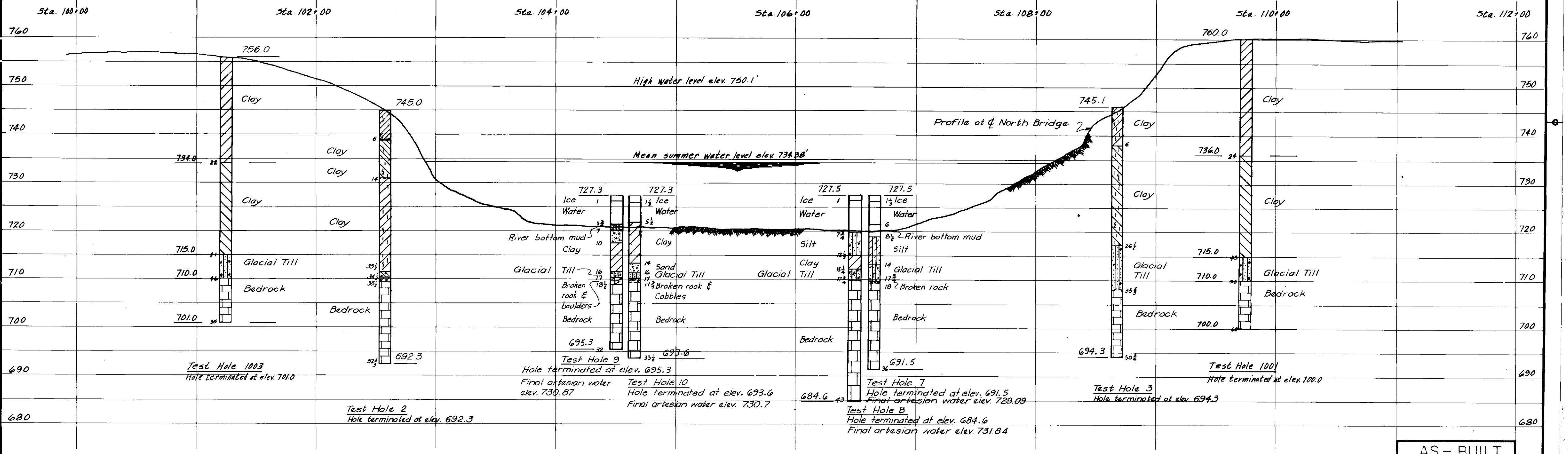
DRAWING NO. B-5092-205

W.L.W. NO. 74012-21



TEST HOLE LOCATION PLAN

Scale: 1"=40'-0"



TEST HOLE DATA

Scale: Horiz. 1"=40'-0"
Vert. 1"=10'-0"

Legend:

- ● Bridge test holes
- ◐ Aqueduct test holes for information only, taken from existing data (1960) see aqueduct soils data.

Notes:

1. Subsurface information shown on this drawing was obtained solely for use in establishing design controls for the project. This information is not guaranteed to be accurate or all-inclusive and it is not to be construed as part of the plans governing construction of the project. The Contractor is to satisfy himself as to actual condition prevailing at the site.
2. Water levels measured at James Avenue Pumping Station.
3. The above drawing should be read in conjunction with Klohn Leonoff Consultant Ltd's Report W-1064.

AS-BUILT

DATE	FB NO	PAGE
Nov. 16/79		

ISSUED FOR TENDER

NO.	REVISIONS		BY

D. L. MILLS
March 24/77

THE CITY OF WINNIPEG
WORKS & OPERATIONS DEPARTMENT
STREETS & TRANSPORTATION DIVISION

W. L. WARDROP & ASSOCIATES LTD.
ENGINEERING CONSULTANTS
WINNIPEG · THUNDER BAY · REGINA · BARRIE · EDMONTON

APPROVED BY: DR Greene DATE 25 Mar/77

DRAWN BY LMG DATE
PRELIM. CHK. LWS Jan 1/77
DESIGN KWS DATE NOV.76
CHECK DM Jan/77

ROUTE 165

NORTH BRIDGE SOILS DATA

APPROVED BY: Lowell R Campbell 25/3/77
MANAGER OF STREETS AND TRAFFIC DATE

SCALE: AS SHOWN

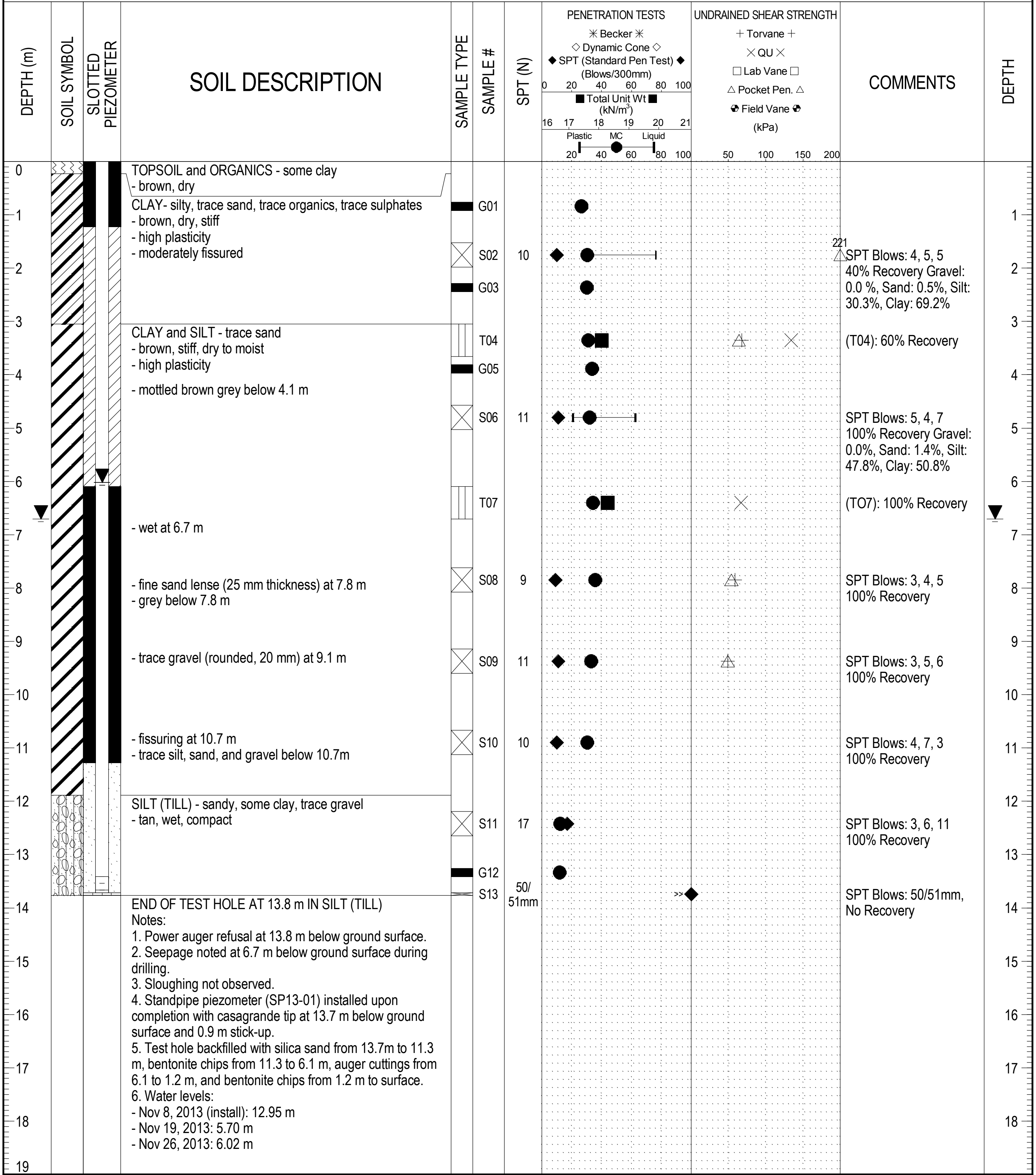
DRAWING NO. B-5092-206

W.L.W. NO. 74012-21

PROJECT: FGSV Interceptor Siphon	CLIENT: City of Winnipeg	TESTHOLE NO: **TH13-01**
LOCATION: Upper Bank of Red River, UTM: 14 U, N 5520496, E 0633705		PROJECT NO.: 60274906
CONTRACTOR: Paddock Drilling	METHOD: Truck Mounted Acker MP-8	ELEVATION (m):

SAMPLE TYPE: ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ▨ NO RECOVERY ▮ CORE

BACKFILL TYPE: ■ BENTONITE ⁘ GRAVEL ▥ SLOUGH ⁙ GROUT ▨ CUTTINGS ⁘ SAND



Depth (m)	Soil Description
0	TOPSOIL and ORGANICS - some clay - brown, dry
0.6	CLAY- silty, trace sand, trace organics, trace sulphates - brown, dry, stiff - high plasticity - moderately fissured
3.0	CLAY and SILT - trace sand - brown, stiff, dry to moist - high plasticity - mottled brown grey below 4.1 m - wet at 6.7 m - fine sand lense (25 mm thickness) at 7.8 m - grey below 7.8 m - trace gravel (rounded, 20 mm) at 9.1 m - fissuring at 10.7 m - trace silt, sand, and gravel below 10.7m
11.9	SILT (TILL) - sandy, some clay, trace gravel - tan, wet, compact

Sample Type	Sample #	SPT (N)	Comments
■	G01		
☒	S02	10	SPT Blows: 4, 5, 5 40% Recovery Gravel: 0.0 %, Sand: 0.5%, Silt: 30.3%, Clay: 69.2%
■	G03		
▥	T04		(T04): 60% Recovery
■	G05		
☒	S06	11	SPT Blows: 5, 4, 7 100% Recovery Gravel: 0.0%, Sand: 1.4%, Silt: 47.8%, Clay: 50.8%
▥	T07		(TO7): 100% Recovery
☒	S08	9	SPT Blows: 3, 4, 5 100% Recovery
☒	S09	11	SPT Blows: 3, 5, 6 100% Recovery
☒	S10	10	SPT Blows: 4, 7, 3 100% Recovery
☒	S11	17	SPT Blows: 3, 6, 11 100% Recovery
■	G12		
☒	S13	50/ 51mm	SPT Blows: 50/51mm, No Recovery

END OF TEST HOLE AT 13.8 m IN SILT (TILL)

Notes:

1. Power auger refusal at 13.8 m below ground surface.
2. Seepage noted at 6.7 m below ground surface during drilling.
3. Sloughing not observed.
4. Standpipe piezometer (SP13-01) installed upon completion with casagrande tip at 13.7 m below ground surface and 0.9 m stick-up.
5. Test hole backfilled with silica sand from 13.7m to 11.3 m, bentonite chips from 11.3 to 6.1 m, auger cuttings from 6.1 to 1.2 m, and bentonite chips from 1.2 m to surface.
6. Water levels:
- Nov 8, 2013 (install): 12.95 m
- Nov 19, 2013: 5.70 m
- Nov 26, 2013: 6.02 m

AECOM

LOGGED BY: Aaron Kaluzniak	COMPLETION DEPTH: 13.76 m
REVIEWED BY: Alex Hill	COMPLETION DATE: 11/8/13
PROJECT ENGINEER: Marvin McDonald	Page 1 of 1

PROJECT: FGSV Interceptor Siphon	CLIENT: City of Winnipeg	TESTHOLE NO: **TH13-02**
LOCATION: Lower Bank of Red River, UTM: 14 U, N 5520490, E 0633691		PROJECT NO.: 60274906
CONTRACTOR: Paddock Drilling	METHOD: Truck Mounted Acker SS-3	ELEVATION (m):

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◿ NO RECOVERY ▌ CORE

BACKFILL TYPE ■ BENTONITE ⁘ GRAVEL ▥ SLOUGH ⁘ GROUT ▨ CUTTINGS ⁘ SAND

SOIL DESCRIPTION

TOPSOIL and ORGANICS - some clay
- brown, dry

CLAY- trace to some sand, trace silt, trace organics
- grey-brown, dry to moist, firm to stiff
- Intermediate to high plasticity

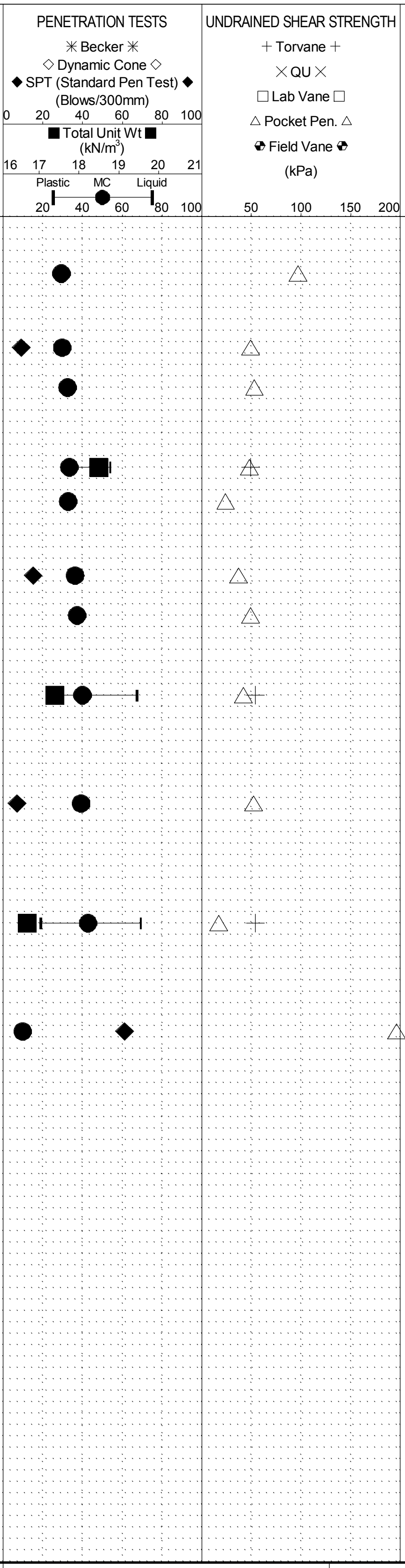
CLAY and SILT - trace sand, trace organics
- brown, firm to stiff, dry to moist
- high plasticity
- greyish brown below 3.5 m
- grey, moist, silty, below 5.0 m

CLAY- silty
- brown to greyish brown, firm, moist
- high plasticity
- grey, wet below 7.2 m
- intermittant sand seams (<25 mm thickness) below 7.2 m
- fine sand layer (<76 mm thickness) between 8.10 m and 8.20 m
- grey, very soft below 9.1 m
- trace gravel below 9.8 m

SILT (TILL) - gravelly, some sand, trace to some clay
- tan, wet, compact to very dense

END OF TEST HOLE AT 11.6 m IN SILT (TILL)
Notes:
1. Power auger refusal at 11.6 m below ground surface on suspected bedrock.
2. Seepage noted at 4.9 m below ground surface during drilling.
3. No sloughing observed.
4. Standpipe piezometer (SP13-02) installed upon completion with casagrande tip at 11.6 m below ground surface and 0.91 m stick-up.
5. Test hole backfilled with silica sand from 11.6m to 10.4 m, bentonite chips from 10.4 to ground surface.
6. Water levels:
   - Nov 19, 2013 (install): 10.29 m
   - Nov 26, 2013: 5.97 m

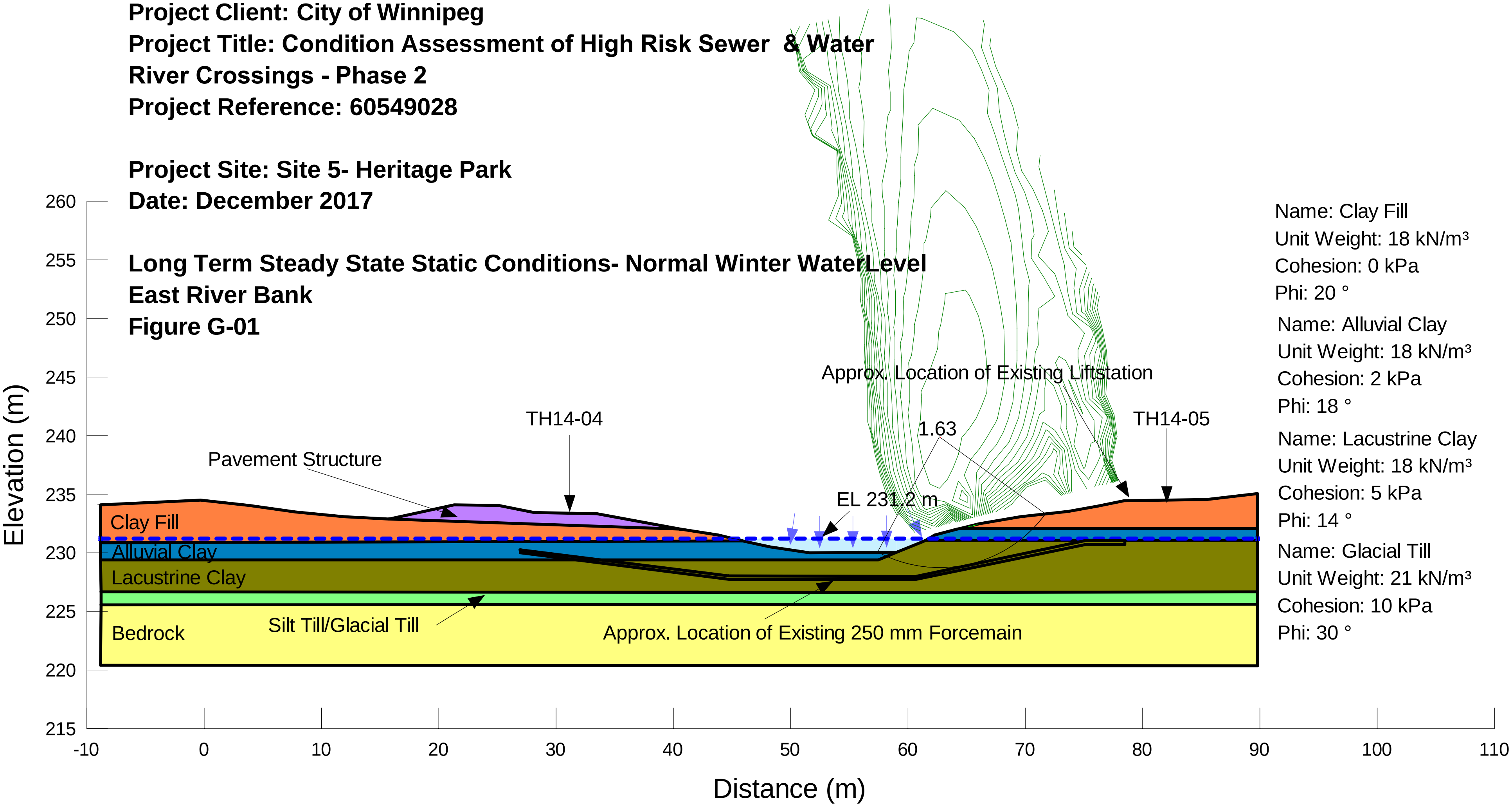
SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS
■	G1		
☒	S2	9	SPT Blows: 3, 4, 5 61% Recovery
■	G3		
▥	T4		100% Recovery
■	G5		Gravel: 0.1 %, Sand: 5.2%, Silt: 44.0%, Clay: 50.7%
☒	S6	15	SPT Blows: 3, 6, 9 100% Recovery
■	G7		Gravel: 0.0 %, Sand: 0.0%, Silt: 39.0%, Clay: 61.0%
▥	T8		100% Recovery
☒	S9	7	SPT Blows: 3, 4, 3 100% Recovery
▥	T10		100% Recovery Gravel: 1.4 %, Sand: 10.6%, Silt: 27.9%, Clay: 60.1%
☒	S11	61	SPT Blows: 20, 28, 33 78% Recovery
	S12		SPT Blows: 51/0 mm



LOGGED BY: Sam Oshati	COMPLETION DEPTH: 11.58 m
REVIEWED BY: Alex Hill	COMPLETION DATE: 11/19/13
PROJECT ENGINEER: Marvin McDonald	

Appendix G

Slope Stability Outputs



Project Client: City of Winnipeg
Project Title: Condition Assessment of High Risk Sewer & Water
River Crossings - Phase 2
Project Reference: 60549028

Project Site: Site 5- Heritage Park
Date: December 2017

Long Term Steady State Static Conditions- Normal Winter WaterLevel
East River Bank
Figure G-01

Name: Clay Fill
Unit Weight: 18 kN/m³
Cohesion: 0 kPa
Phi: 20 °

Name: Alluvial Clay
Unit Weight: 18 kN/m³
Cohesion: 2 kPa
Phi: 18 °

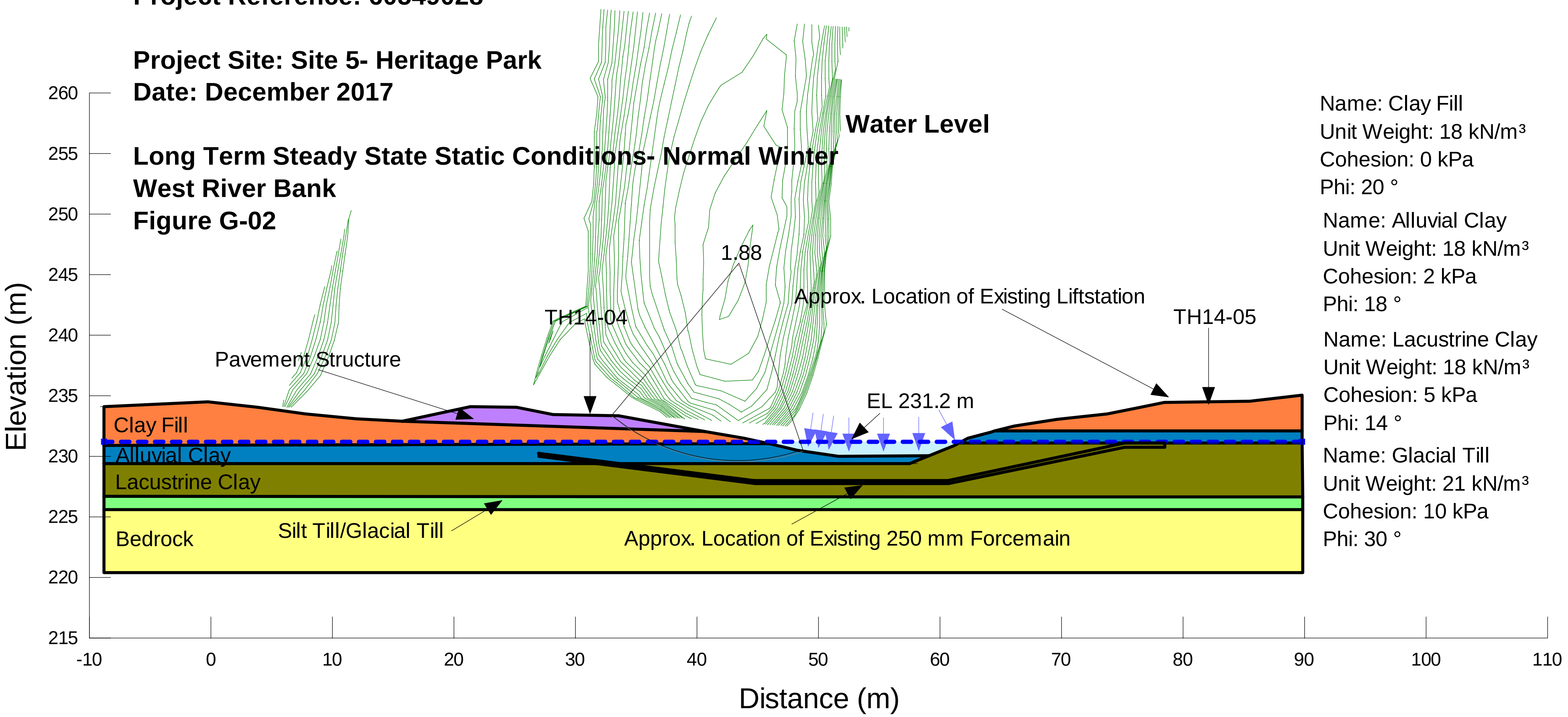
Name: Lacustrine Clay
Unit Weight: 18 kN/m³
Cohesion: 5 kPa
Phi: 14 °

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

Project Client: City of Winnipeg
Project Title: Condition Assessment of High Risk Sewer
& Water River Crossings - Phase 2
Project Reference: 60549028

Project Site: Site 5- Heritage Park
Date: December 2017

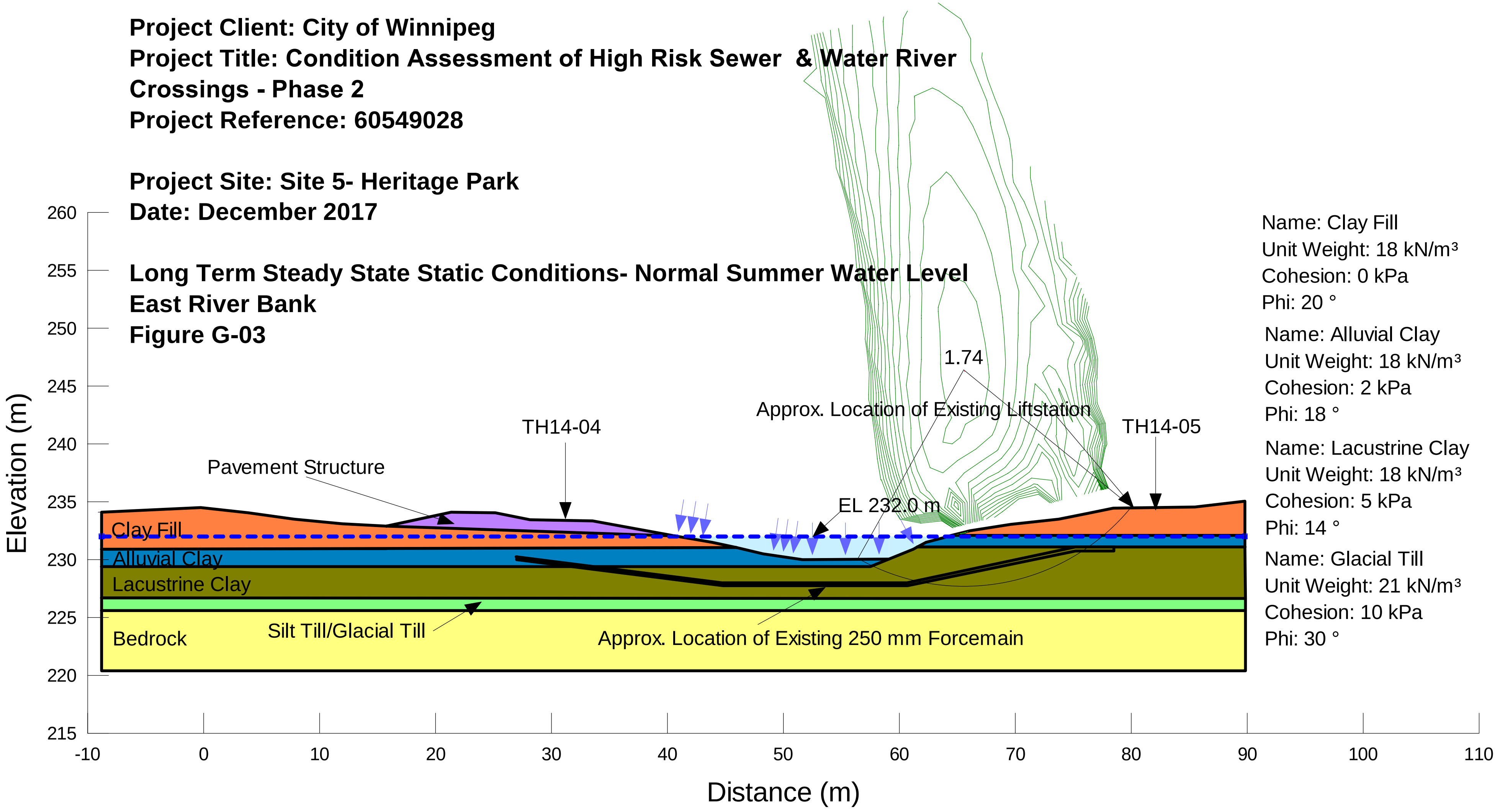
Long Term Steady State Static Conditions- Normal Winter
West River Bank
Figure G-02



Project Client: City of Winnipeg
Project Title: Condition Assessment of High Risk Sewer & Water River Crossings - Phase 2
Project Reference: 60549028

Project Site: Site 5- Heritage Park
Date: December 2017

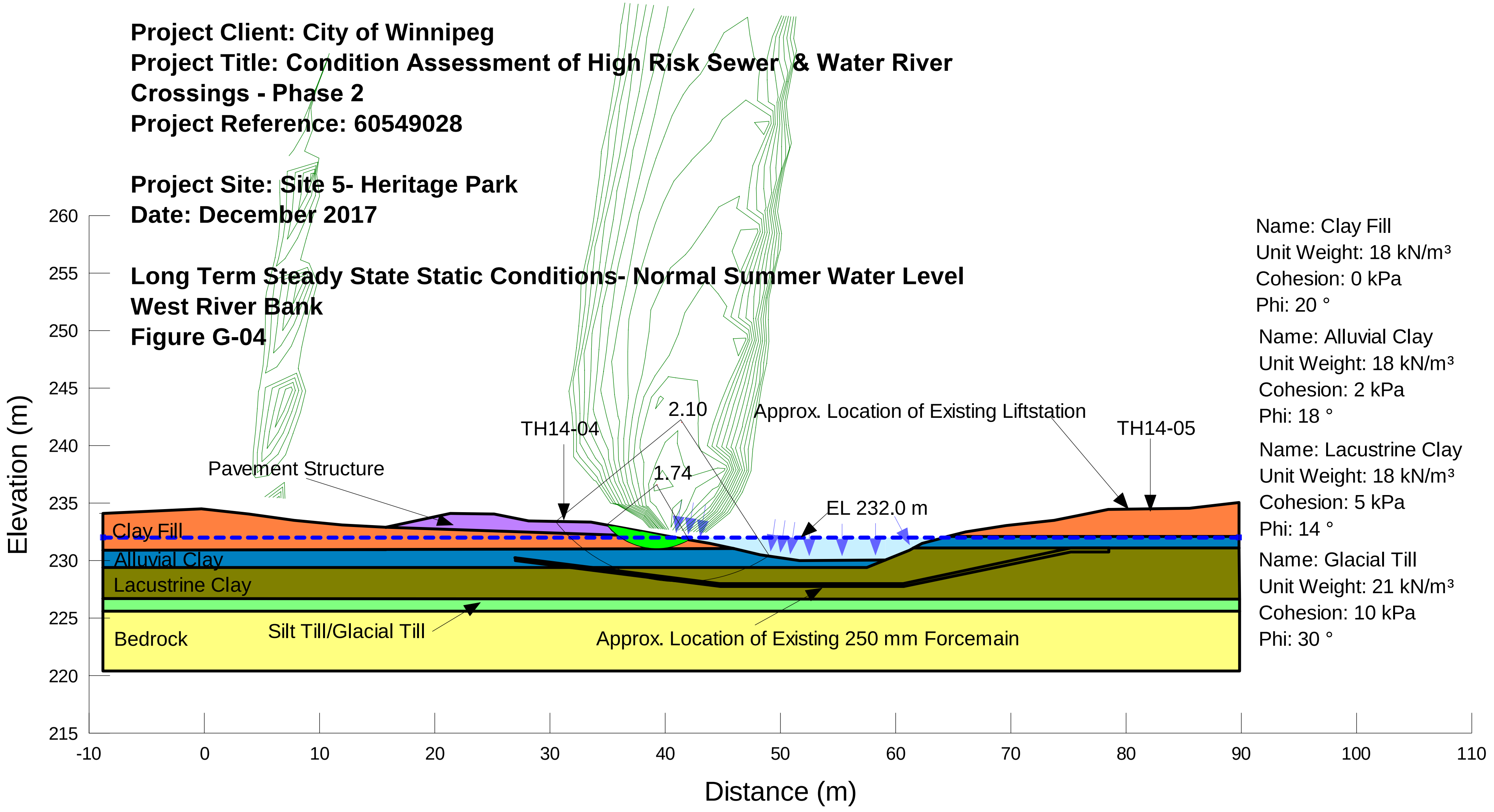
Long Term Steady State Static Conditions- Normal Summer Water Level
East River Bank
Figure G-03



Project Client: City of Winnipeg
Project Title: Condition Assessment of High Risk Sewer & Water River Crossings - Phase 2
Project Reference: 60549028

Project Site: Site 5- Heritage Park
Date: December 2017

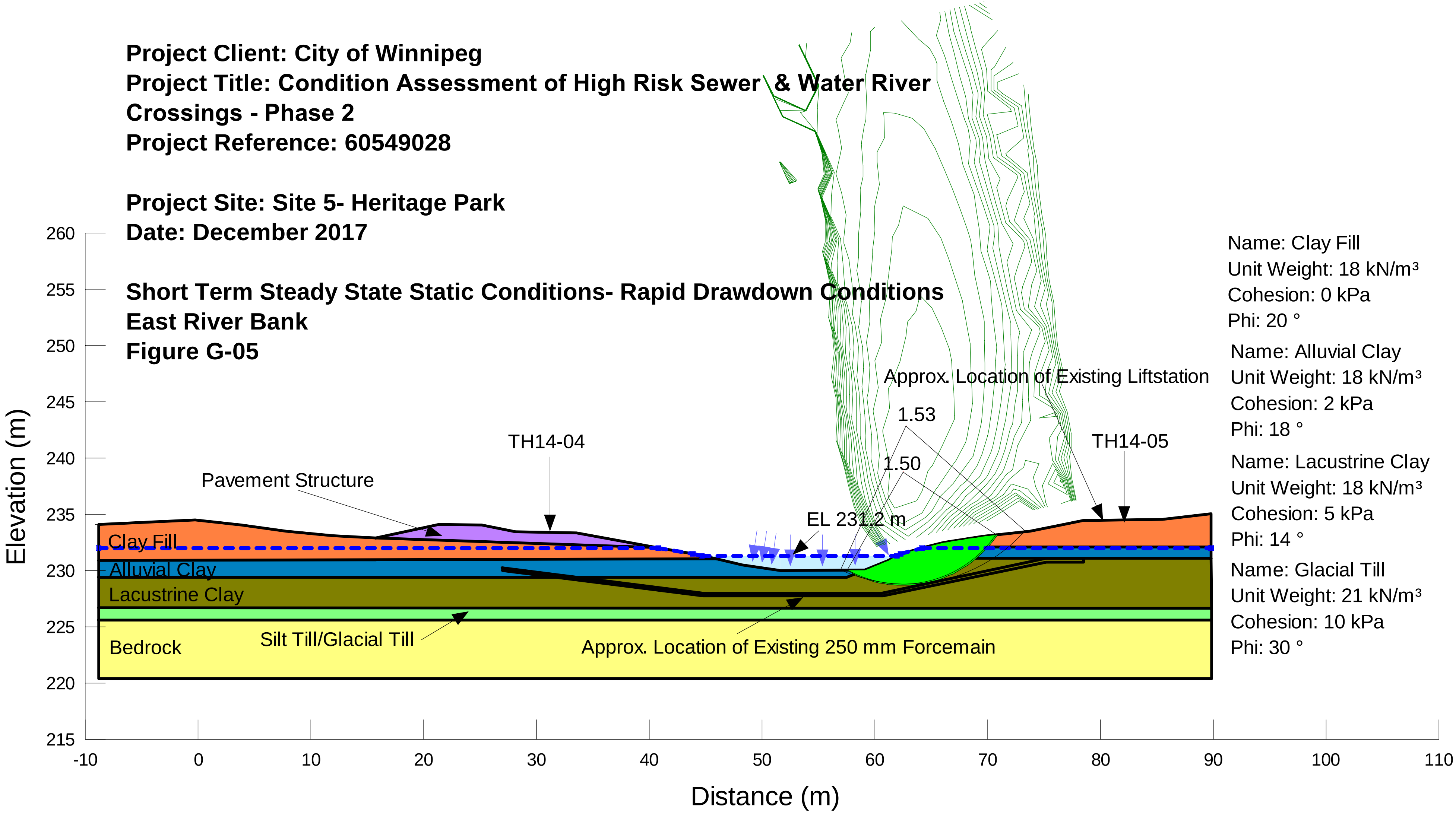
Long Term Steady State Static Conditions- Normal Summer Water Level
West River Bank
Figure G-04



Project Client: City of Winnipeg
Project Title: Condition Assessment of High Risk Sewer & Water River Crossings - Phase 2
Project Reference: 60549028

Project Site: Site 5- Heritage Park
Date: December 2017

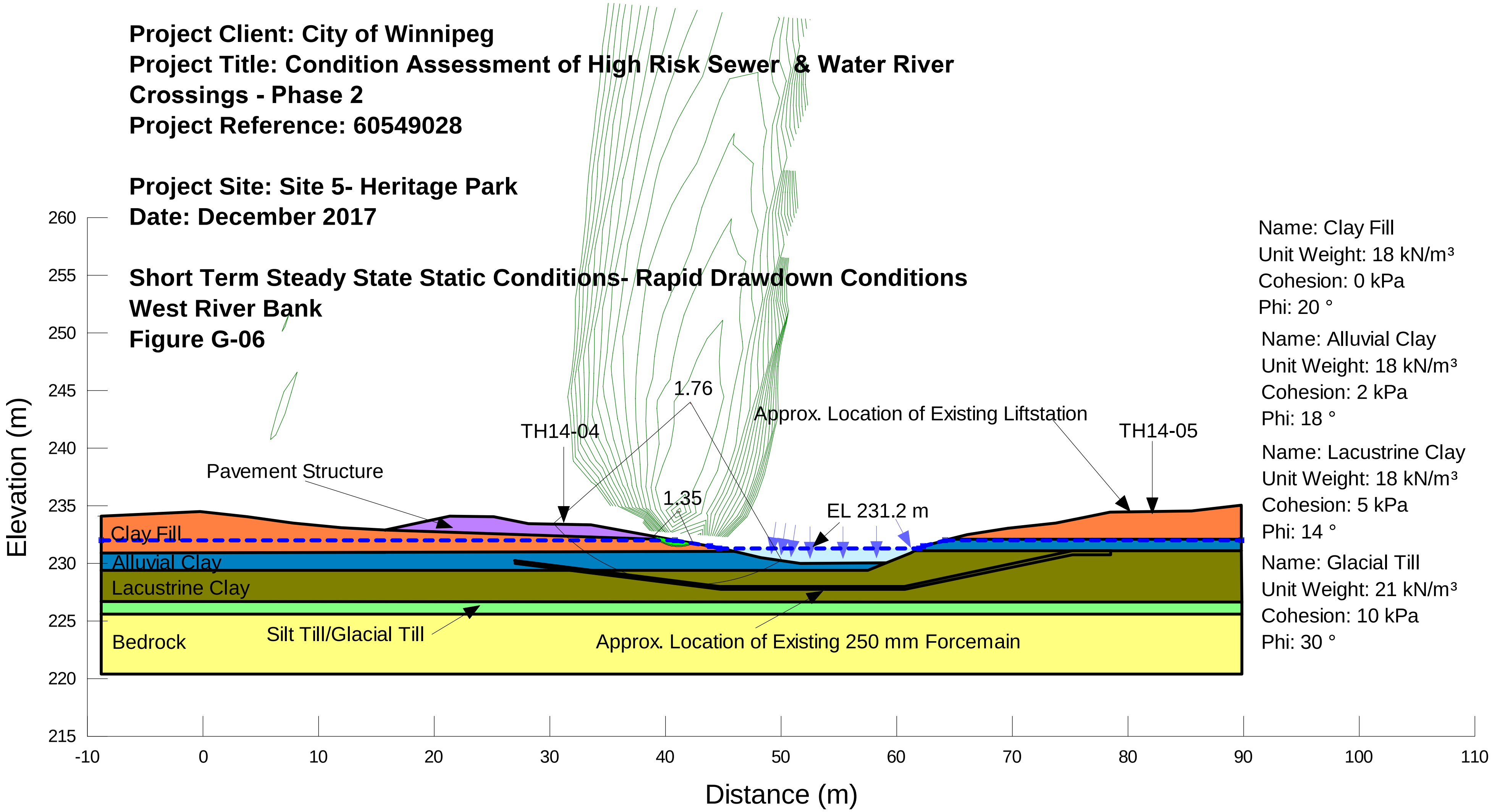
Short Term Steady State Static Conditions- Rapid Drawdown Conditions
East River Bank
Figure G-05



Project Client: City of Winnipeg
Project Title: Condition Assessment of High Risk Sewer & Water River Crossings - Phase 2
Project Reference: 60549028

Project Site: Site 5- Heritage Park
Date: December 2017

Short Term Steady State Static Conditions- Rapid Drawdown Conditions
West River Bank
Figure G-06



Project Client: City of Winnipeg
Project Title: Condition Assessment of High Risk Sewer & Water River Crossings - Phase 2
Project Reference: 60549028

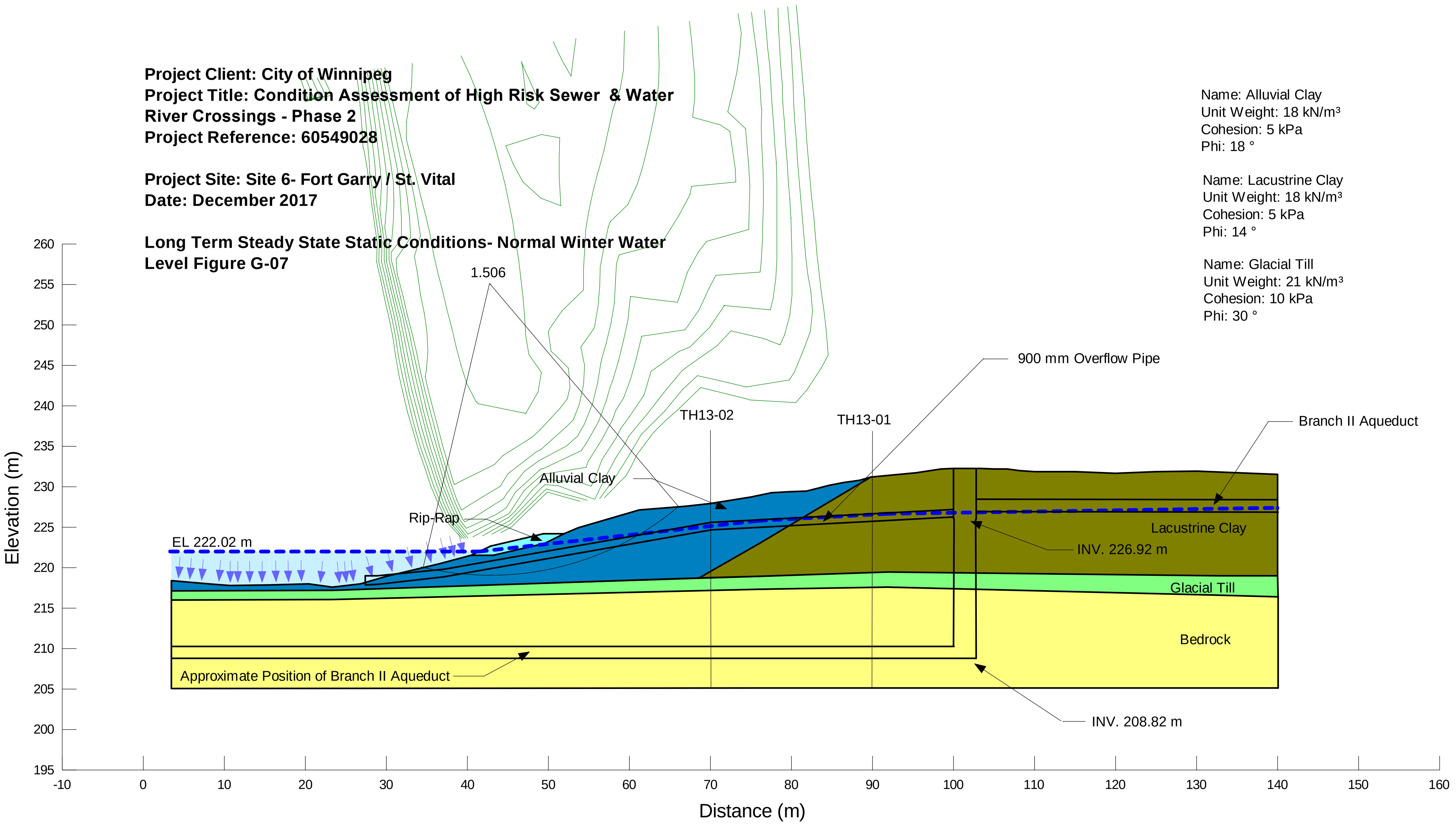
Project Site: Site 6- Fort Garry / St. Vital
Date: December 2017

Long Term Steady State Static Conditions- Normal Winter Water Level Figure G-07

Name: Alluvial Clay
Unit Weight: 18 kN/m³
Cohesion: 5 kPa
Phi: 18 °

Name: Lacustrine Clay
Unit Weight: 18 kN/m³
Cohesion: 5 kPa
Phi: 14 °

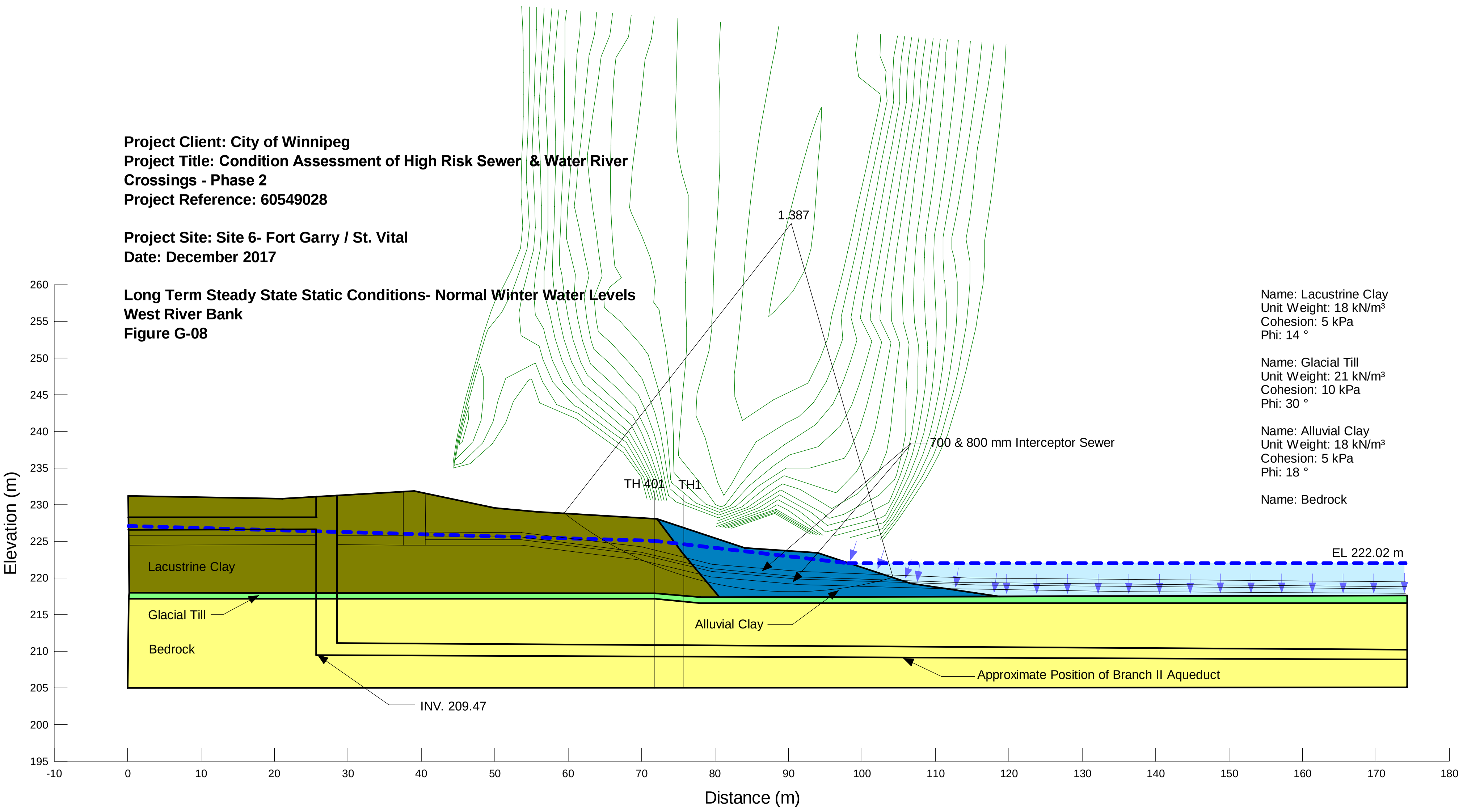
Name: Glacial Till
Unit Weight: 21 kN/m³
Cohesion: 10 kPa
Phi: 30 °



Project Client: City of Winnipeg
Project Title: Condition Assessment of High Risk Sewer & Water River Crossings - Phase 2
Project Reference: 60549028

Project Site: Site 6- Fort Garry / St. Vital
Date: December 2017

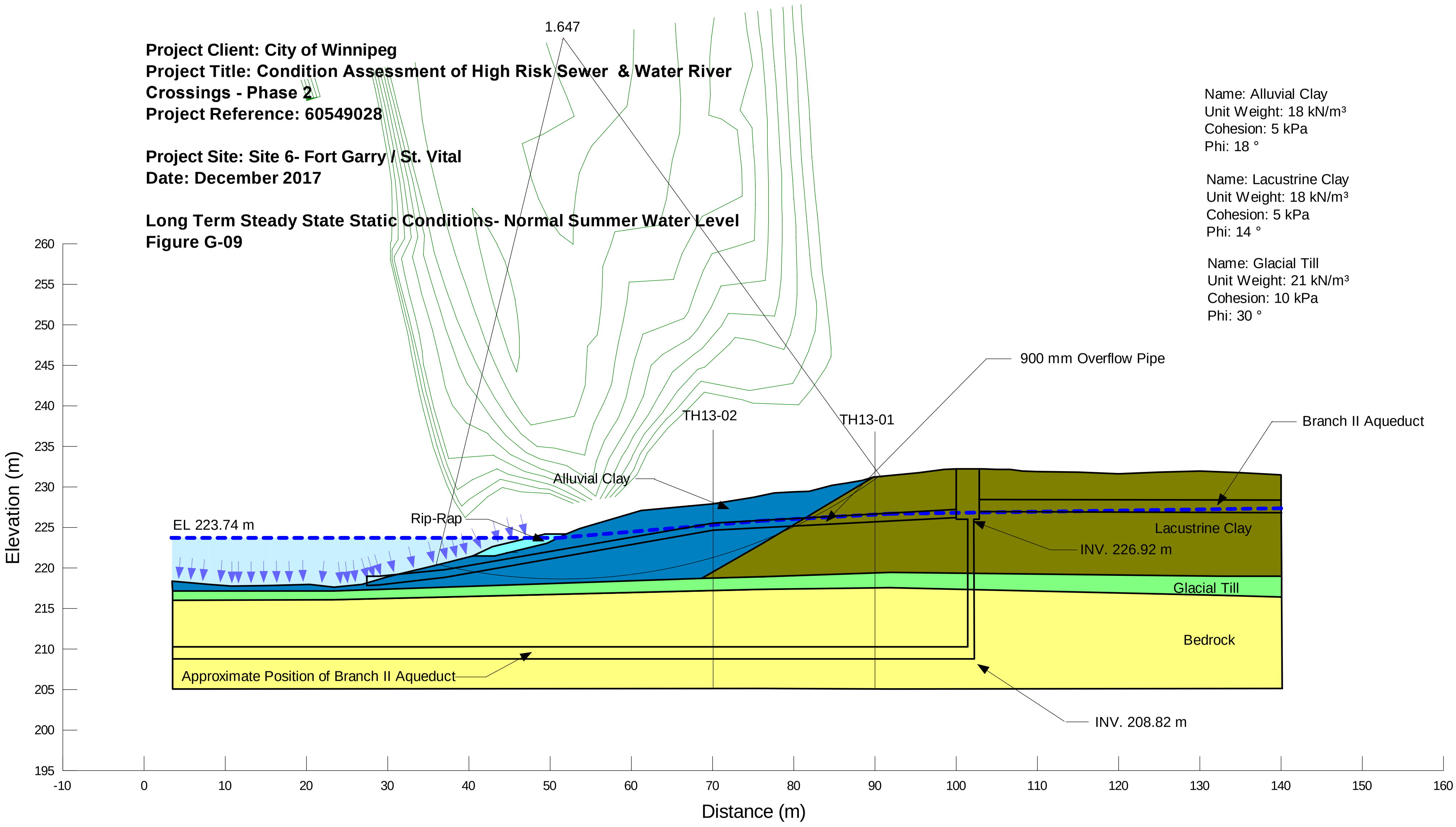
Long Term Steady State Static Conditions- Normal Winter Water Levels
West River Bank
Figure G-08



Project Client: City of Winnipeg
Project Title: Condition Assessment of High Risk Sewer & Water River Crossings - Phase 2
Project Reference: 60549028

Project Site: Site 6- Fort Garry / St. Vital
Date: December 2017

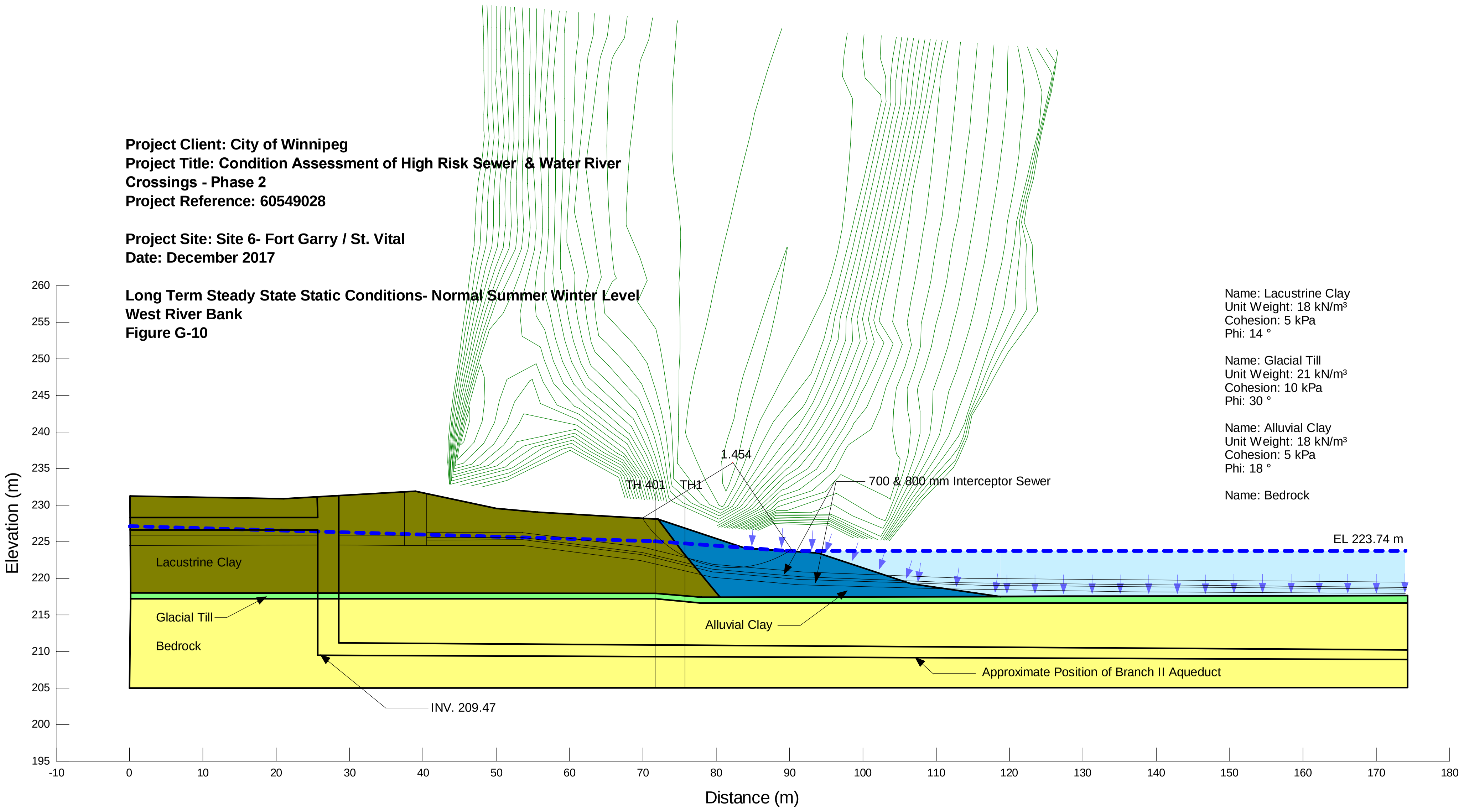
Long Term Steady State Static Conditions- Normal Summer Water Level
Figure G-09



Project Client: City of Winnipeg
Project Title: Condition Assessment of High Risk Sewer & Water River Crossings - Phase 2
Project Reference: 60549028

Project Site: Site 6- Fort Garry / St. Vital
Date: December 2017

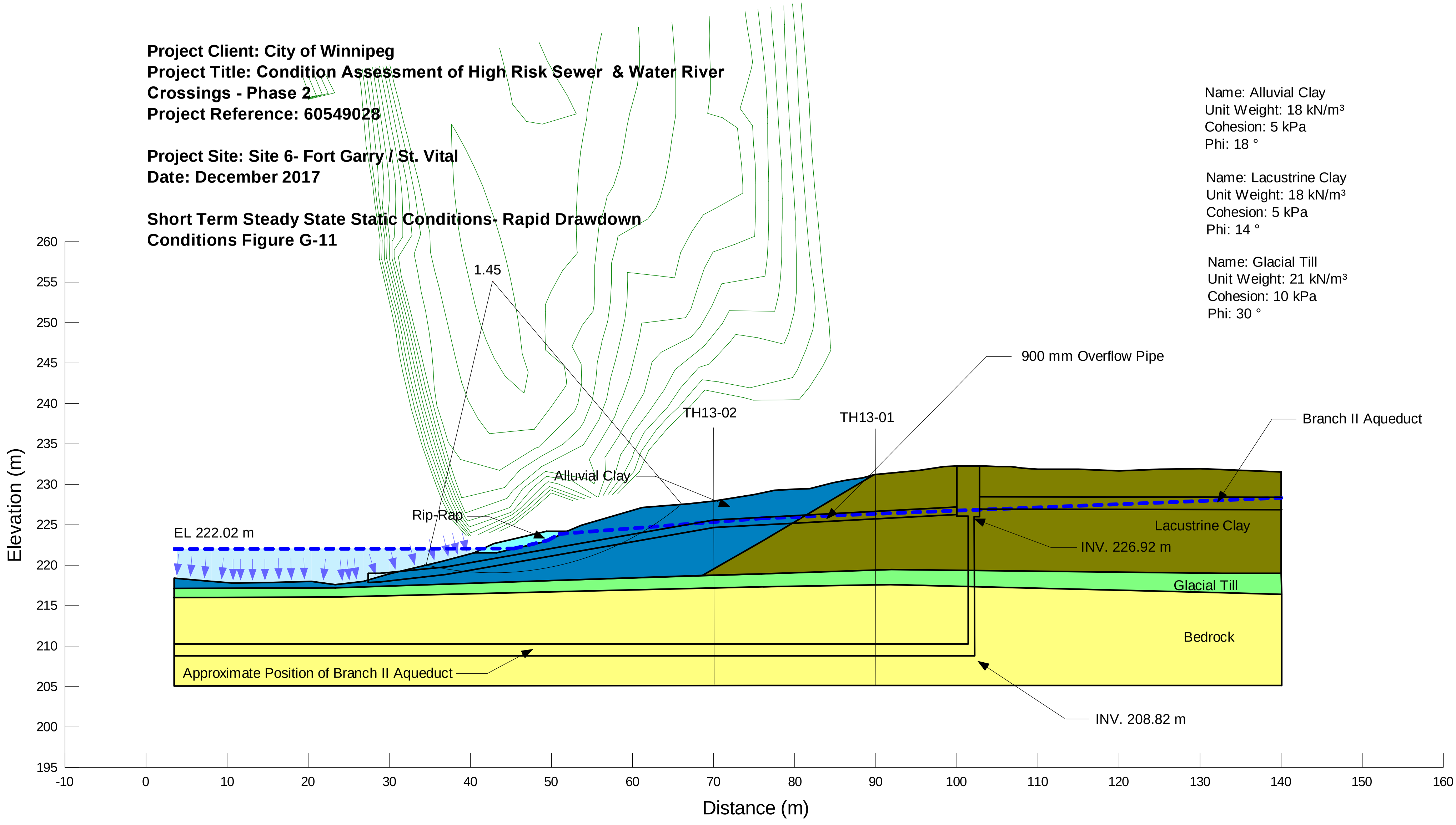
Long Term Steady State Static Conditions- Normal Summer Winter Level
West River Bank
Figure G-10



Project Client: City of Winnipeg
Project Title: Condition Assessment of High Risk Sewer & Water River Crossings - Phase 2
Project Reference: 60549028

Project Site: Site 6- Fort Garry / St. Vital
Date: December 2017

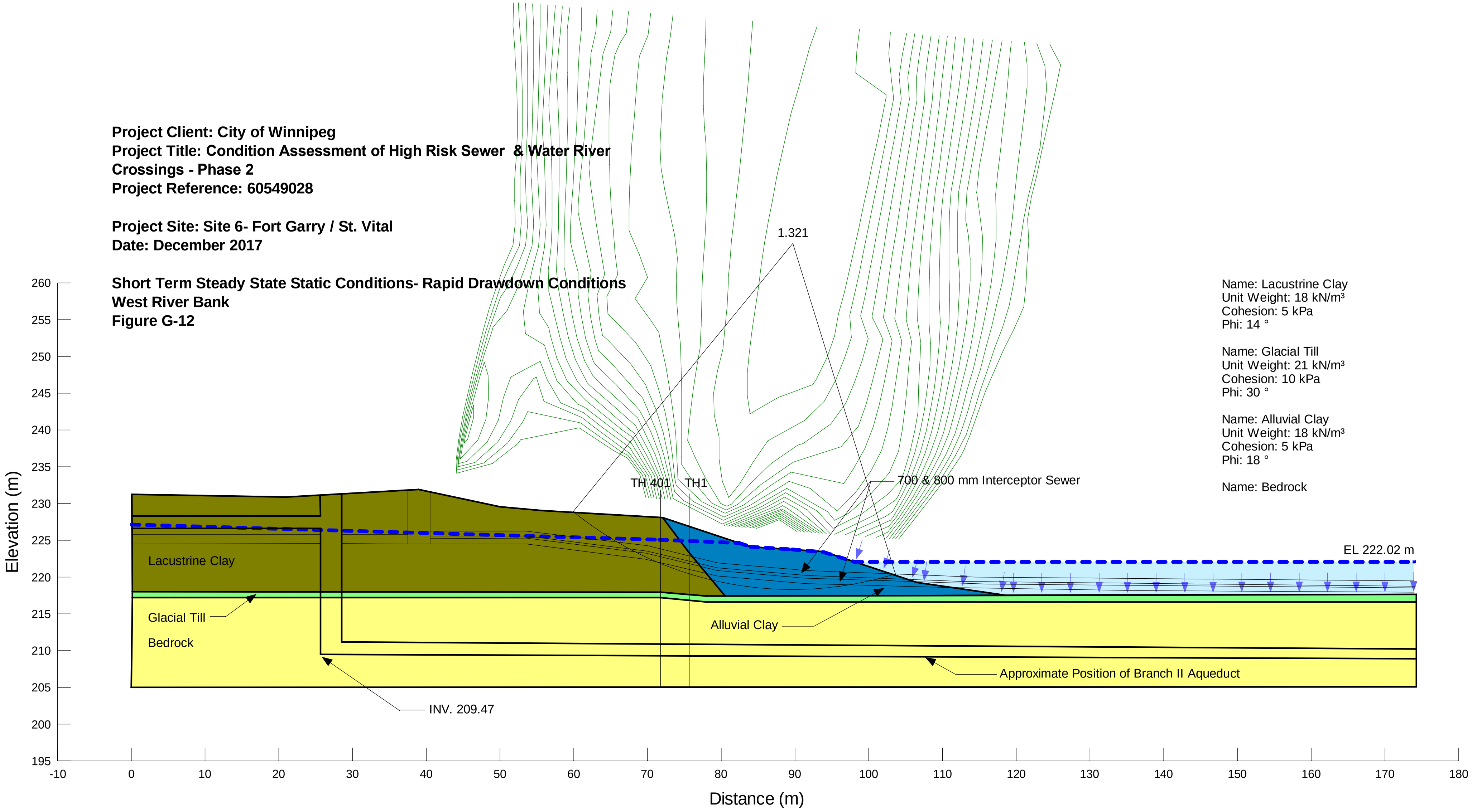
Short Term Steady State Static Conditions- Rapid Drawdown Conditions Figure G-11



Project Client: City of Winnipeg
Project Title: Condition Assessment of High Risk Sewer & Water River Crossings - Phase 2
Project Reference: 60549028

Project Site: Site 6- Fort Garry / St. Vital
Date: December 2017

Short Term Steady State Static Conditions- Rapid Drawdown Conditions
West River Bank
Figure G-12



AECOM

AECOM
99 Commerce Drive
Winnipeg, MB, Canada R3P 0Y7
www.aecom.com

204 477 5381 tel
204 284 2040 fax

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 insides 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.

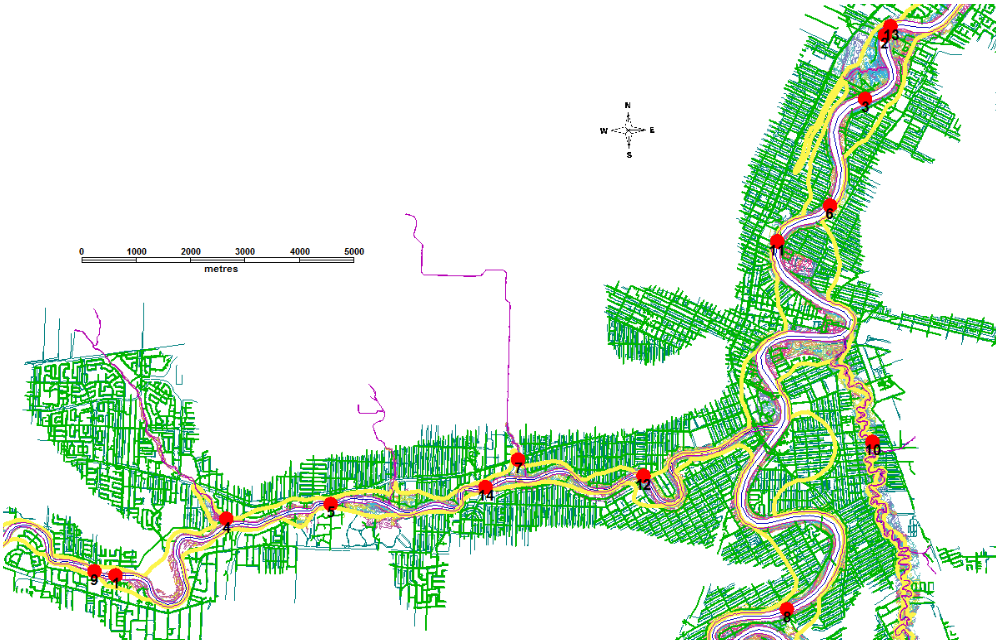


Figure 1 – River crossing locations by site number as described in Table 1.

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 signicant 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 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.

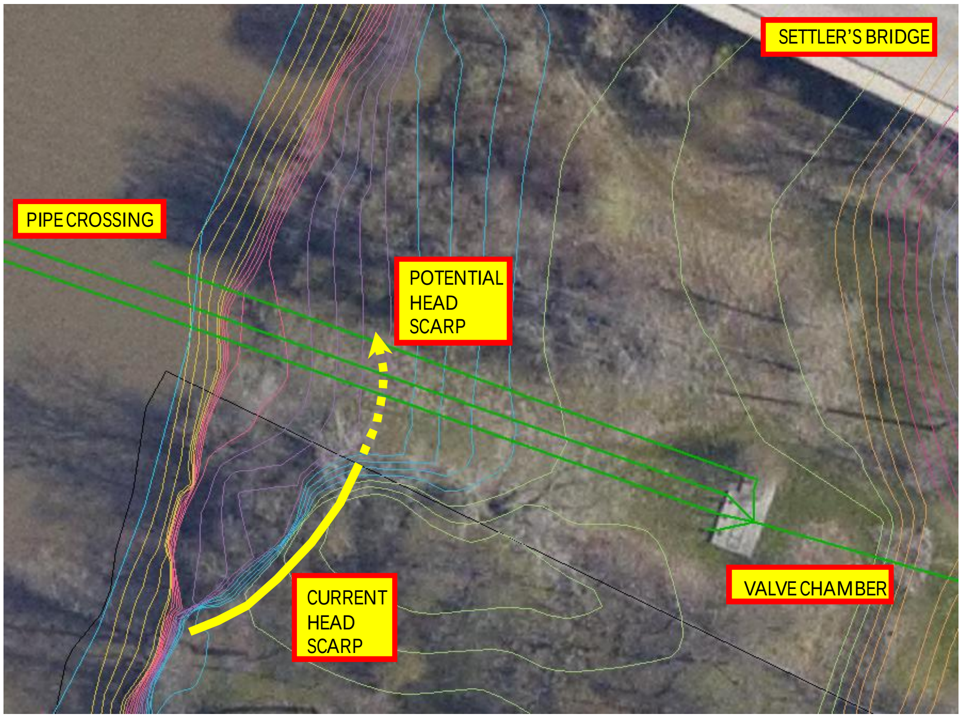


Figure 3 – Scarp south (upstream) of ROW on east bank from 1998 contour data.

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).

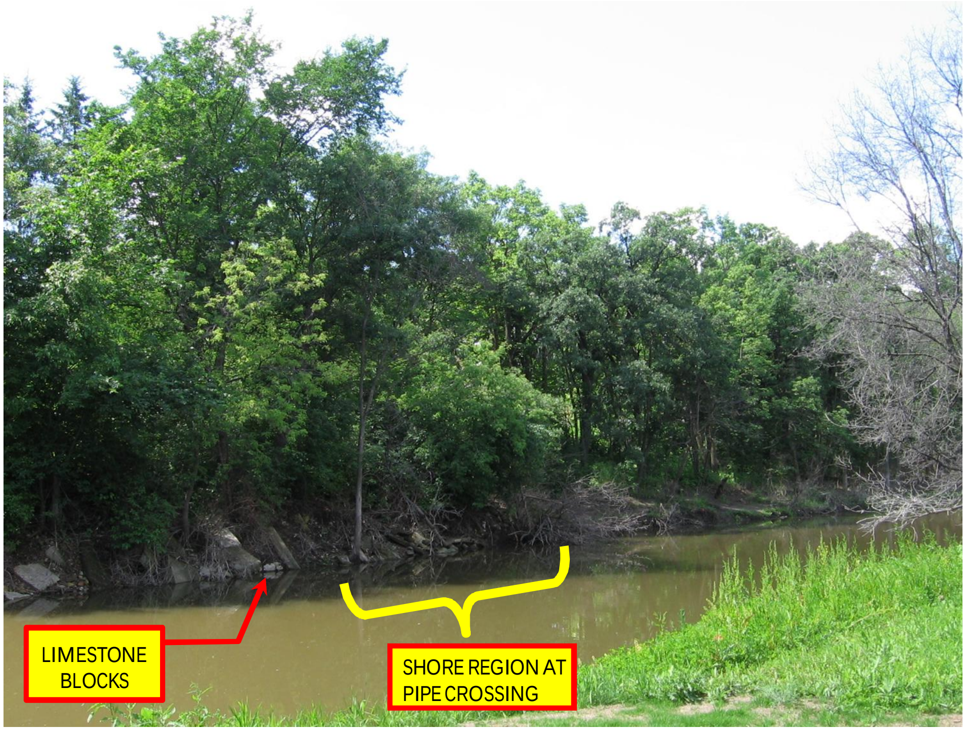


Figure 7 – Steep south bank with sparse limestone blocks and rubble at river edge.

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.

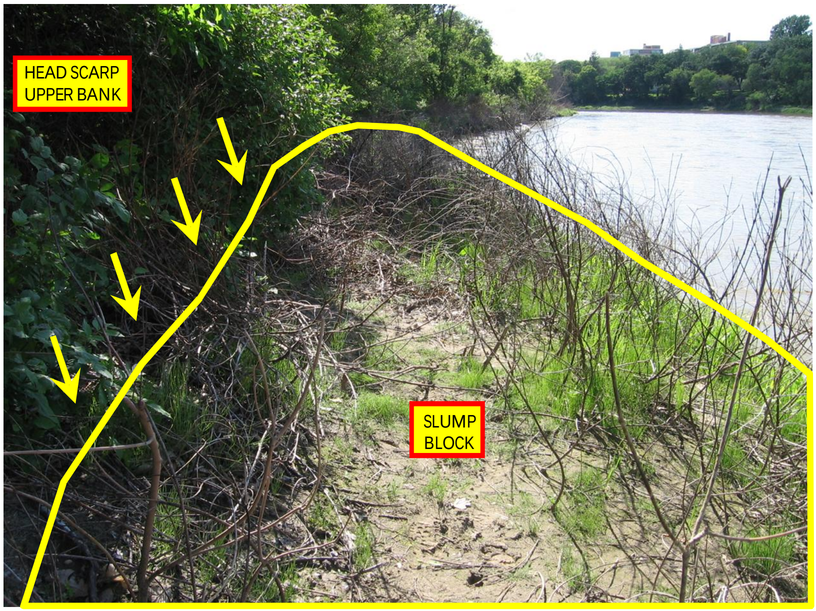


Figure 8 – Top of slump block facing upstream (west) from location of pipe crossing.

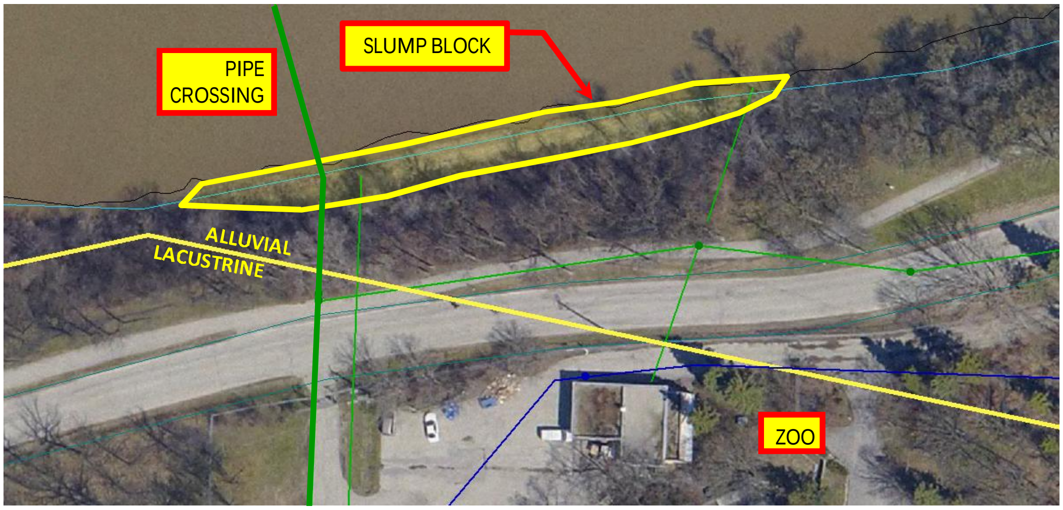


Figure 9 – Extent of slump block relative to pipe crossing.

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 or 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.

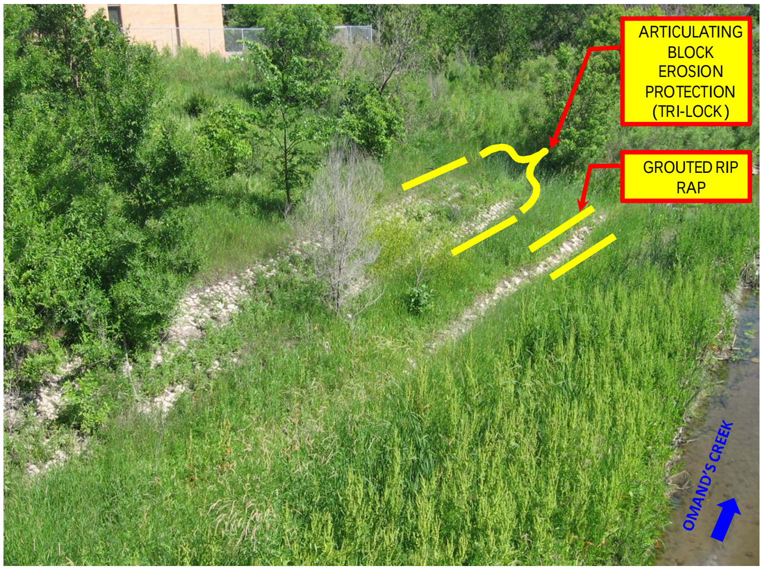


Figure 14 – Articulating block erosion protection (Tri-Lock) and grouted rip rap on east bank. Photograph taken from bridge at Portage Avenue.

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.

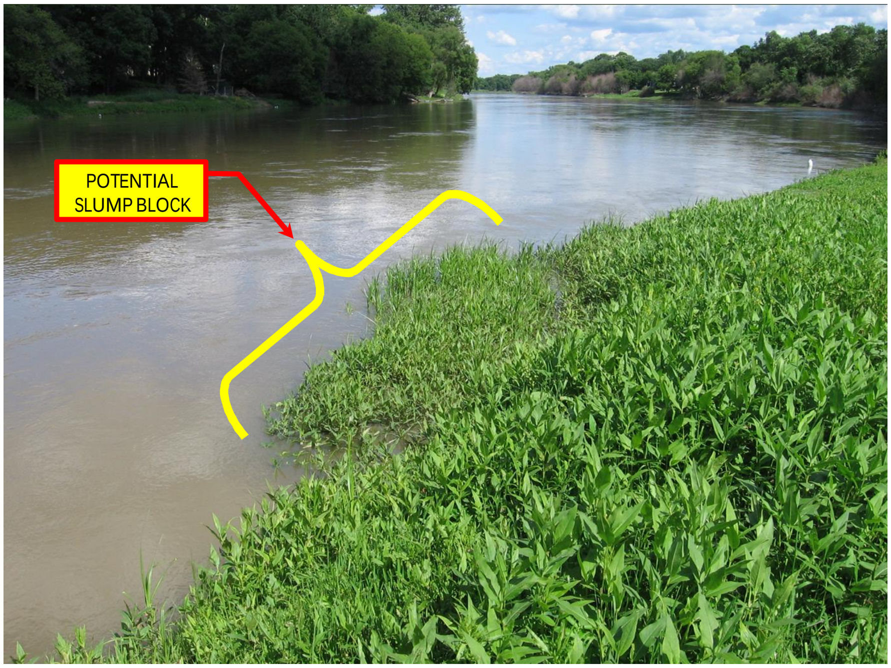


Figure 17 – Partially submerged potential slump block at river bank toe at crossing location.

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.

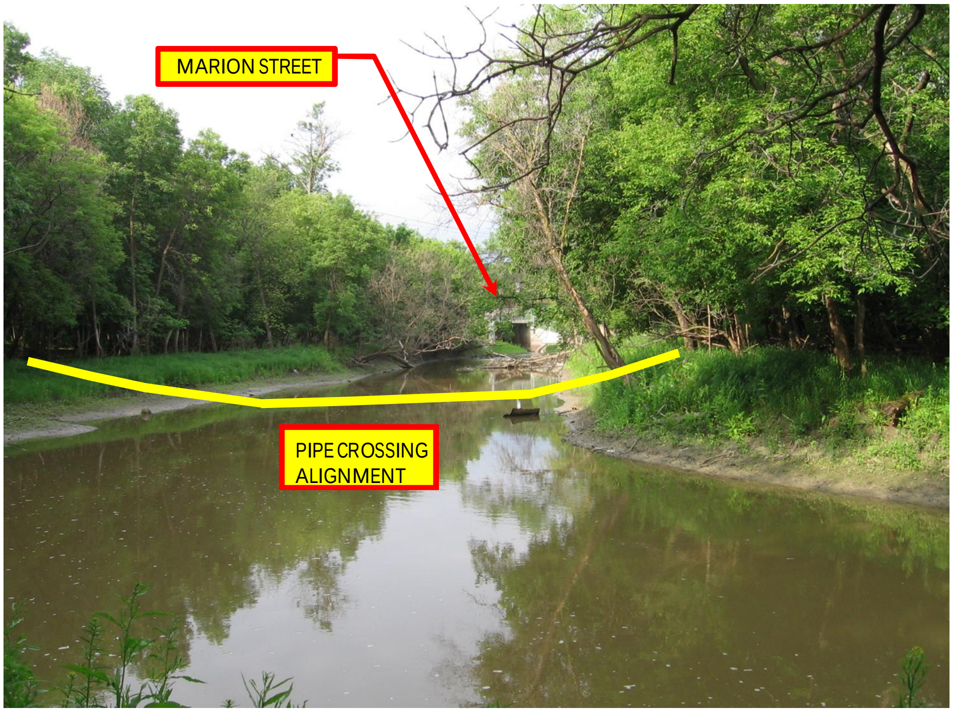


Figure 18 – Flood plain at crossing location.

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 – Gulley 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.

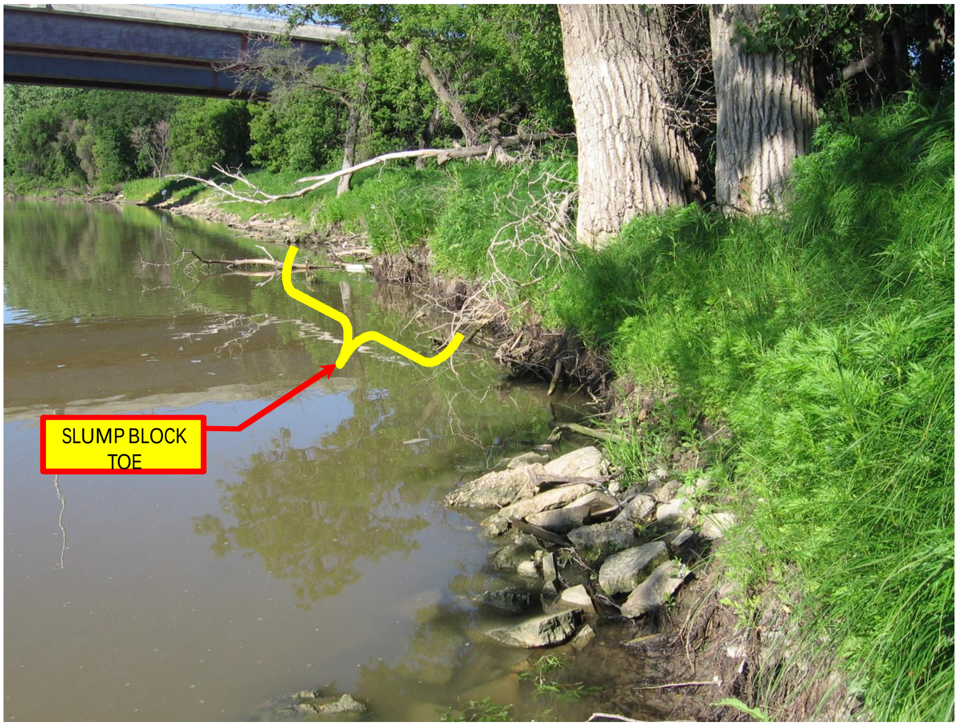


Figure 21 – Toe erosion and toe of slump block upstream (south) of pipe crossing alignment. Photograph taken from pipe crossing.

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.

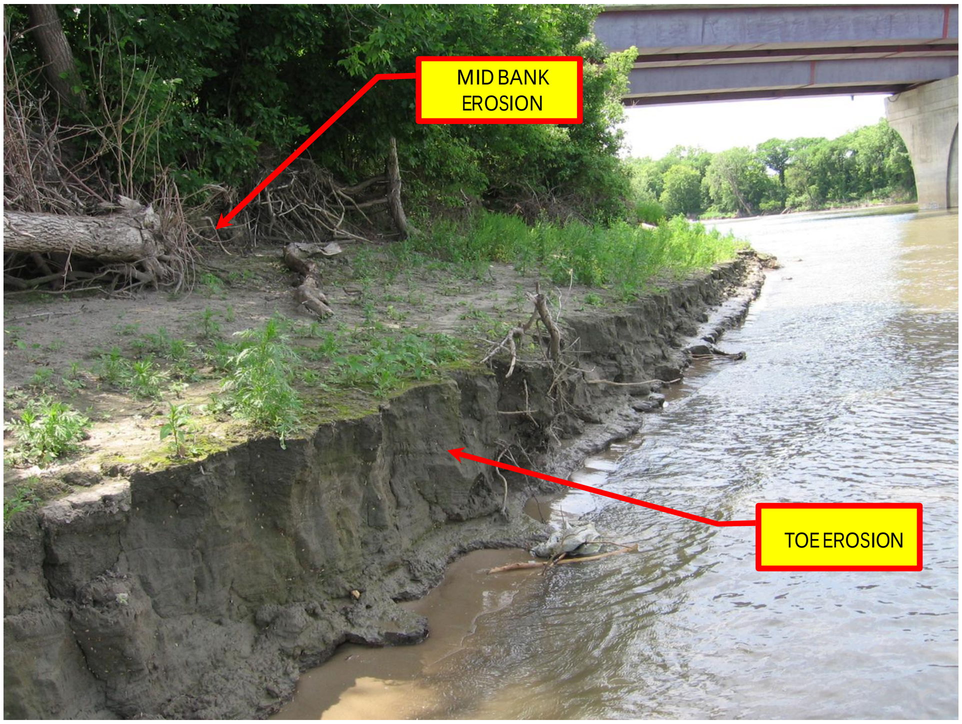


Figure 22 – Toe and mid bank erosion on east bank facing (upstream) south.

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

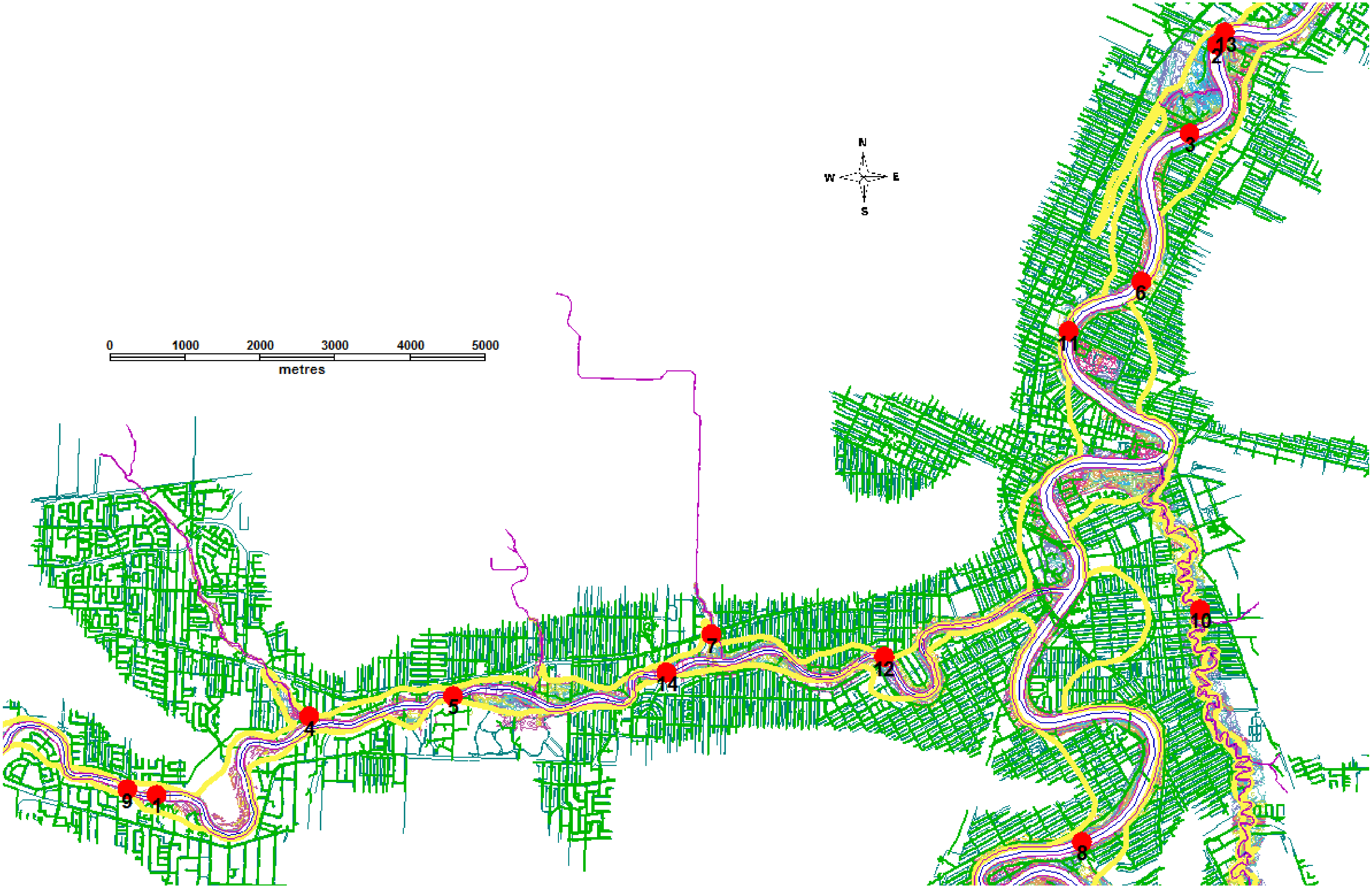
The south bank is located on Wellington Crescent immediately east of St. James Bridge and the abandoned railway bridge crossing. This site was visited on June 26 and August 15, 2012. The river water elevation decreased during this period and allowed more of the lower bank to be visible at the river edge. 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.

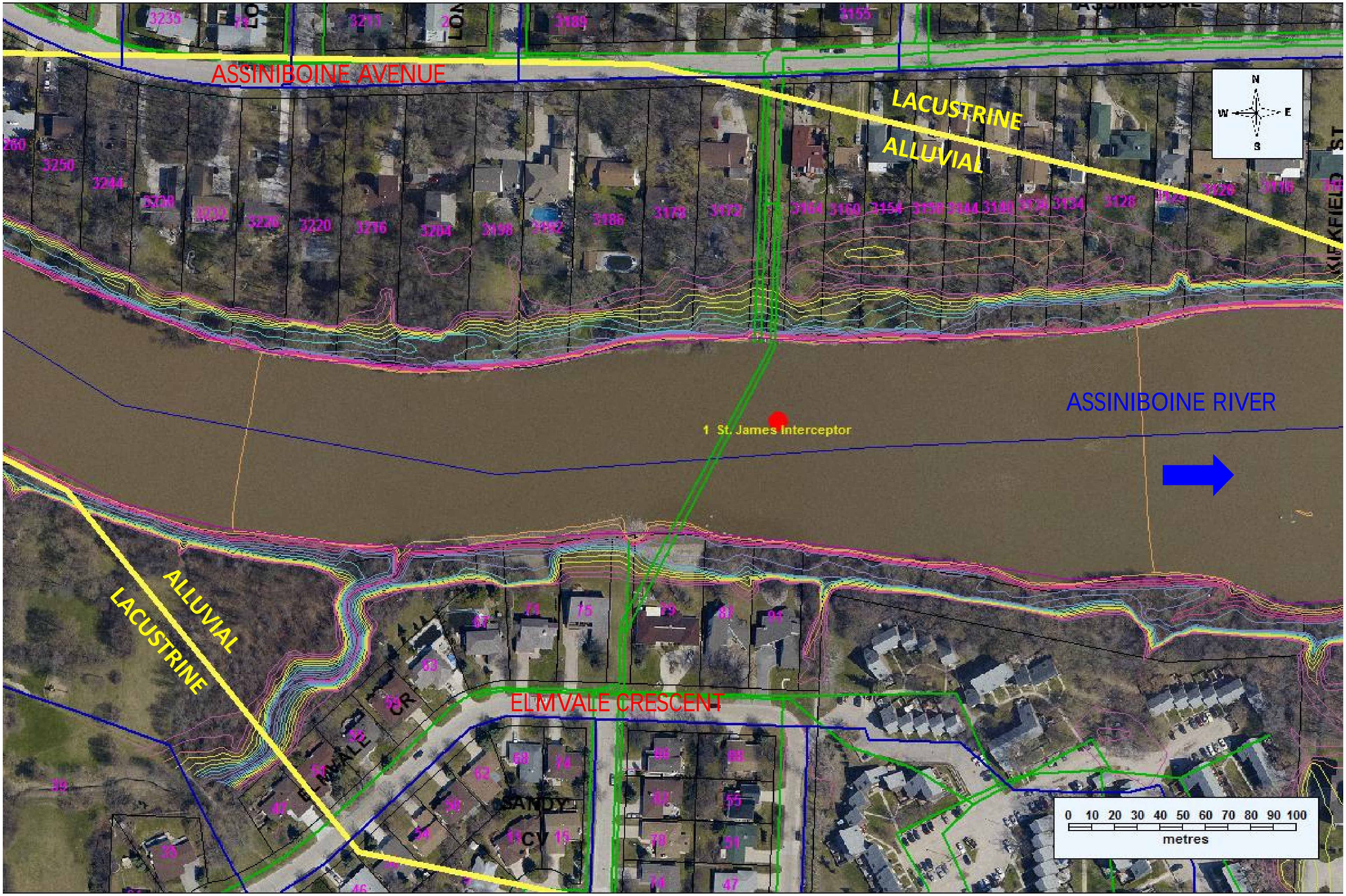
The river level in June obscured the lower bank and did not afford a view of the mechanism underlying the toppling failures. In August, and upon viewing a new section of bank toppling, it is apparent that the bank is being undermined by the river current through erosion processes. Once the overlying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

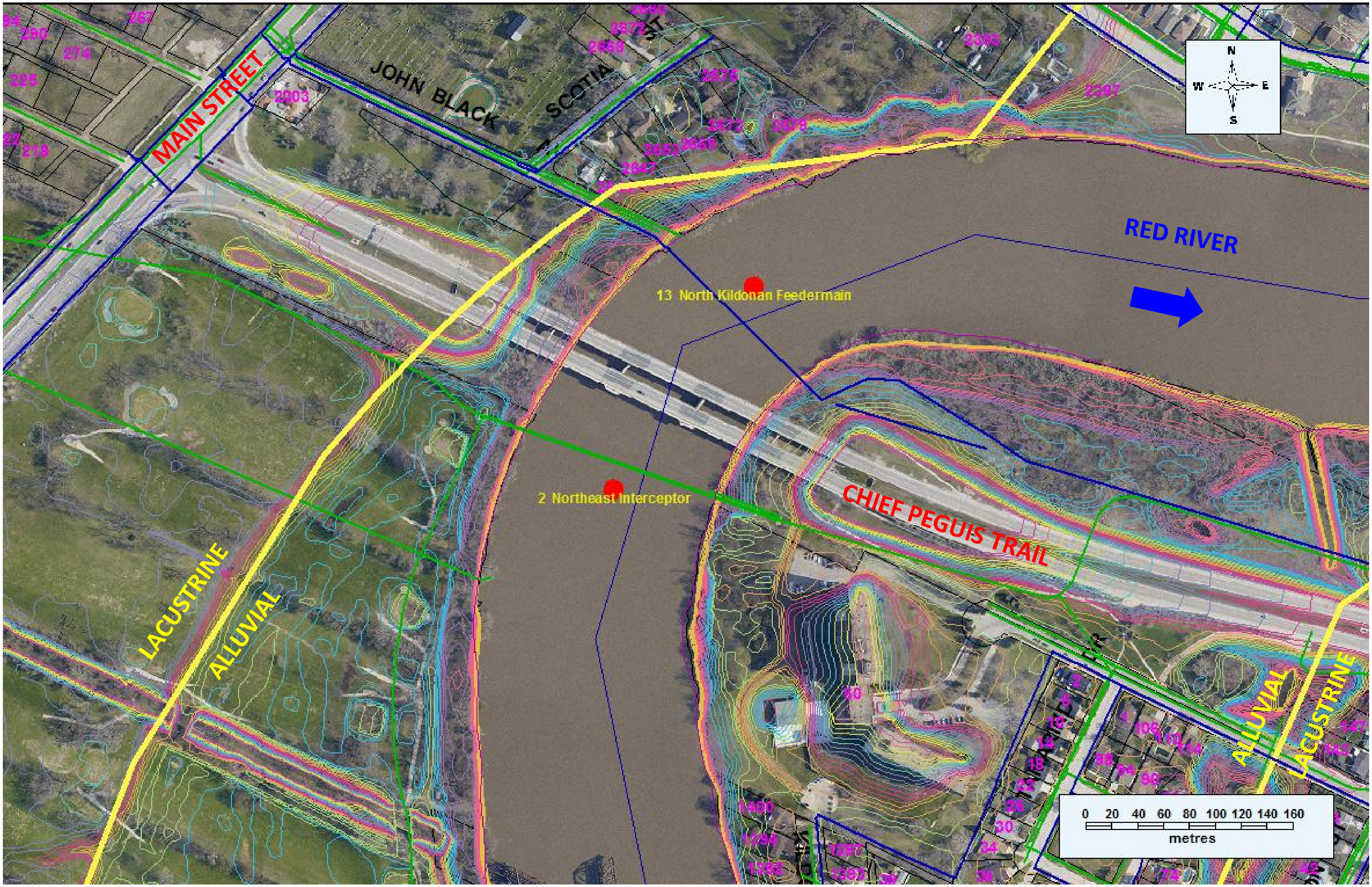
Figure 25 – Fresh slump block toppled between June 26 and August 15, 2012 indicative of active failure pattern.

APPENDIX A
SITE LOCATIONS

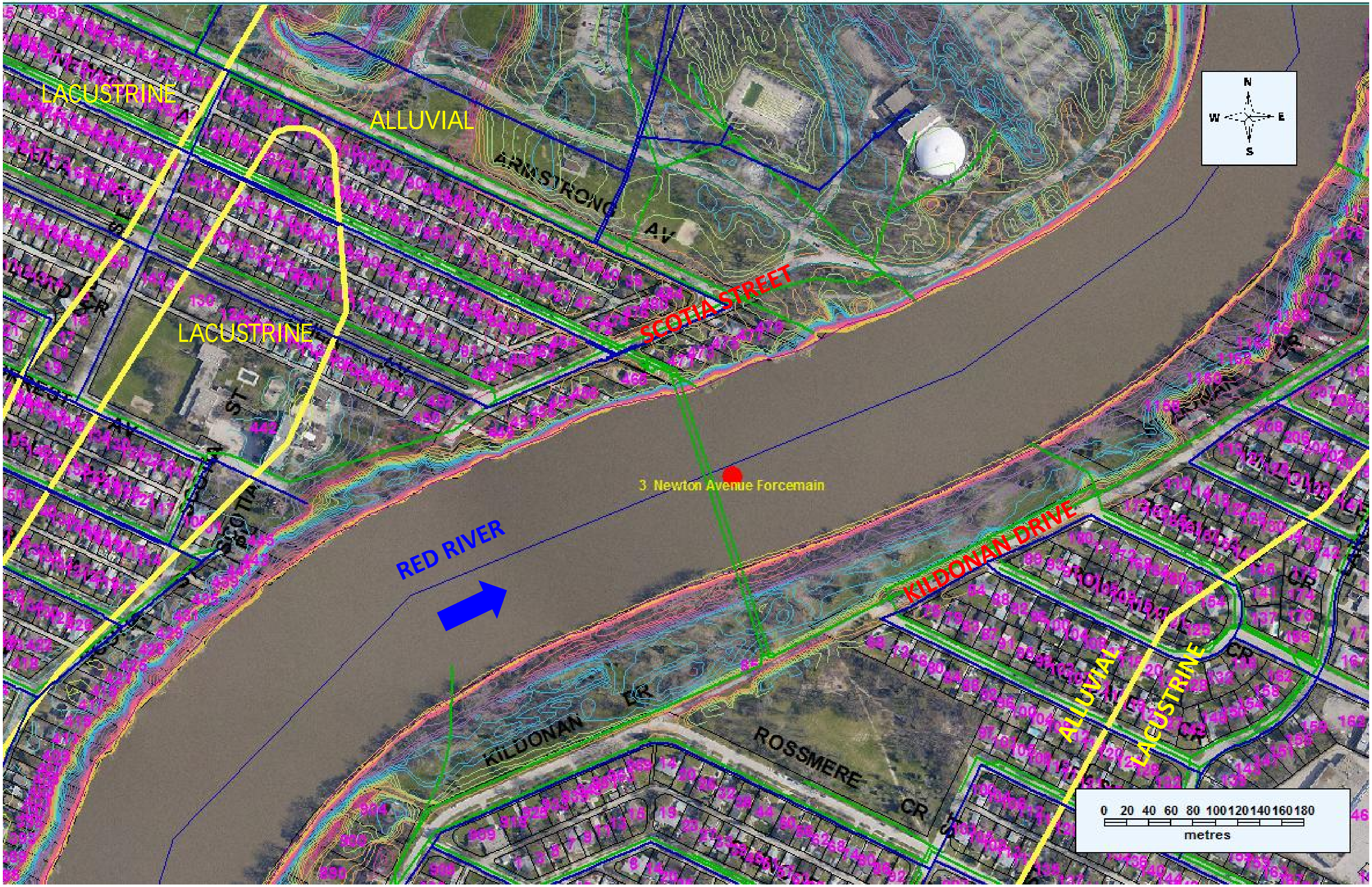

N
W
E
S
0
1000
2000
3000
4000
5000
metres
1
2
3
4
5
6
7
8
9
10
11
12
13
14



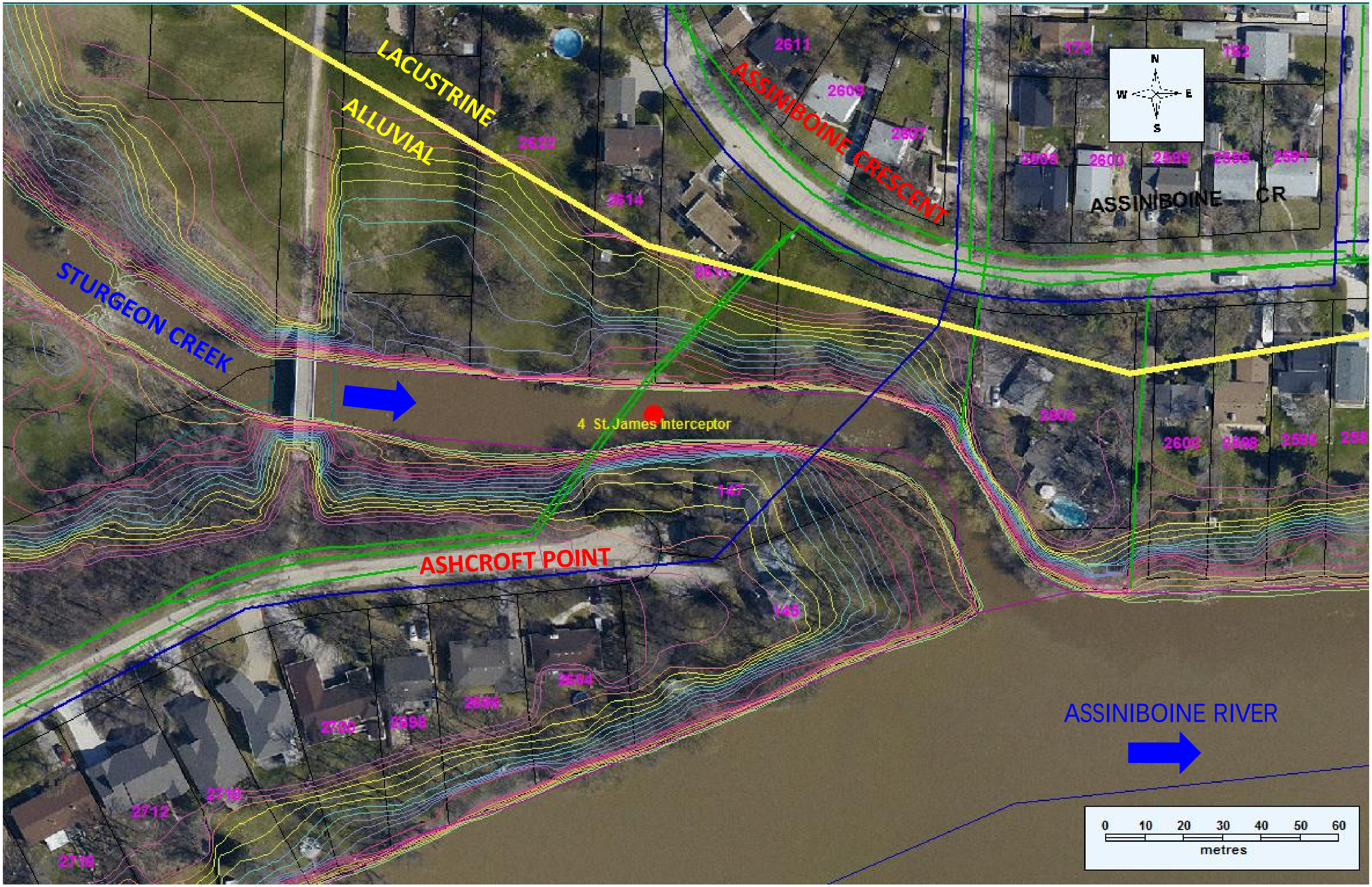
SITE 1: ST. JAMES INTERCEPTOR



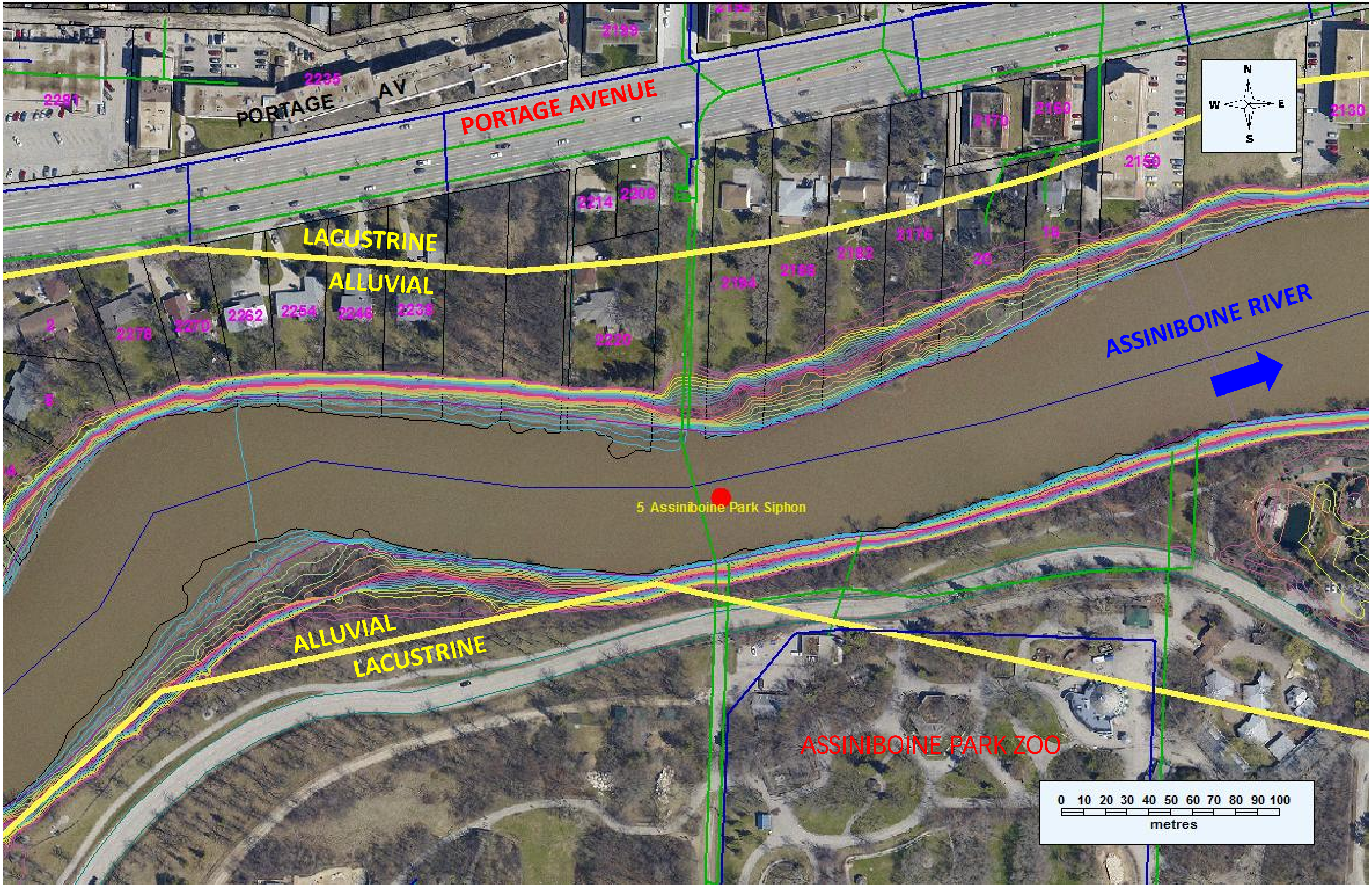
SITE 2: NORTHEAST INTERCEPTOR
SITE 13: NORTH KILDONAN FEEDERMAIN



SITE 3: NEWTON AVENUE FORCEMAIN

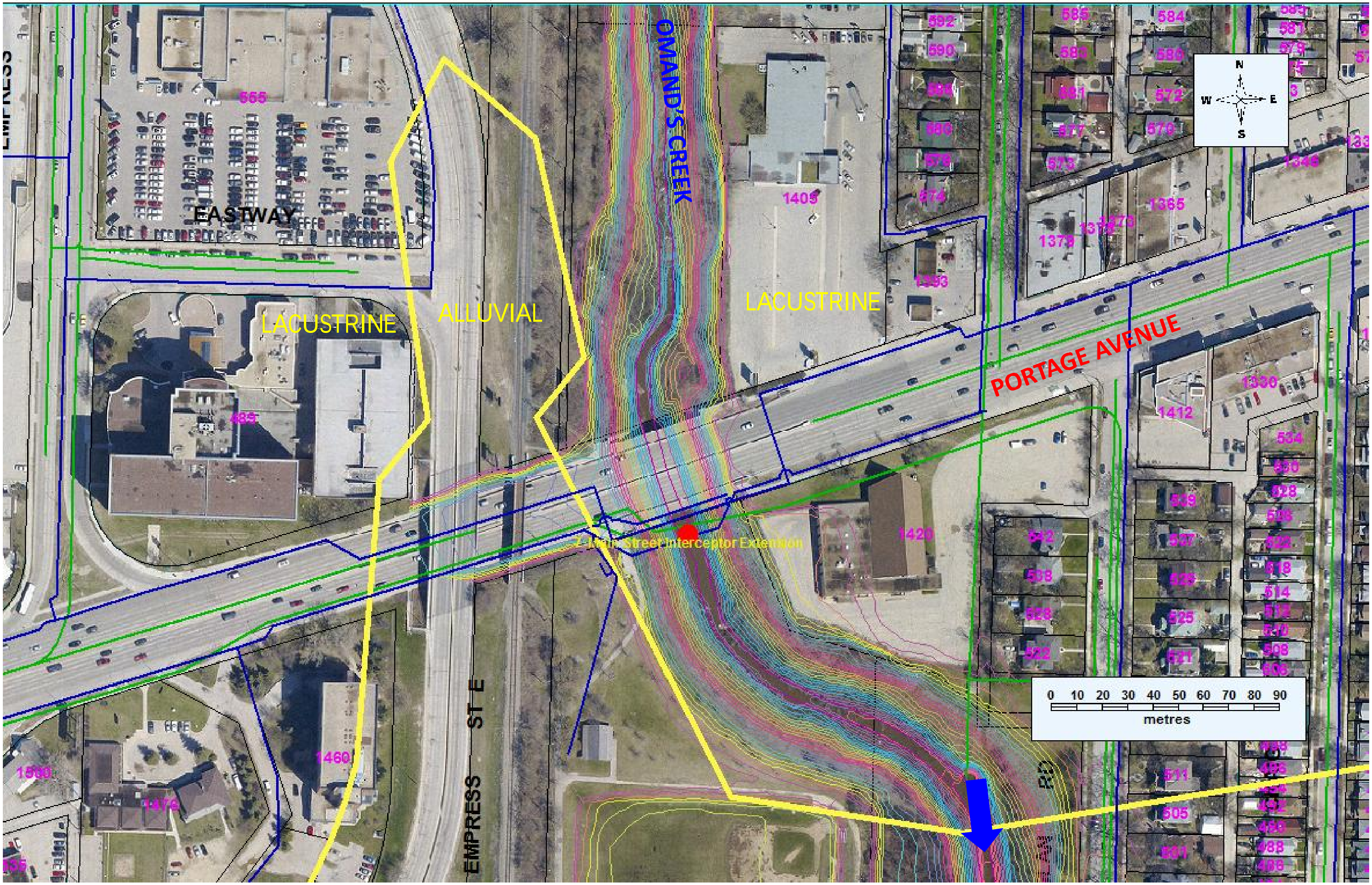


SITE 4: ST. JAMES INTERCEPTOR

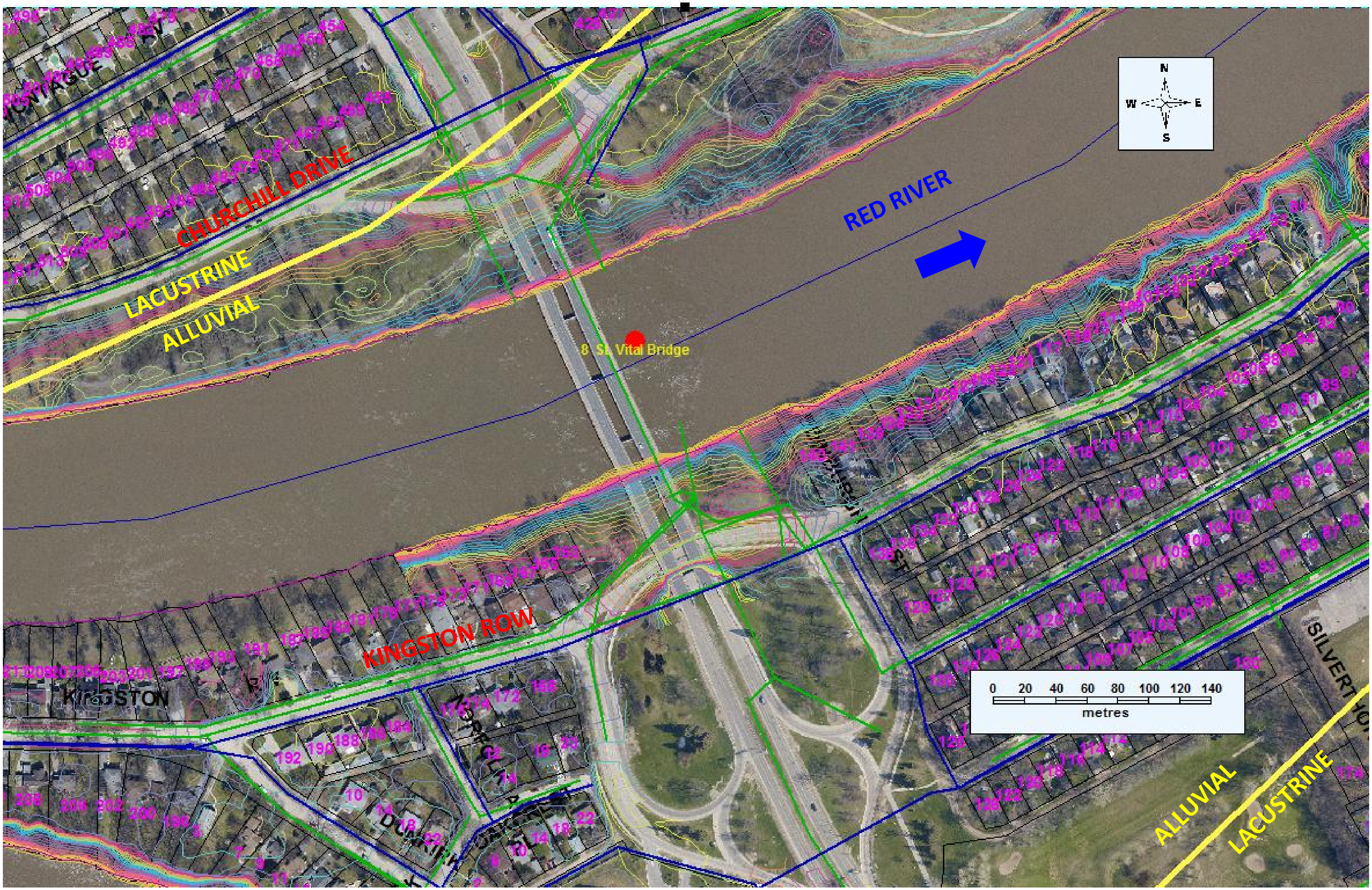


SITE 5: ASSINIBOINE PARK SIPHON

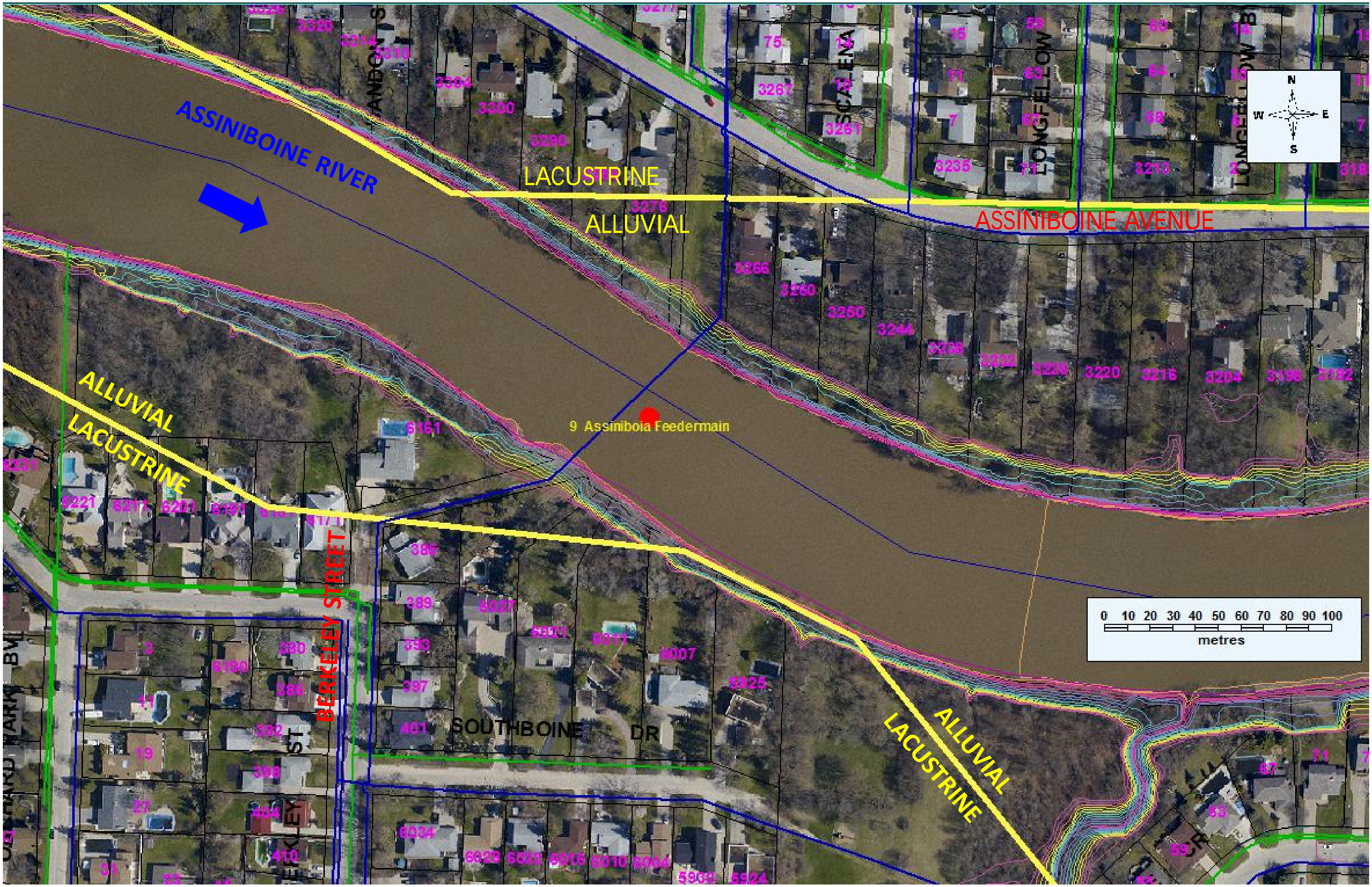
SITE 6: MUNROE POLSON SIPHON



SITE 7: MAIN STREET INTERCEPTOR EXTENSION

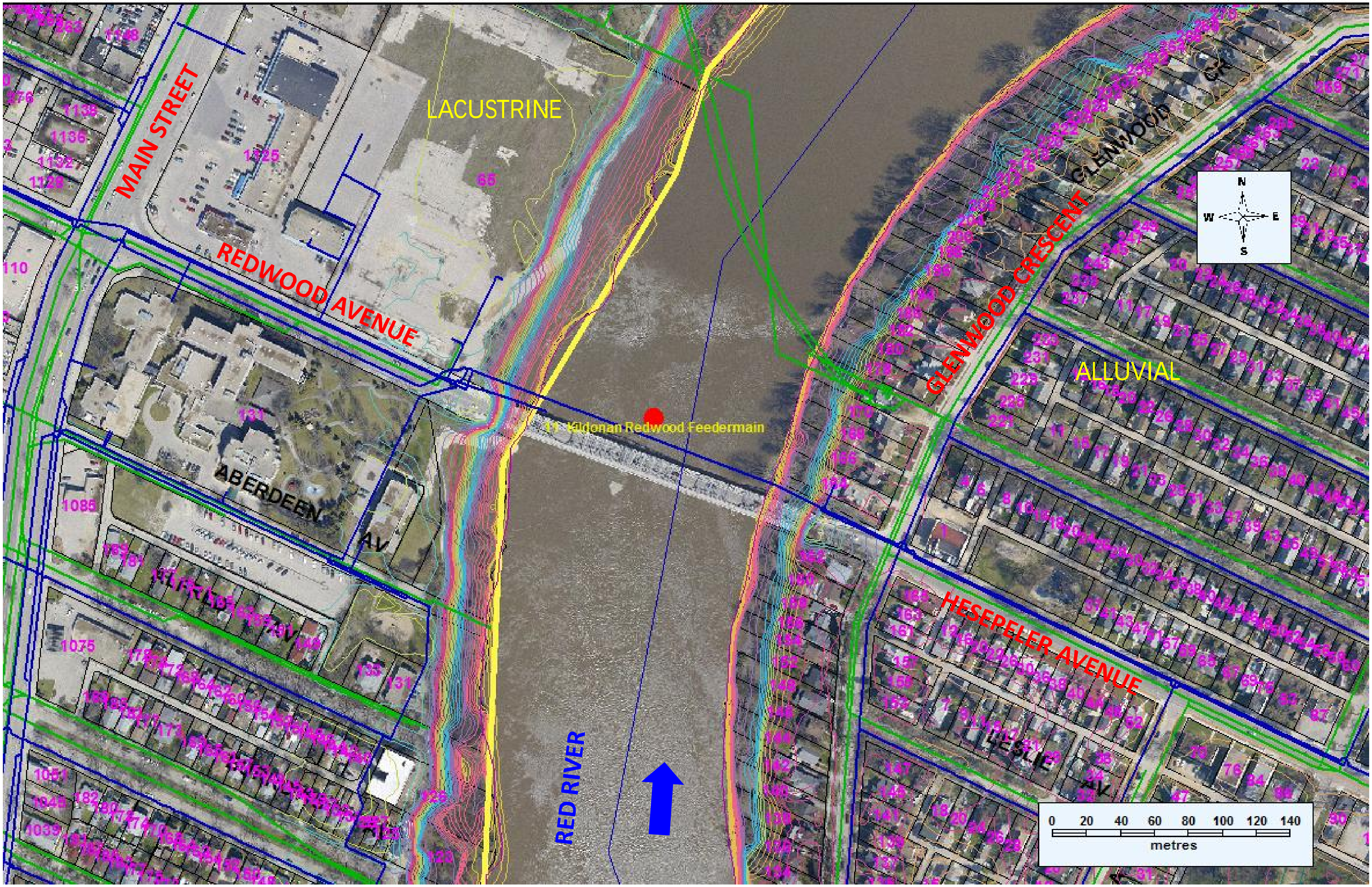


SITE 8: ST. VITAL BRIDGE

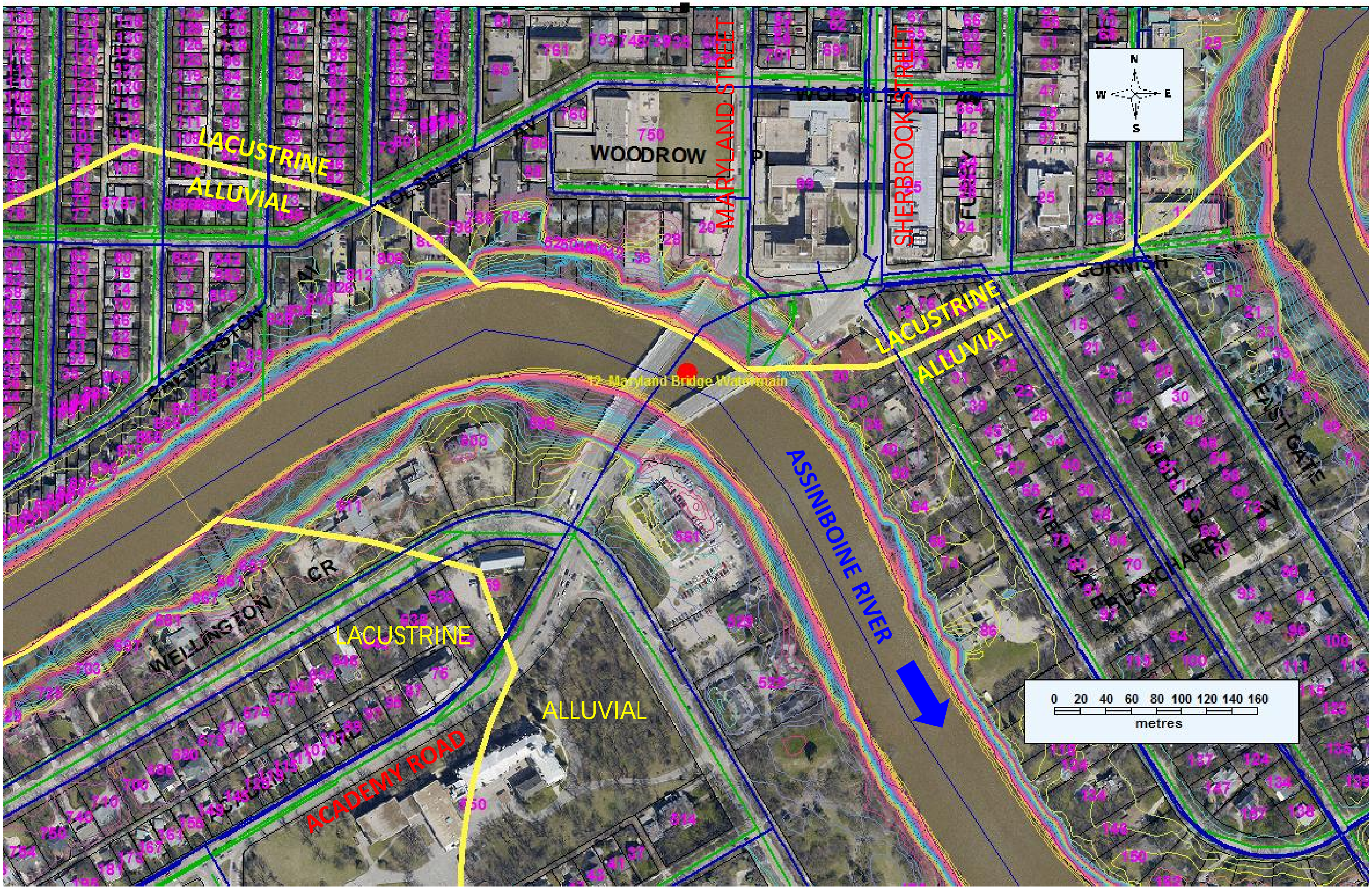


SITE 9: ASSINIBOIA FEEDERMAIN

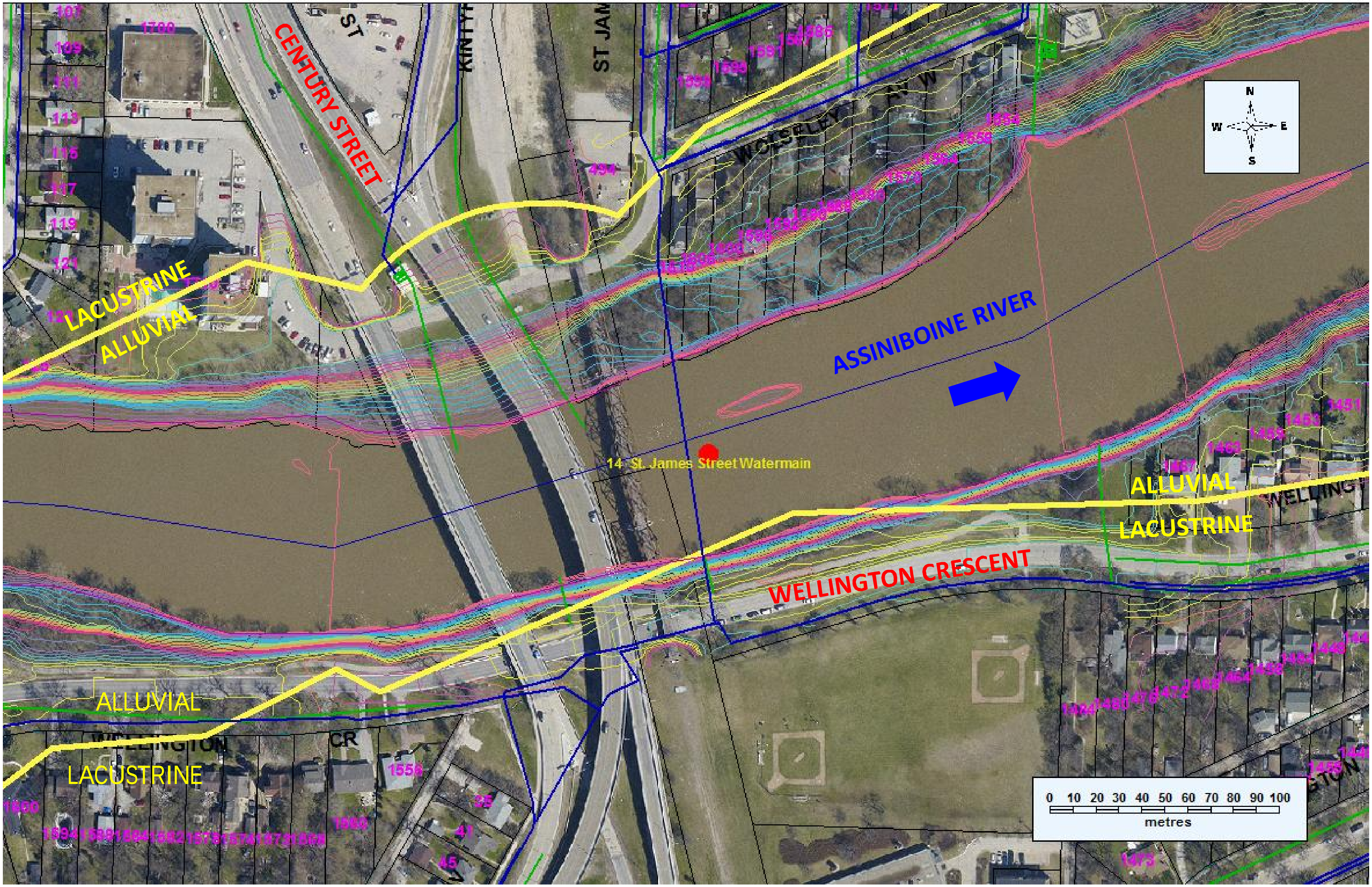
SITE 10: GOULET DOUCET WATERMAIN



SITE 11: KILDONAN REDWOOD FEEDERMAIN



SITE 12: MARYLAND BRIDGE WATERMAIN



SITE 14: ST. JAMES STREET WATERMAIN

APPENDIX B
RIVER BANK CONDITION SUMMARY

SUMMARY OF SITE INSPECTIONS

				FIRST PIPE			SECOND PIPE (IF DUAL PIPE CROSSING)					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)
CROSSING ID NUMBER	LOCATION	PIPE FUNCTION	RIVER	PIPE DIAMETER (mm)	PIPE MATERIAL	INSPECTION LENGTH (m)	PIPE DIAMETER (mm)	PIPE MATERIAL	INSPECTION LENGTH (m)	SIDE OF RIVER	NEIGHBOURING STREET	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	NOT EXIST	
1	St. James Interceptor	SIPHON	ASSINIBOINE	600	Steel	201	500	Steel	201	NORTH	ASSINIBOINE AVE	X				X		X		X		X		X		X				X		X	X
										SOUTH	ELMVALE CRES	X				X	X			X	X		X			X				X		X	X
2	Northeast Interceptor	SIPHON	RED	800	Steel	236	500	Steel	236	WEST	MAIN ST			X	X		X			X	X		X			X			X		X		XX
										EAST	WHELLAMS LANE	X			X		X			X		X	X		X			X		X	X		XX
3	Newton Avenue Forcemain	FORCEMAIN	RED	350	Steel	297	350	HDPE	297	WEST	SCOTIA ST	X				X		X		X		X		X	X		X			X		X	X
										EAST	FRASER'S GROVE	X				X		X		X		X	X			X				X		X	X
4	St. James Interceptor	FORCEMAIN	STURGEON	450	Steel	101	450	Steel	101	NORTH	ASSINIBOINE CRES			X		X	X			X		X				X				X	X		X
										SOUTH	ASHCROFT PT	X			X		X			X	X		X		X			X		X	X		XXX
5	Assiniboine Park Siphon	SIPHON	ASSINIBOINE	200	Steel	120				NORTH	PORTAGE AVE			X		X	X			X		X	X		X			X		X		X	X
										SOUTH	ASSINIBOINE PARK DR			X	X		X			X	X		X			X				X		X	XXX
6	Munroe Polson Siphon	SIPHON	RED	450	Steel	269	300	Steel	269	WEST	SCOTIA ST	X				X	X			X		X	X		X			X	X			X	X
										EAST	HENDERSON HWY		X			X		X		X		X		X	X		X			X		X	XXX
7	Main Street Interceptor Extension	SIPHON	OMANDS	600	Steel	64				WEST	PORTAGE AVE		X			X		X		X		X		X	X		X			X	X		X
										EAST	PORTAGE AVE		X			X		X		X		X		X	X		X			X	X		X
8	St. Vital Bridge	FORCEMAIN	RED	500	Steel	202				NORTH	CHURCHILL DR	X				X	X			X		X		X	X		X			X	X		X
										SOUTH	KINGSTON ROW	X				X	X		X		X		X		X			X	X		X		XXX
9	Assiniboia Feedermain	FEEDERMAIN	ASSINIBOINE	600	Steel	185				NORTH	ASSINIBOINE AVE			X		X	X			X		X		X		X				X		X	X
										SOUTH	SOUTHBOINE DR			X	X			X		X		X	X			X				X		X	X
10	Goulet Doucet Watermain	WATERMAIN	SEINE	400	Steel	120				WEST	GOULET ST			X		X		X		X		X	X			X				X		X	X
										EAST	DOUCET ST			X		X		X		X		X		X		X				X		X	X
11	Kildonan Redwood Feedermain	FEEDERMAIN	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		XXX
12	Maryland Bridge Watermain	WATERMAIN	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		XX
13	North Kildonan Feedermain	FEEDERMAIN	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		XX
14	St. James Street Watermain	WATERMAIN	ASSINIBOINE	450	VCI	185				NORTH	WOLSELEY AVE W	X			X		X			X		X	X			X				X	X		XX
										SOUTH	WELLINGTON CRES		X		X		X			X	X		X			X				X	X		XXX

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 porperties 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 alignmentto 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 gulley	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	Potenial slope inclinometer casings	June 23, 2012
70, 71	2	Northeast Interceptor	W	S	Cracks on abutment apron	June 23, 2012

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

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

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 propery 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 esat 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

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 uptream	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 groutted rip rap	June 27, 2012
440	6	Munroe Polson Siphon	W	N	Recession of river bank at groutted 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

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

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

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

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

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	Gulley 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	Gulley 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

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 gulley 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 wast 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

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 wiht 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 snd 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

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

AECOM
99 Commerce Drive
Winnipeg, MB, Canada R3P 0Y7
www.aecom.com

204 477 5381 tel
204 284 2040 fax

Technical Memorandum

To Marv McDonald

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.

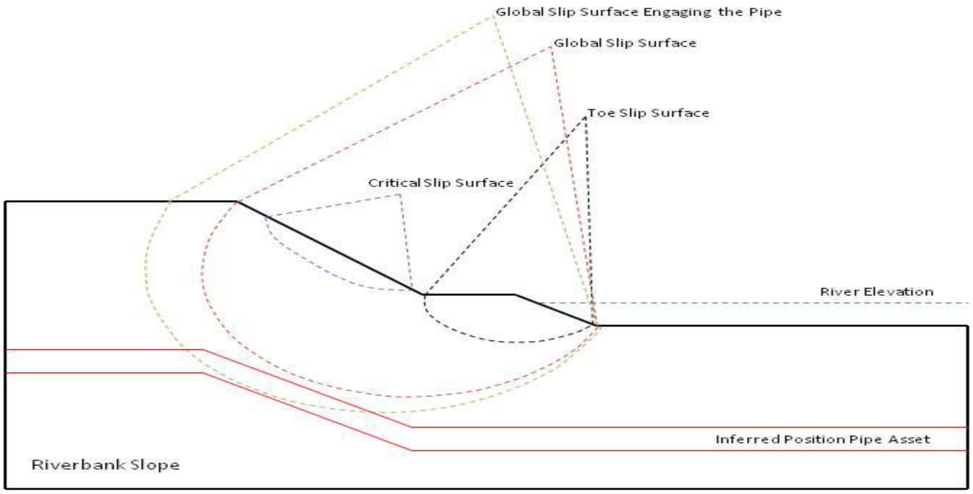


Figure 1: Assessed Slip Surfaces within Analysis

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.

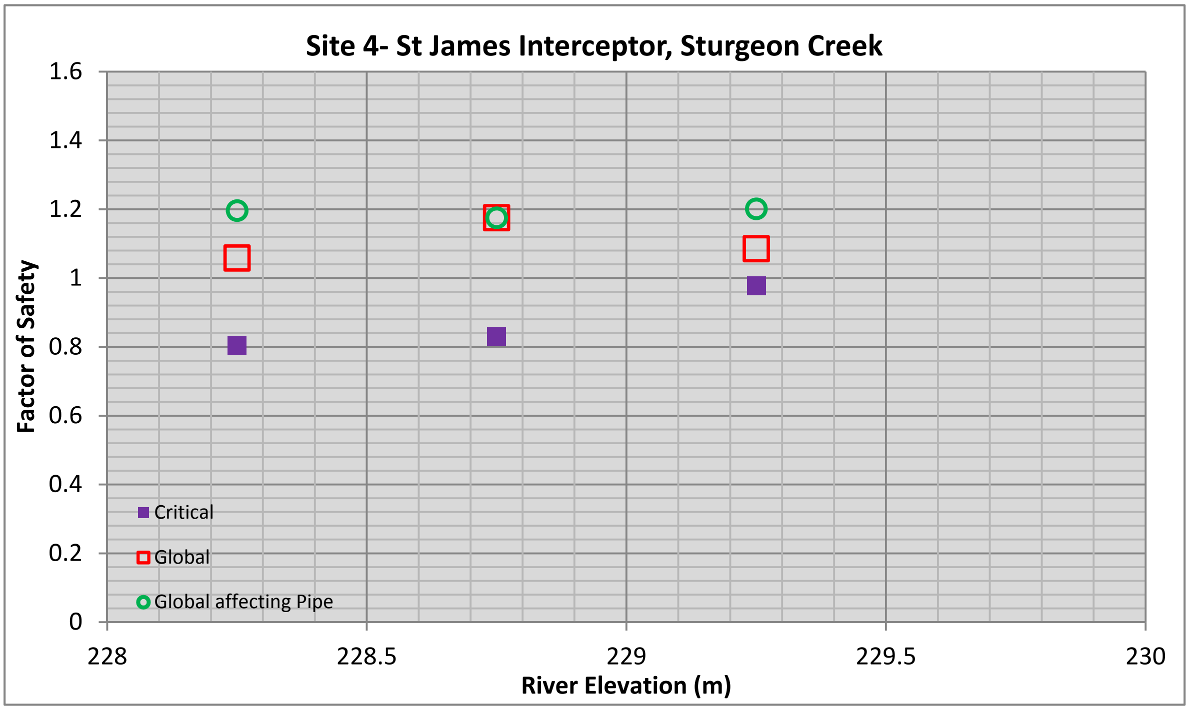


Figure 2: Factor of Safety relating to the Current Condition of Site 4

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.

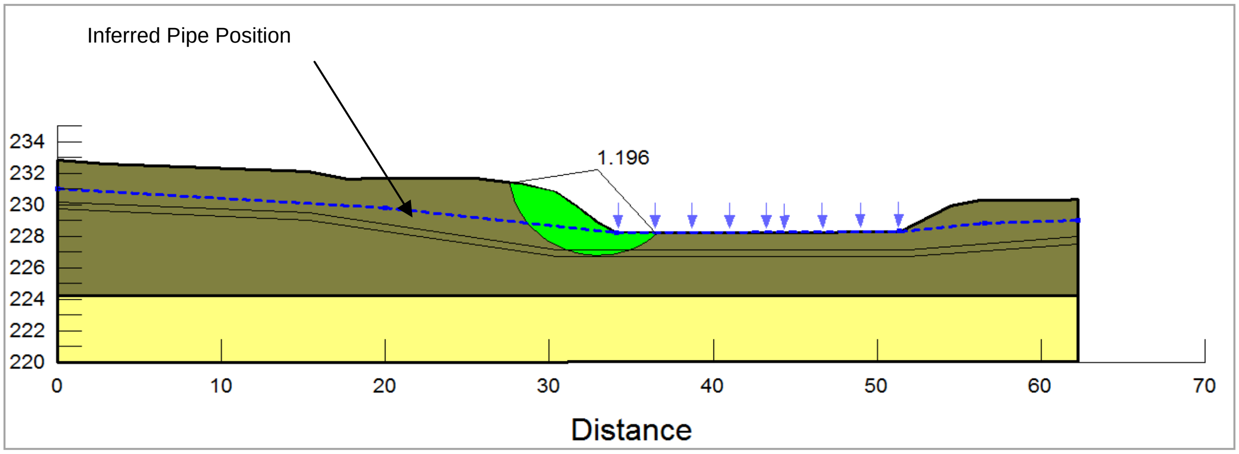


Figure 3: Site 4 St James Interceptor Global Slip Surface Engaging the Pipe (GS+P)

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.

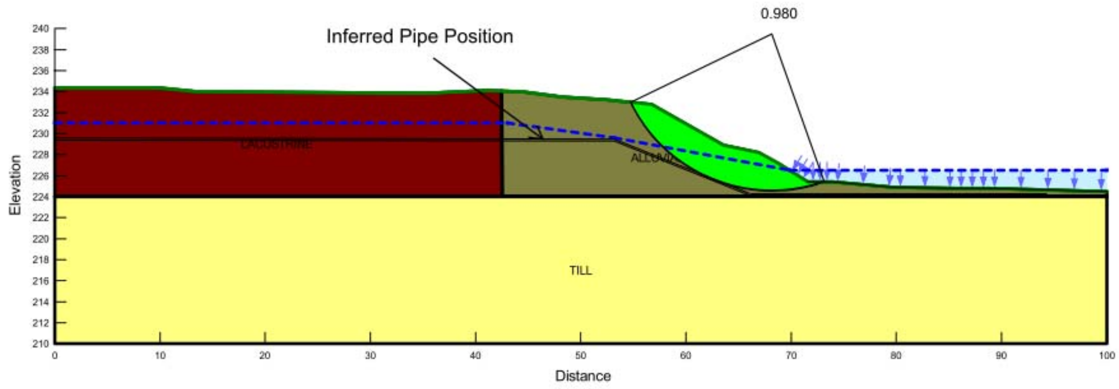


Figure 5: Site 5 Global Slip Surface Engaging the Pipe (GS+P)

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.

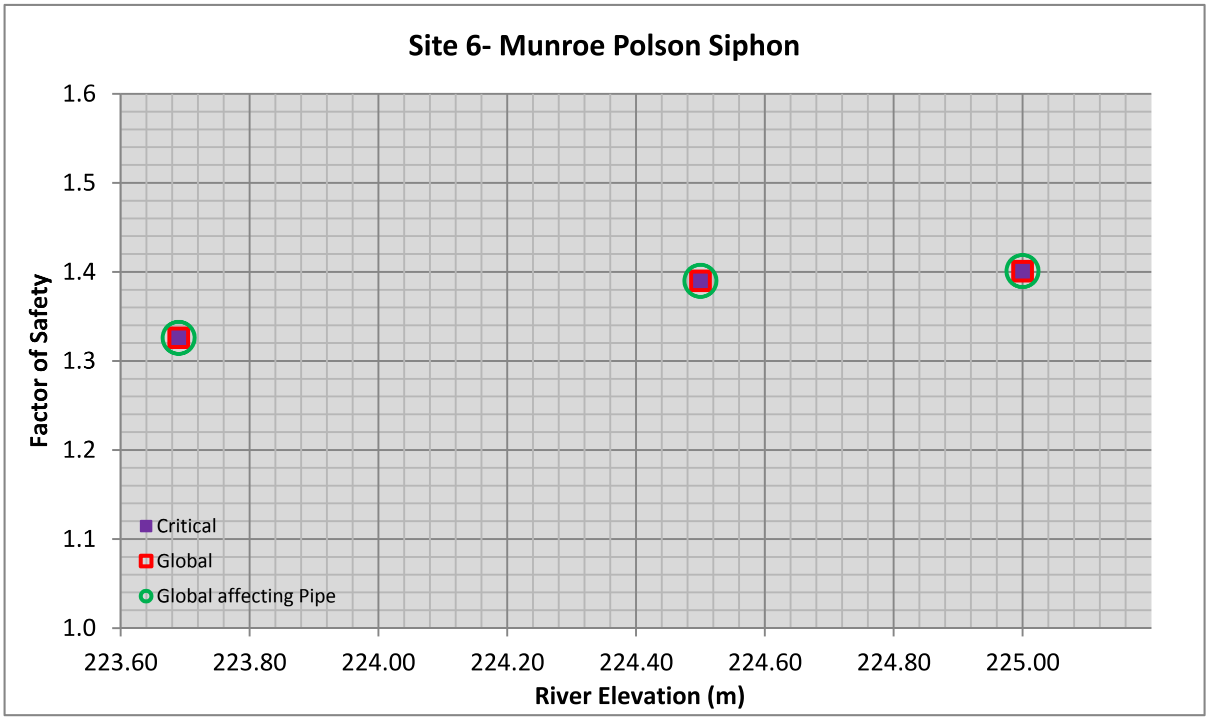


Figure 6: Slope Stability Factor of Safety Relating to the Current Condition to Site 5

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.

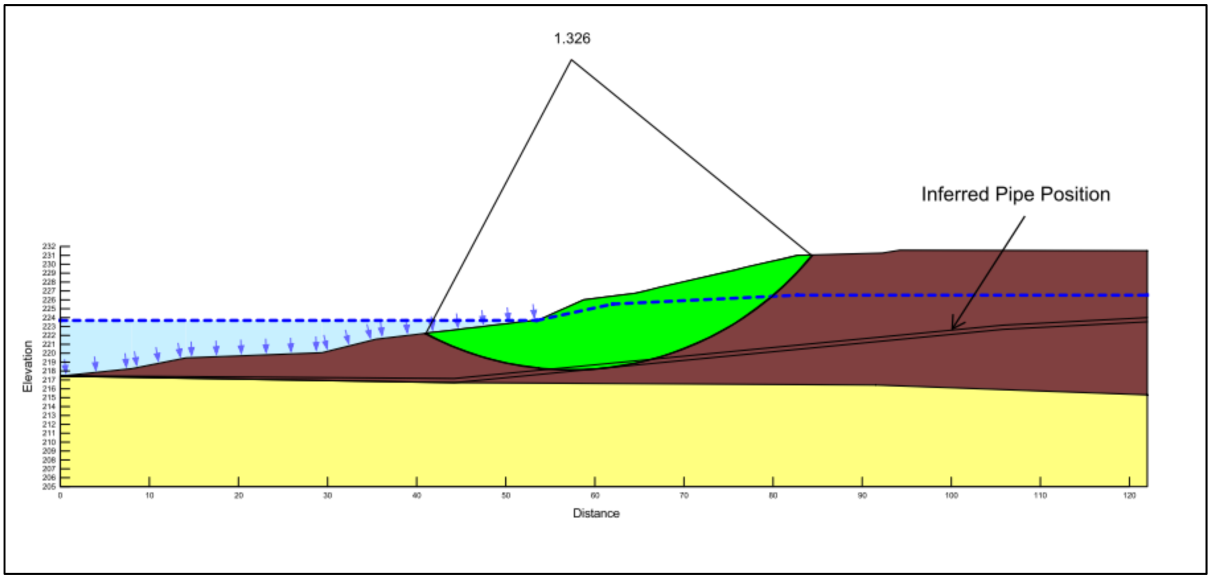


Figure 7: Munroe Polson Siphon- Global Slip Surface Engaging Pipe (GS+P)

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 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.

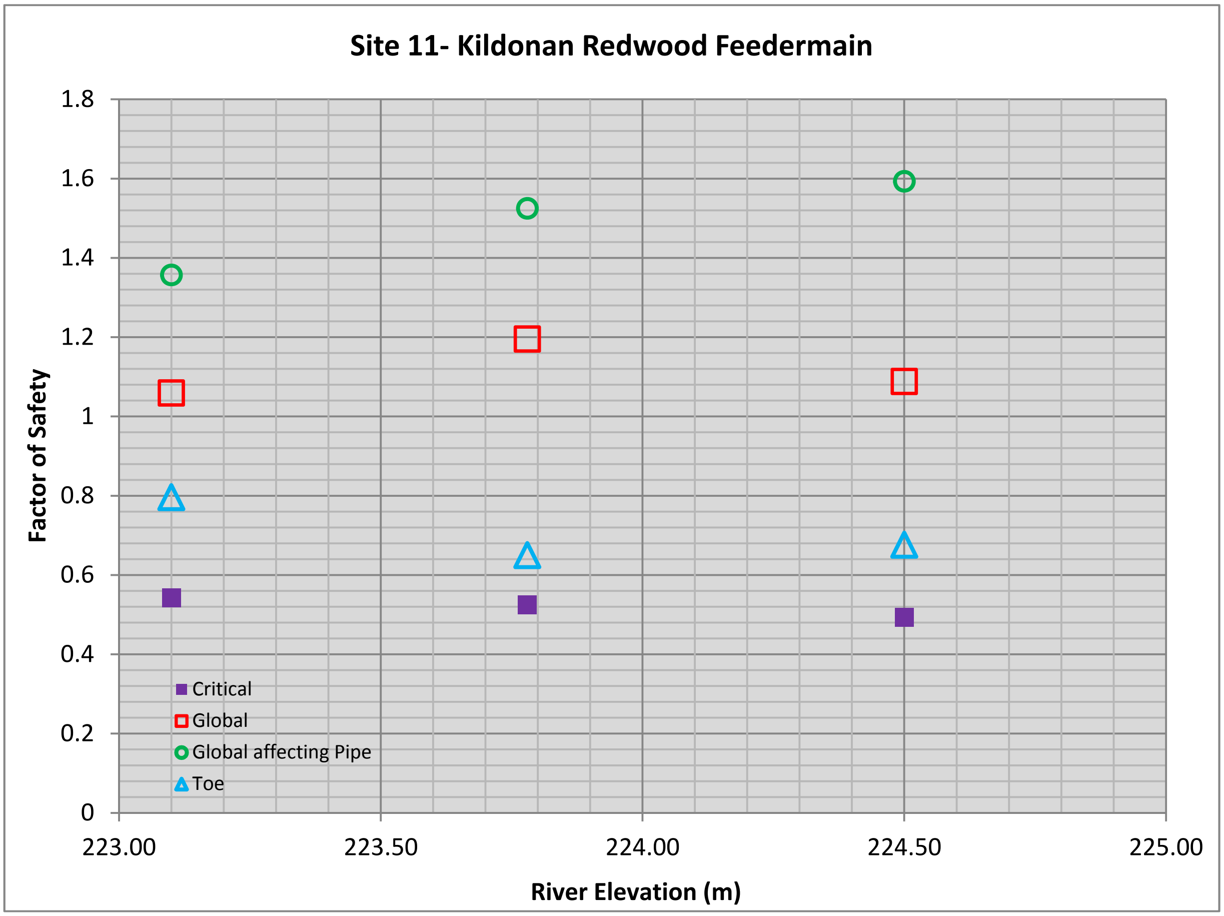


Figure 8: Slope Stability Factor of Safety Relating to the Current Conditions of Site 11

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.

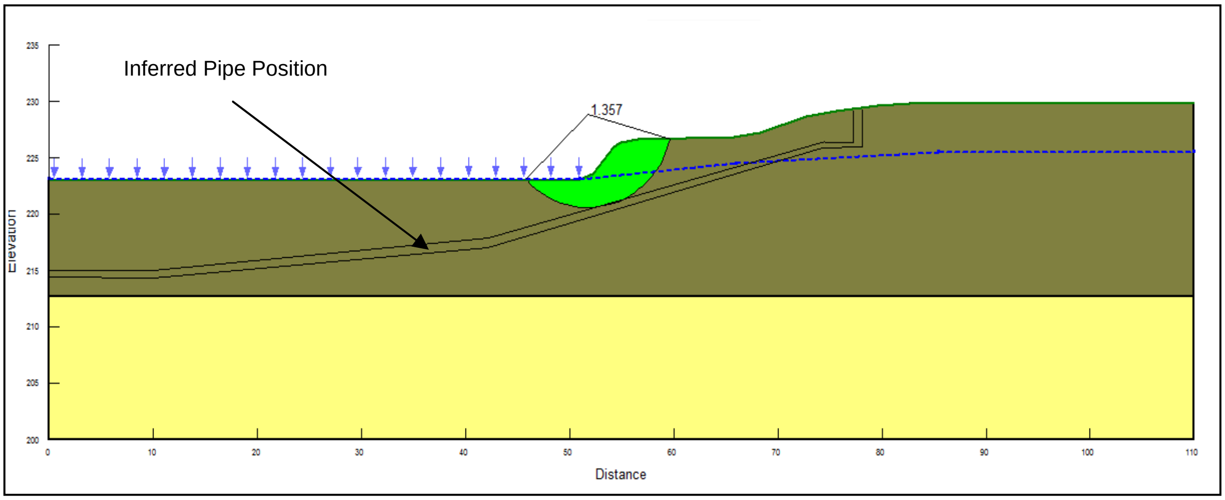


Figure 9: Site 11, Global Slip Surface Engaging the Pipe (GS+P)

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.

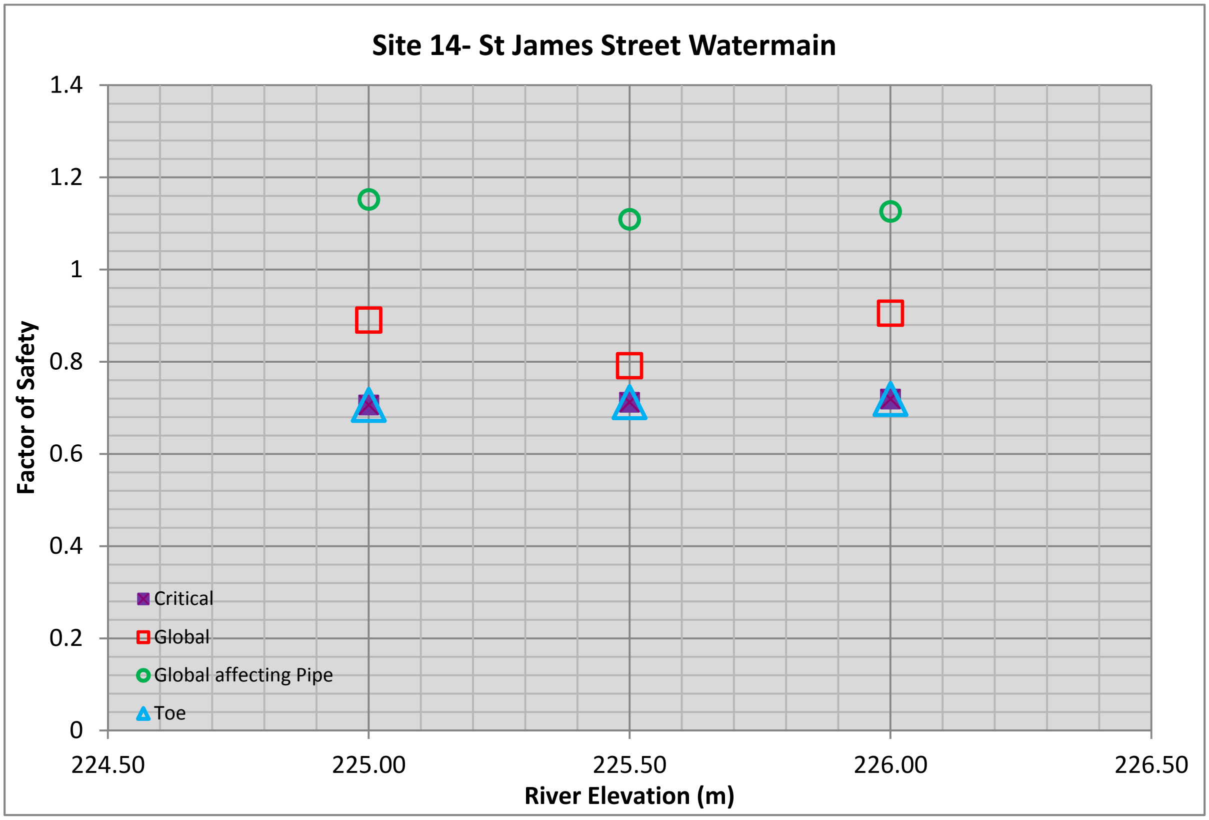


Figure 10: Slope Stability Factor of Safety Relating to the Current Conditions of Site 14

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.

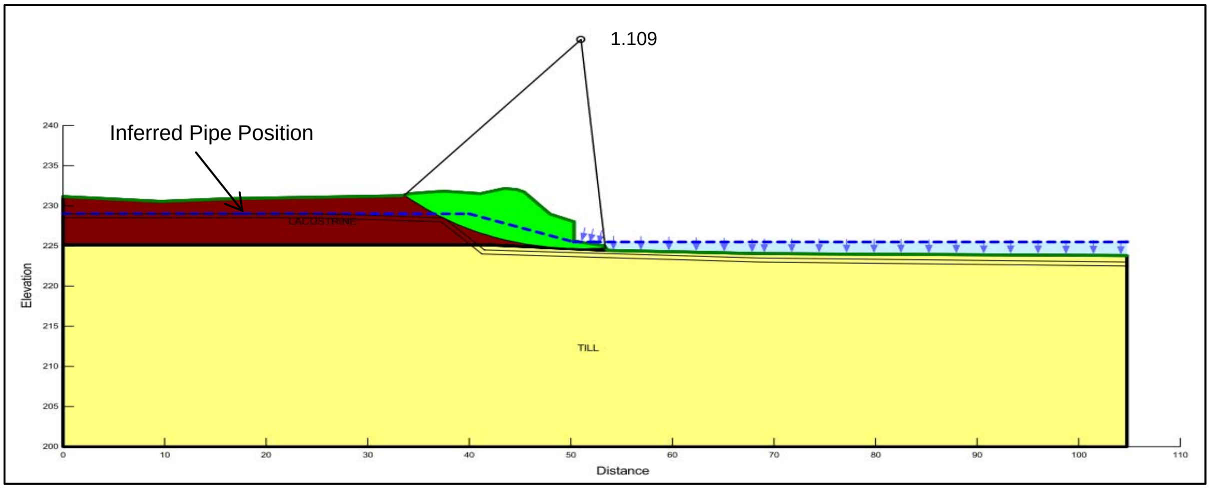


Figure 11: Site 14, Global Slip Surface Engaging the Pipe (GS+P)

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.

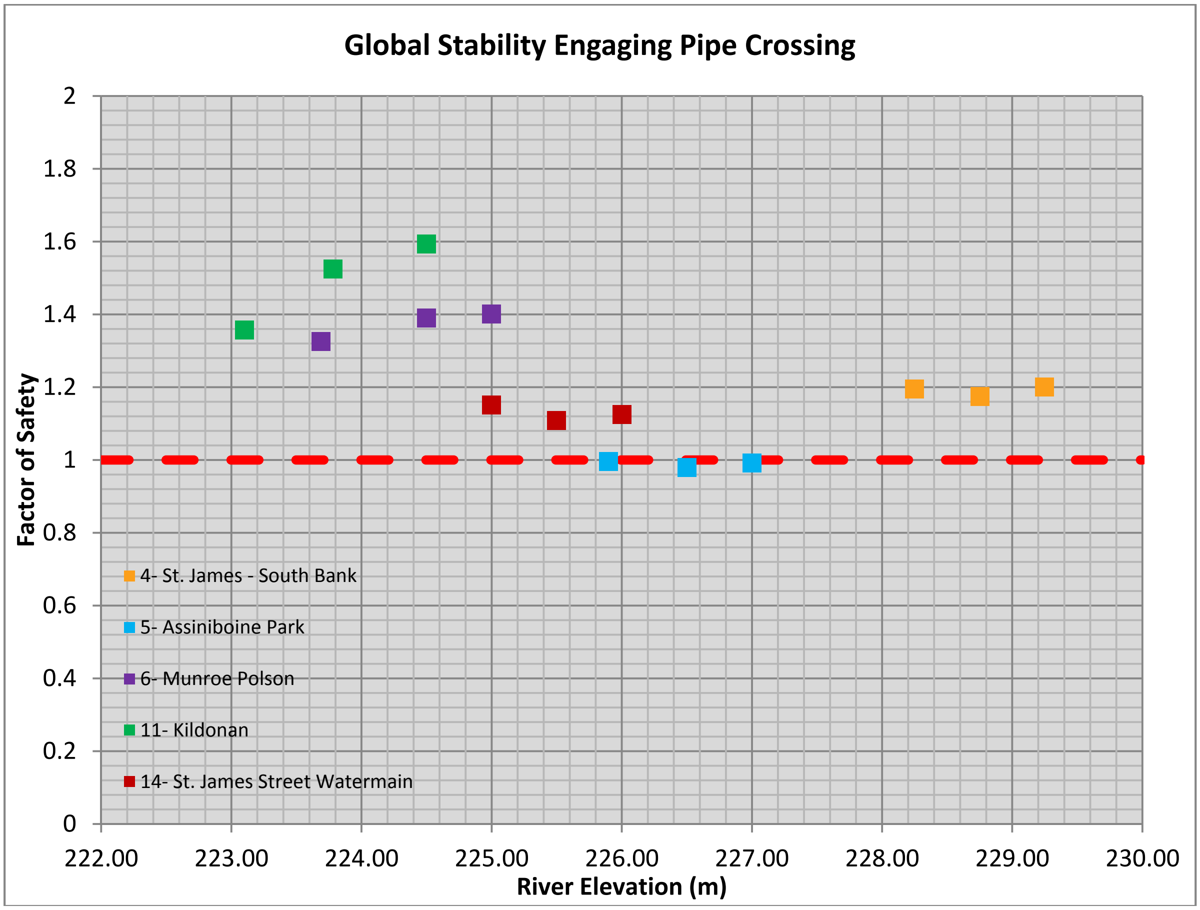


Figure 12: Global Stability Engaging Pipe Crossing

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)	ø	C (kPa)
Intact Alluvial	17	26	0
Residual Alluvial	-	-	-
Intact Lacustrine	17	17	5
Residual Lacustrine	-	-	-
Till	21	30	10

Notes

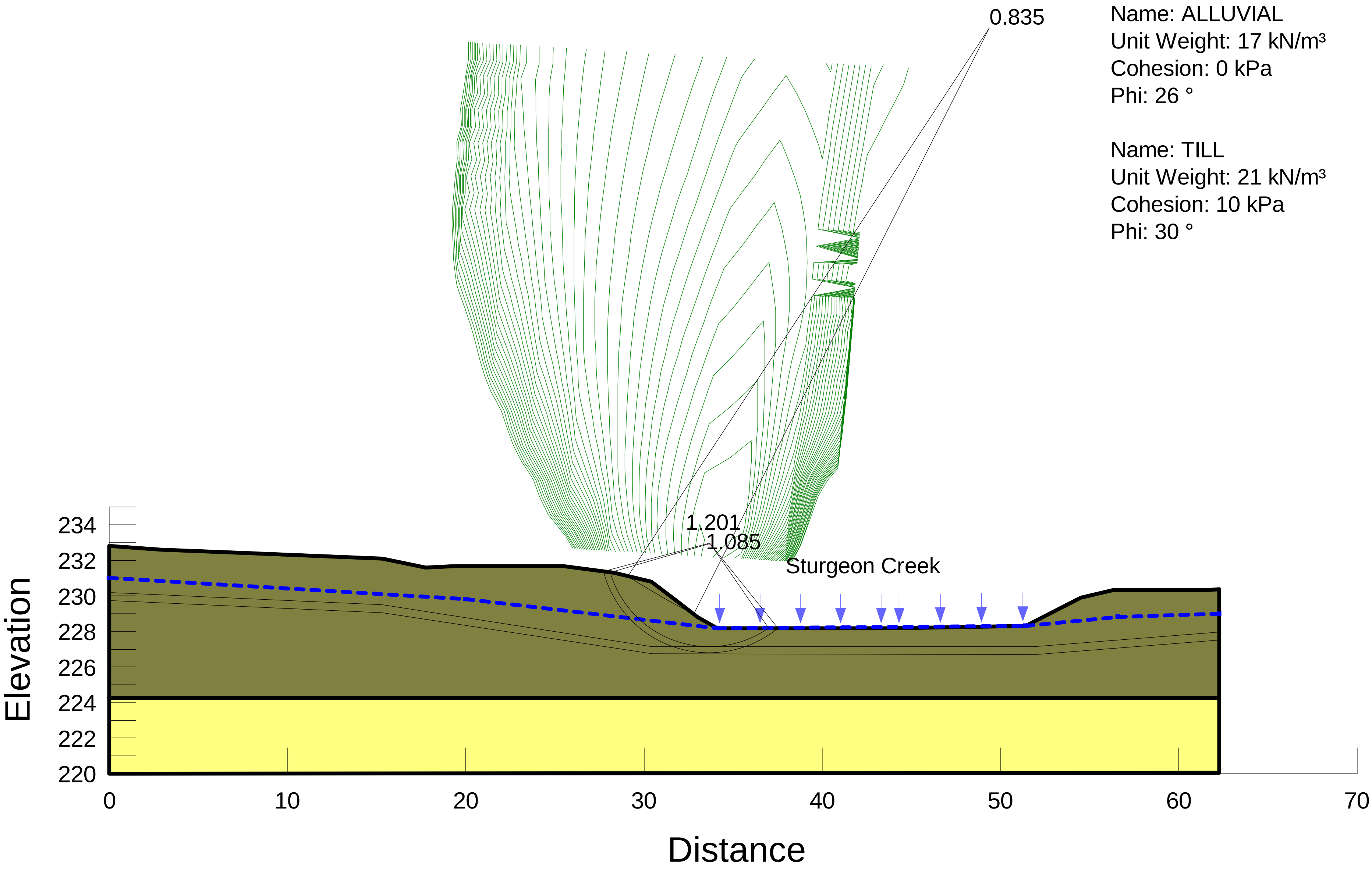
√ Indicates specific application of geometric model

Stability Results

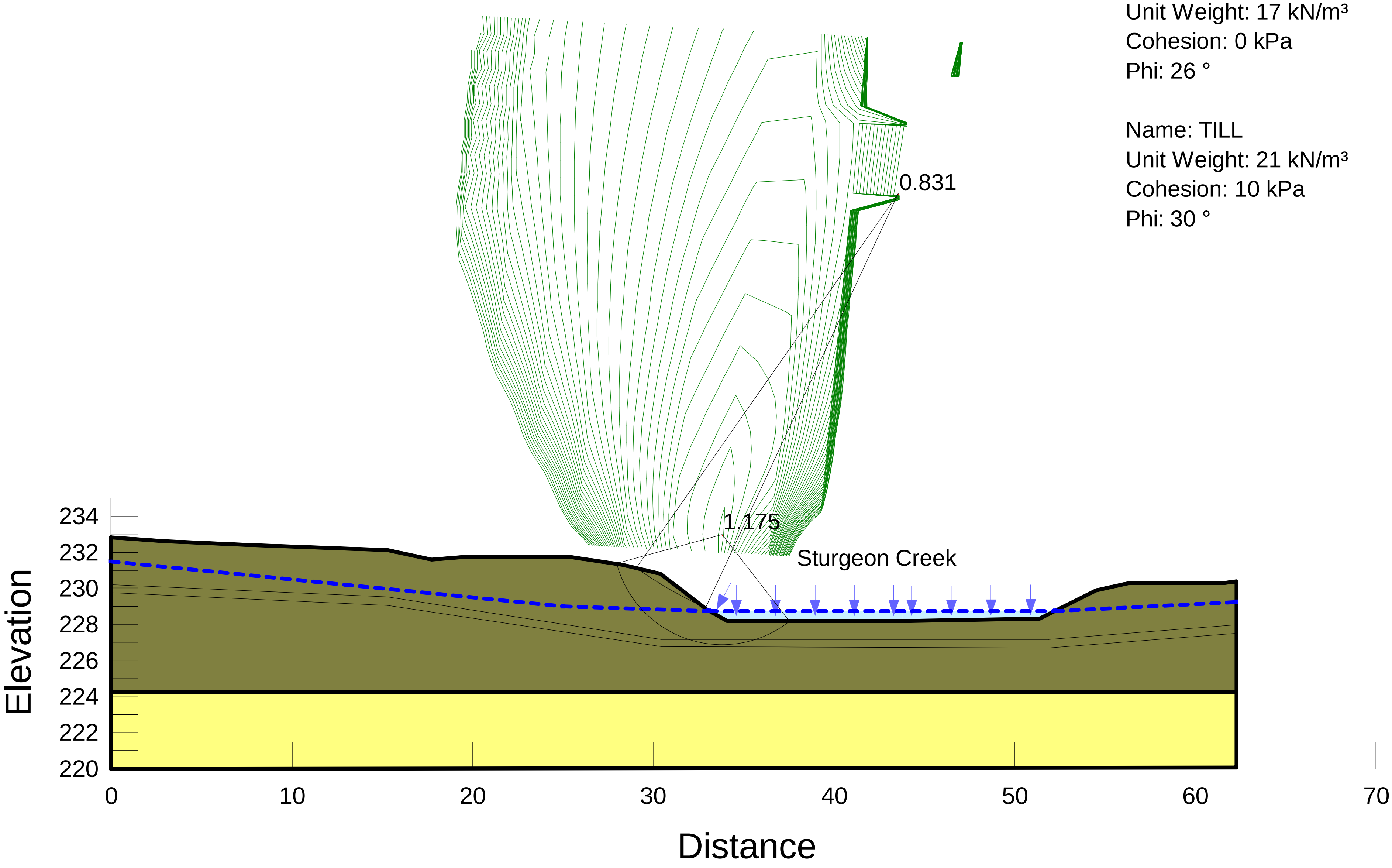
Stability Case	Till Head (m)	River Level (m)	SCARP PRESENT ON ALIGNMENT	RIP RAP AT RIVER BANK TOE	LOWER BANK INSTABILITIES EVIDENT	Pipe Layout	Geometry Cross Section			Grid-Radius								File Name
							As-Built Drawings	Lidar Survey	Bathymetric Survey	Critical	Slip #	Global	Slip #	Global affecting 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



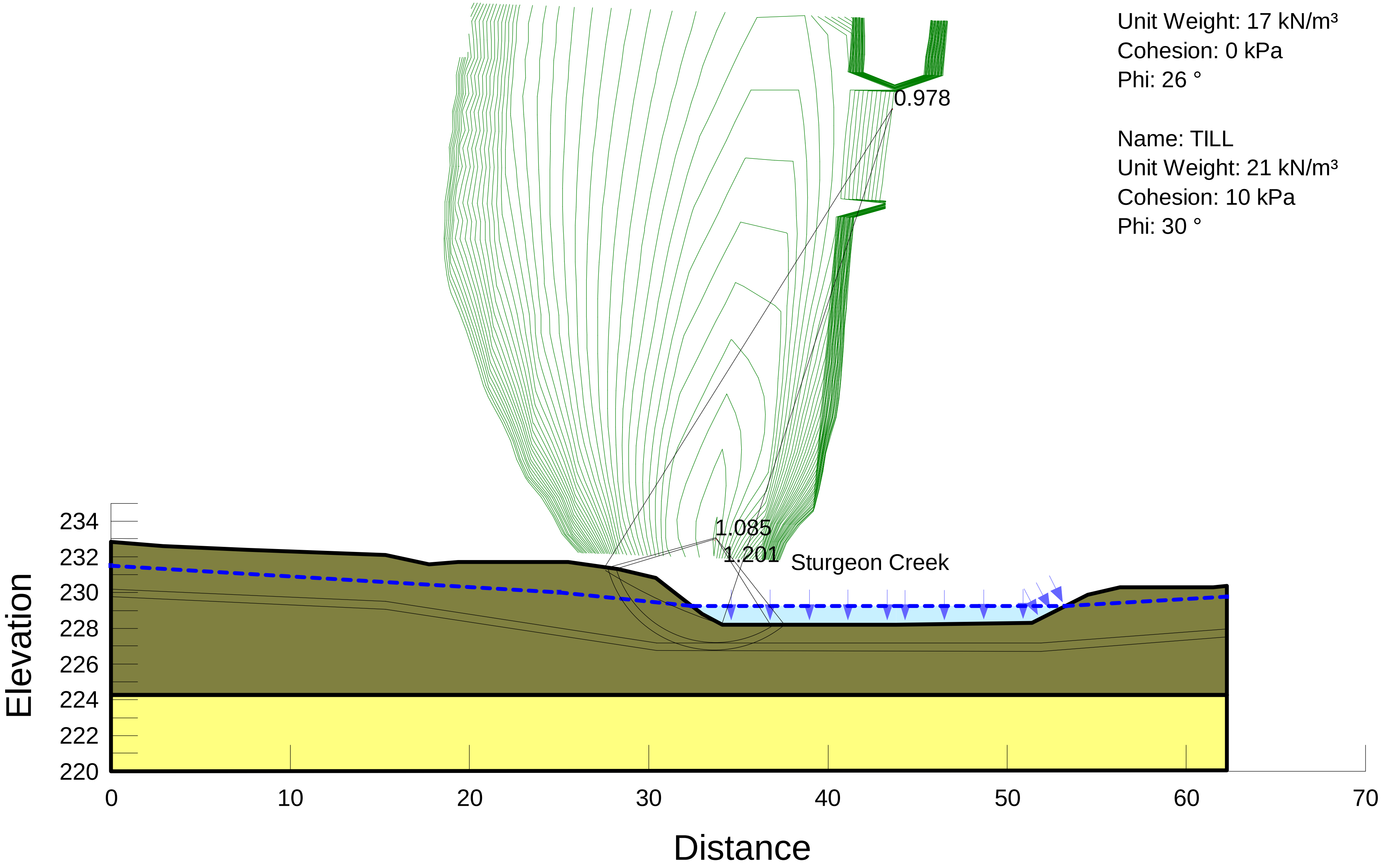
Name: ALLUVIAL
Unit Weight: 17 kN/m³
Cohesion: 0 kPa
Phi: 26 °

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

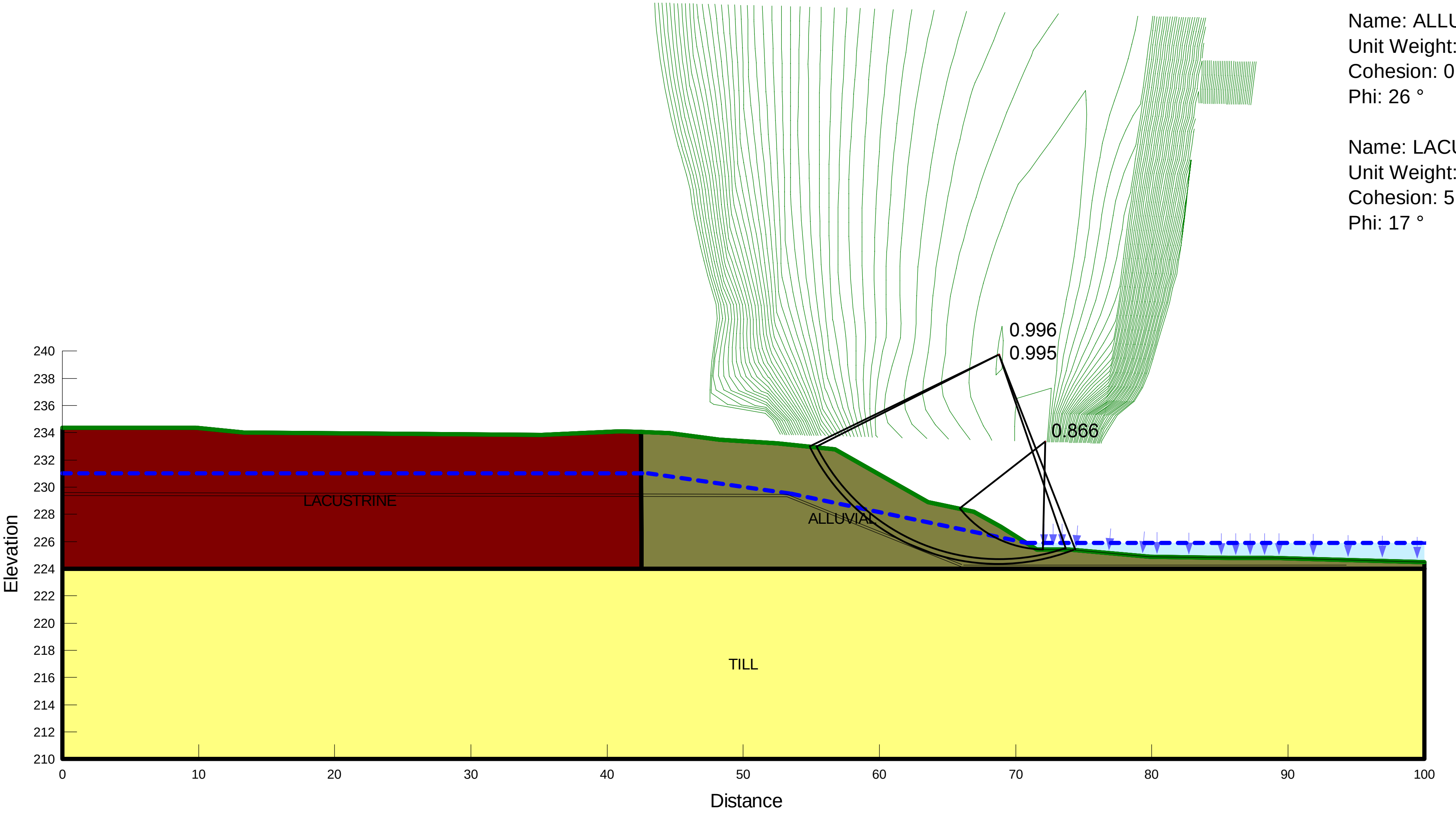
Title: Site 5- Assiniboine Park Siphon

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 °

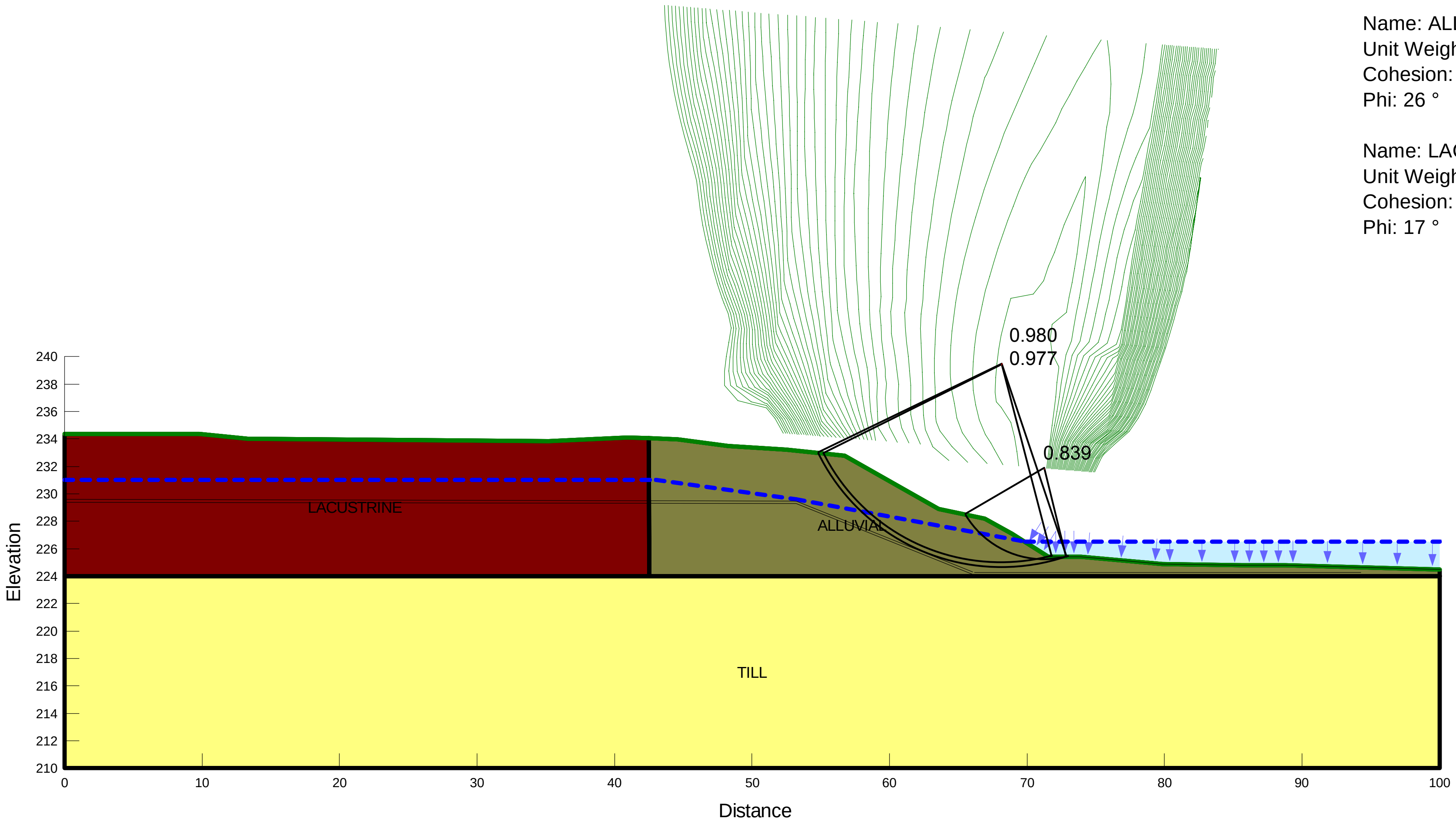


Condition Assessment of High Risk Water and Wastewater River Crossings
Title: Site 5- Assiniboine Park Siphon
Name: Case 5-2

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 °

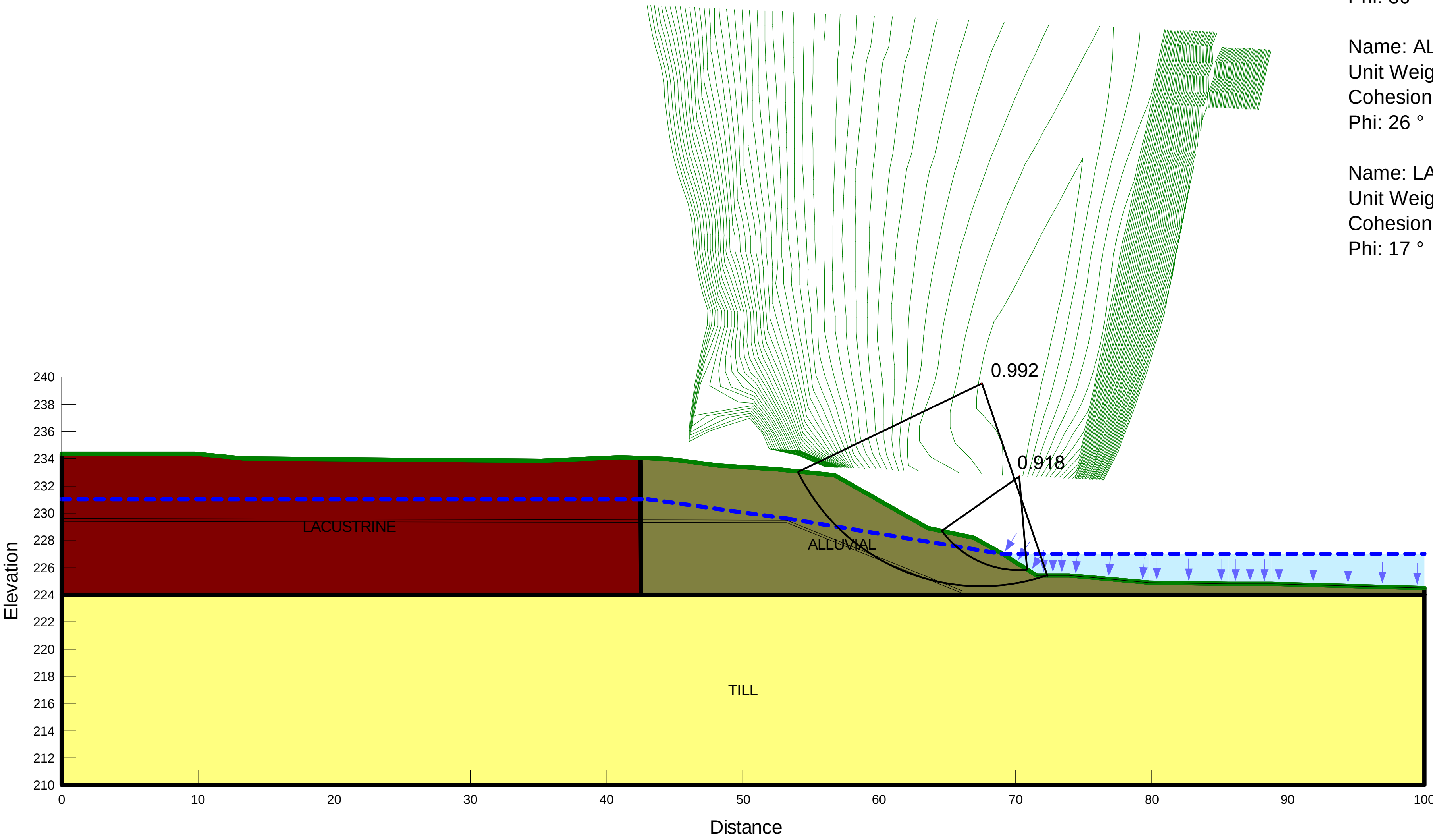


Condition Assessment of High Risk Water and Wastewater River Crossings
Title: Site 5- Assiniboine Park Siphon
Name: Case 5-3

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

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

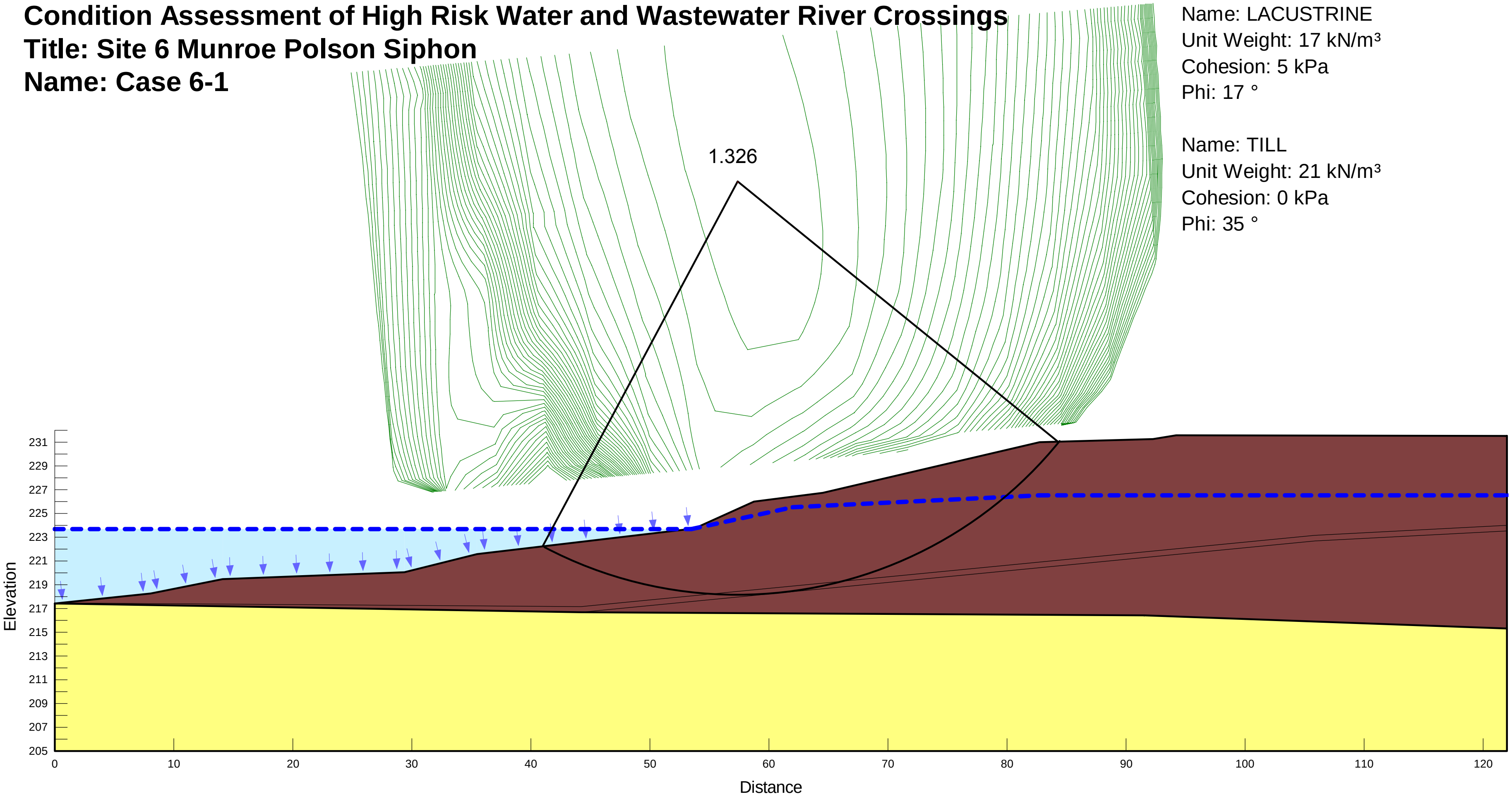
Name: LACUSTRINE
Unit Weight: 17 kN/m³
Cohesion: 5 kPa
Phi: 17 °



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

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

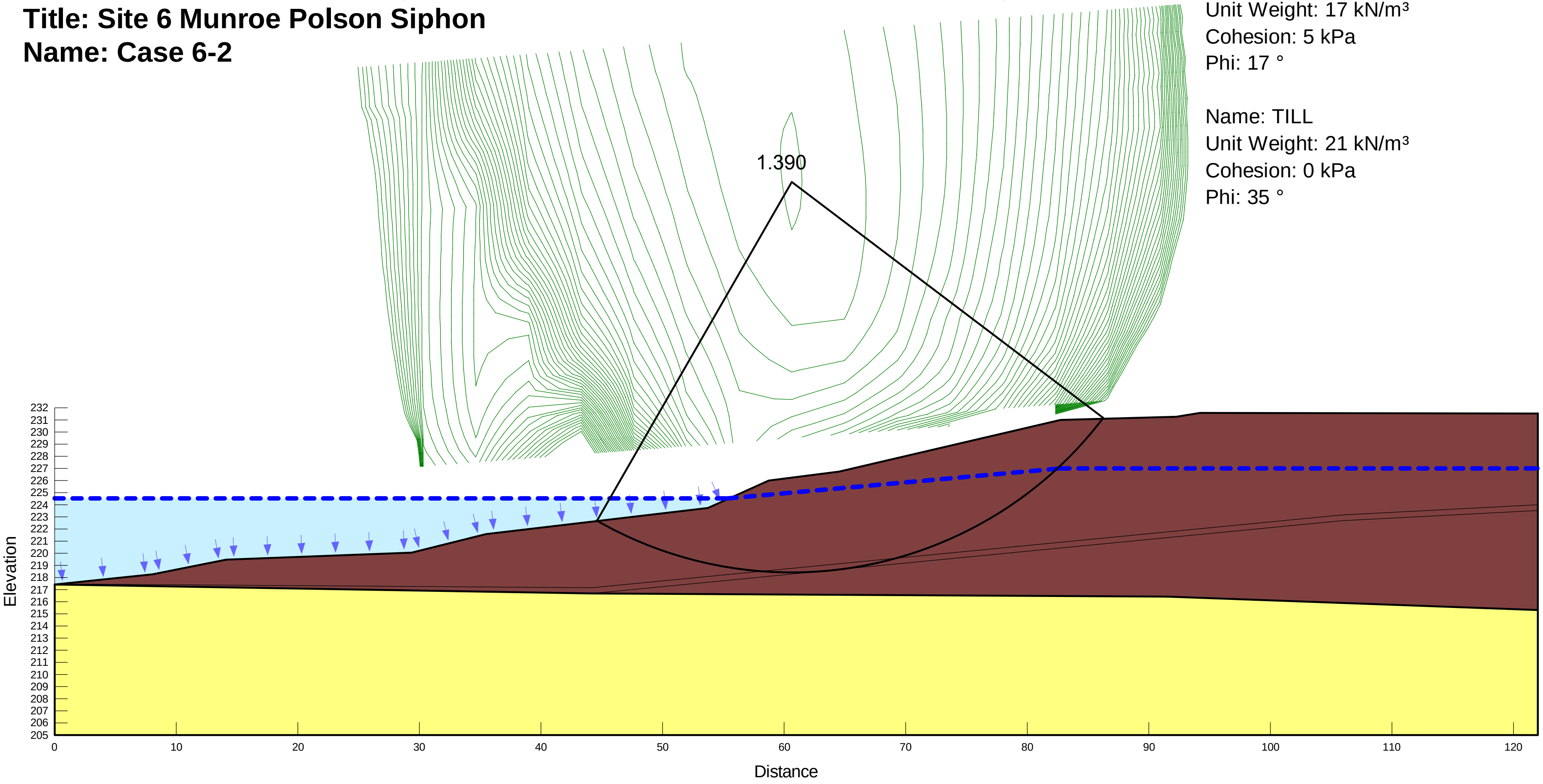
Name: TILL
Unit Weight: 21 kN/m³
Cohesion: 0 kPa
Phi: 35 °



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 °



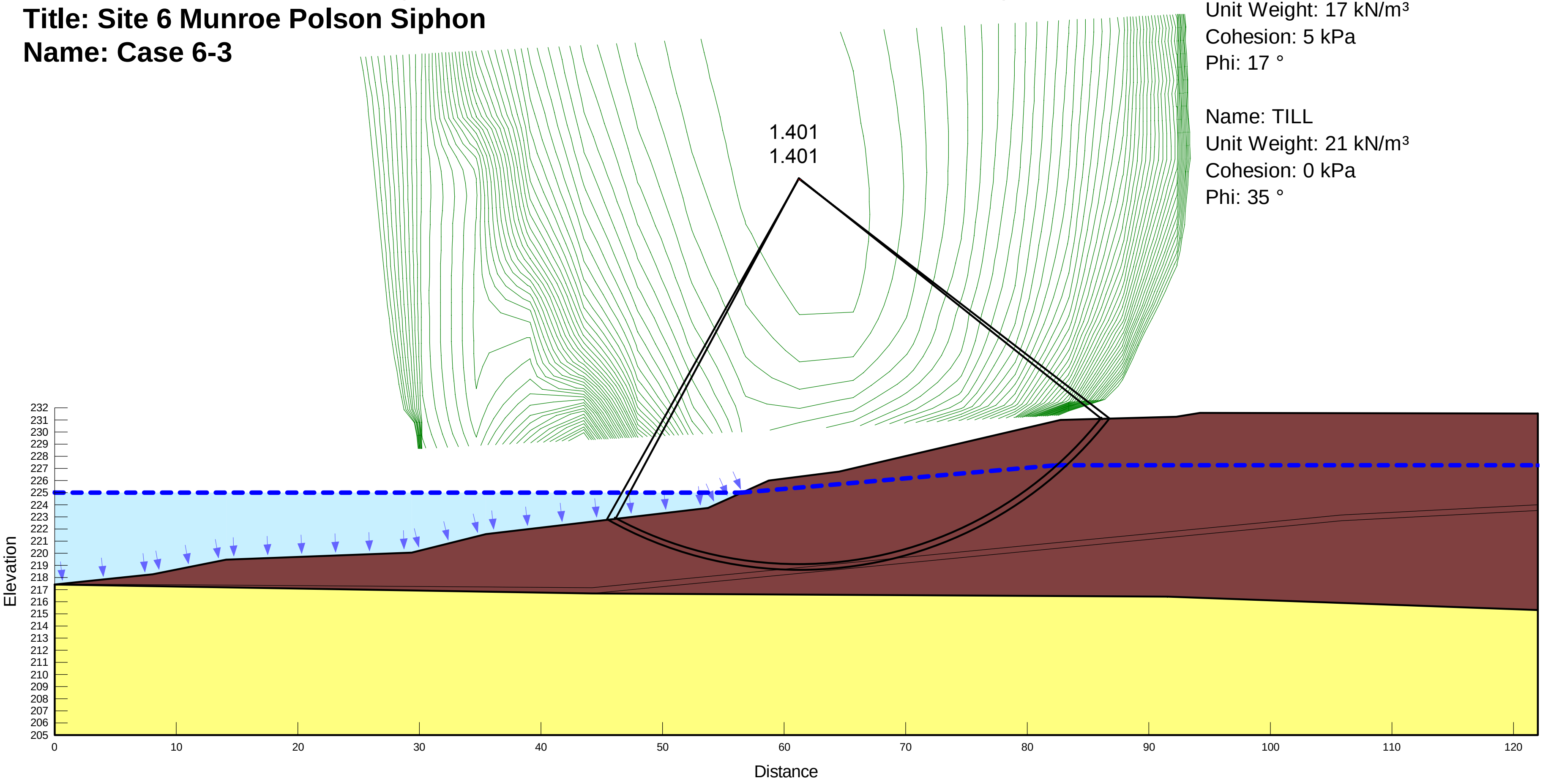
Condition Assessment of High Risk Water and Wastewater River Crossings

Title: Site 6 Munroe Polson Siphon

Name: Case 6-3

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

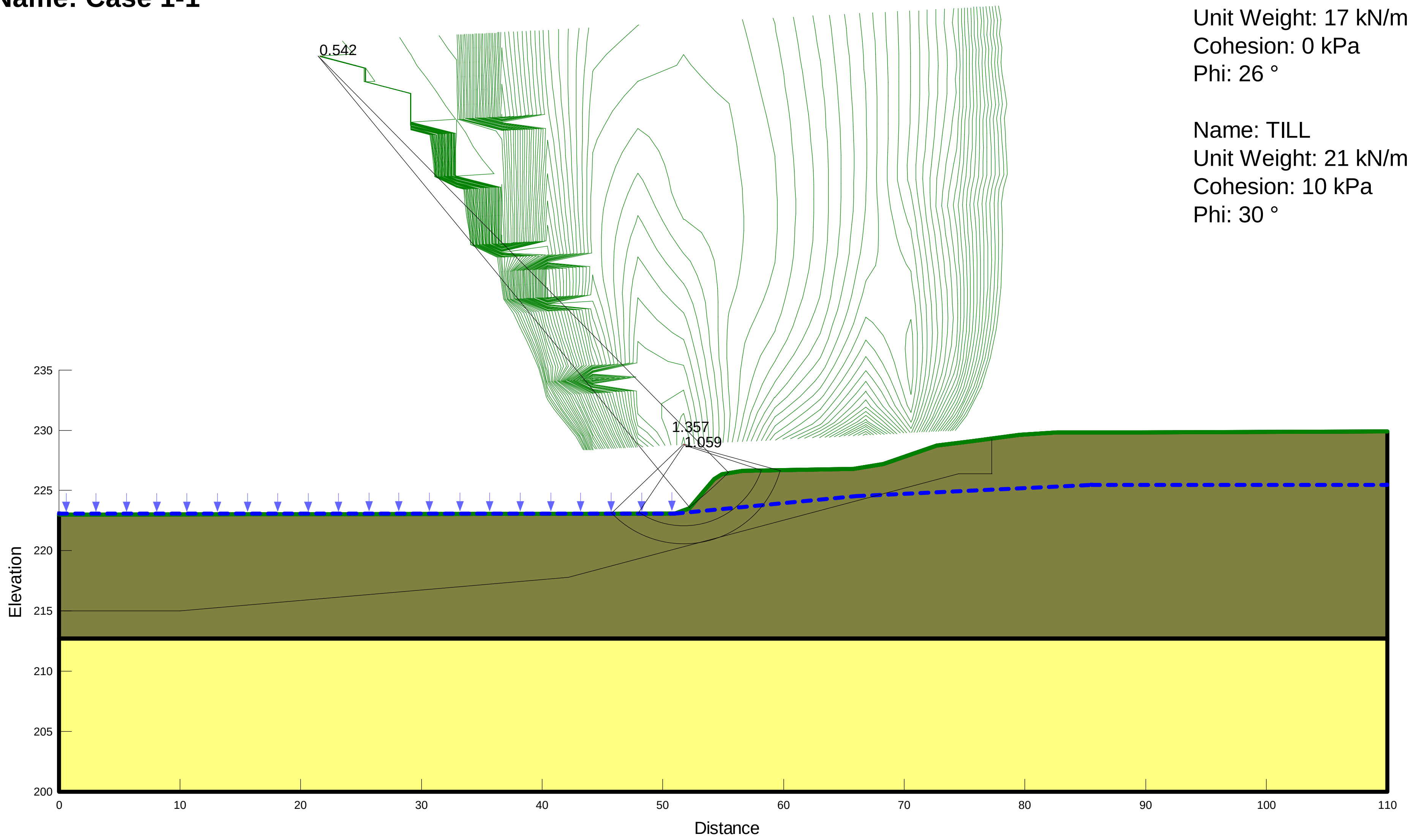
Name: TILL
Unit Weight: 21 kN/m³
Cohesion: 0 kPa
Phi: 35 °



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 °

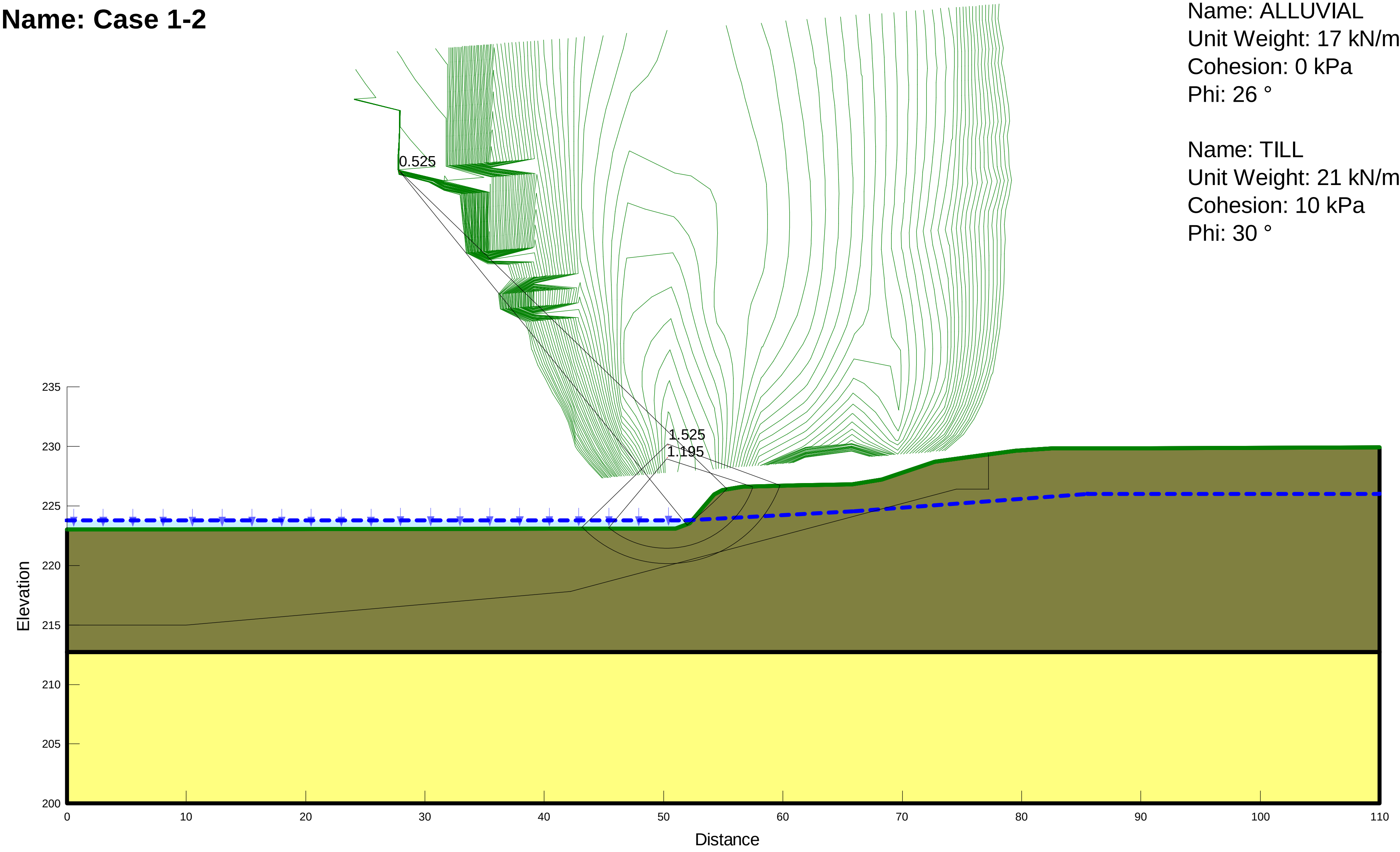
Name: TILL
Unit Weight: 21 kN/m³
Cohesion: 10 kPa
Phi: 30 °



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 °

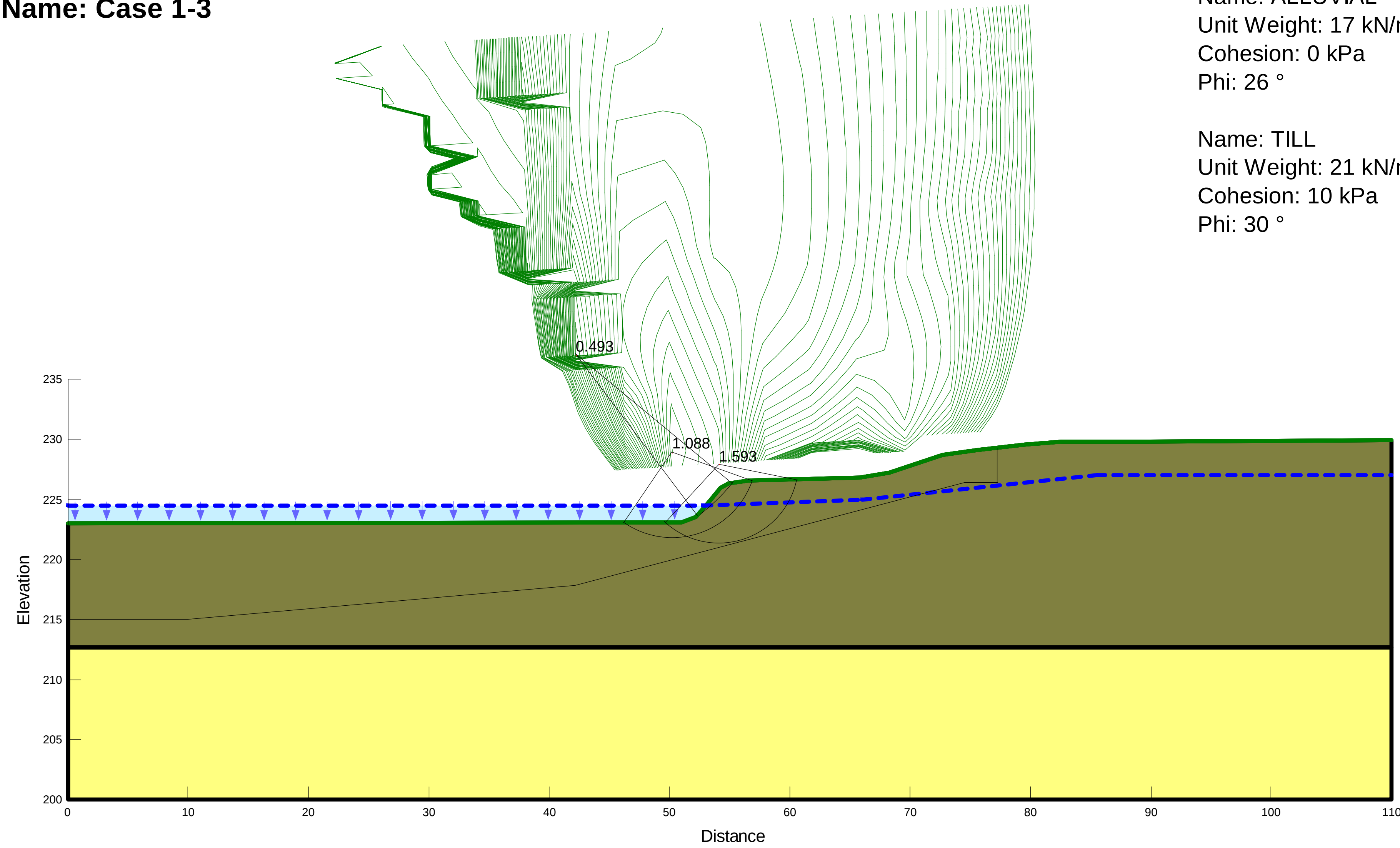
Name: TILL
Unit Weight: 21 kN/m³
Cohesion: 10 kPa
Phi: 30 °



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 °



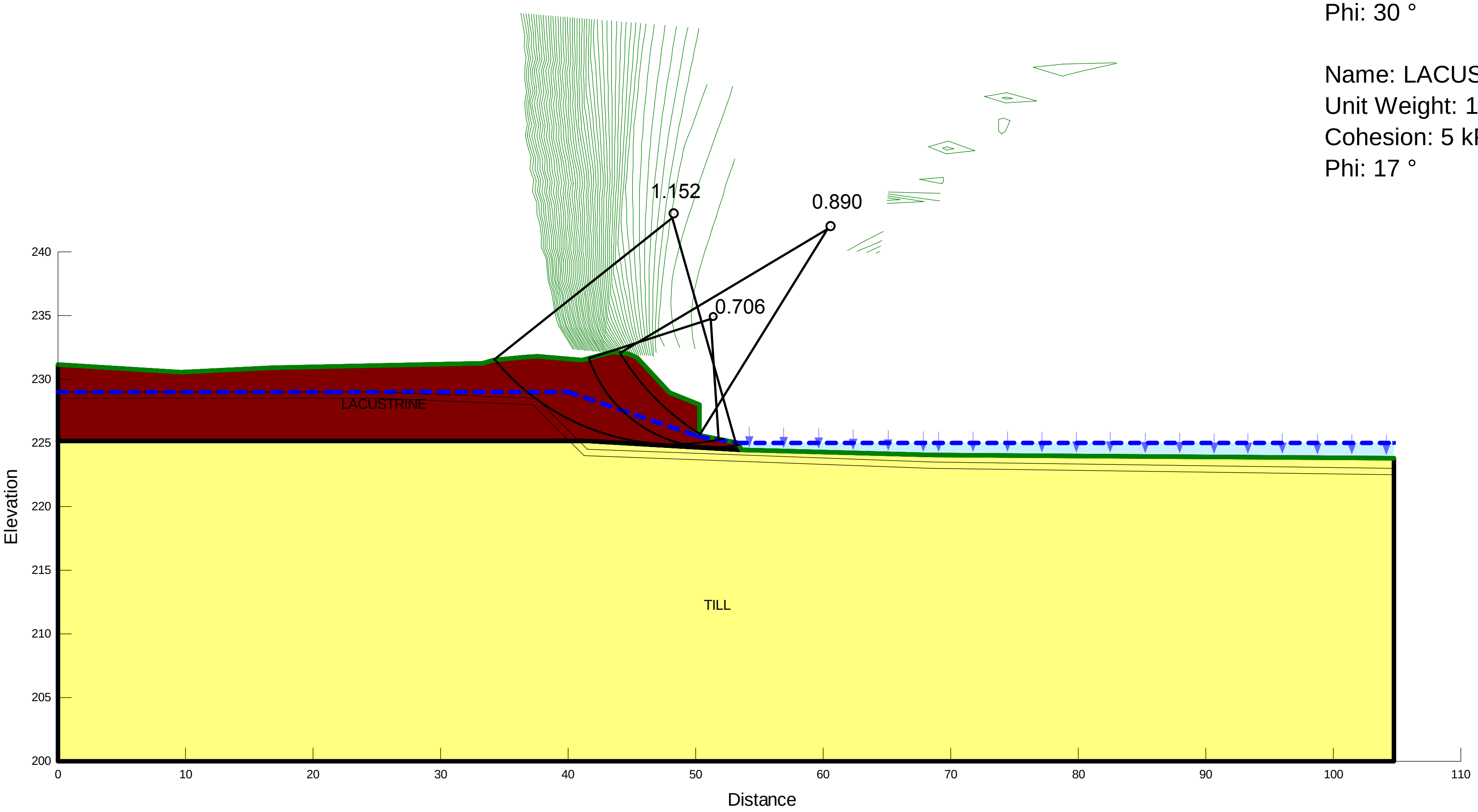
Condition Assessment of High Risk Water and Wastewater River Crossings

Title: Site 14 St James Street Watermain

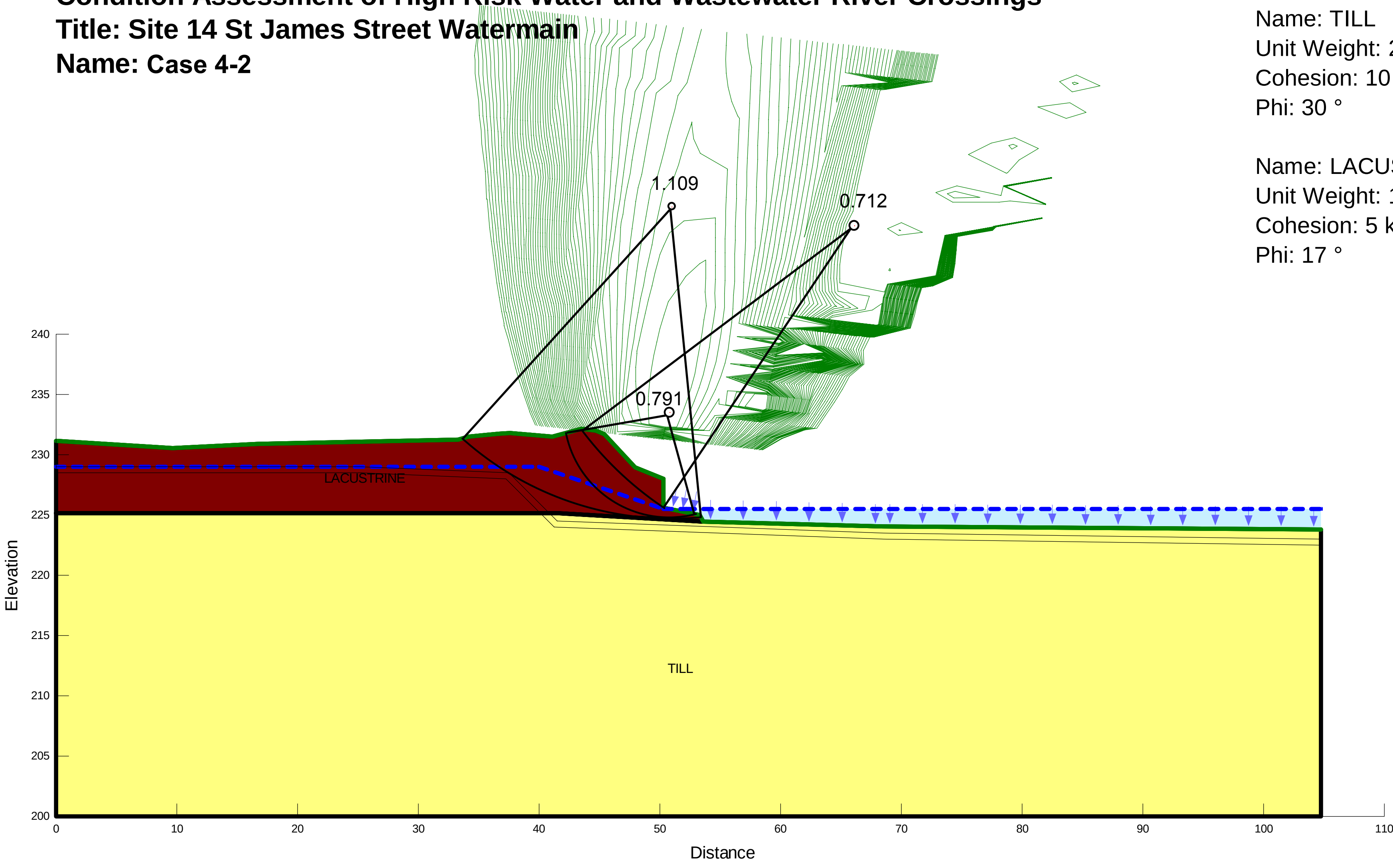
Name: Case 4-1

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



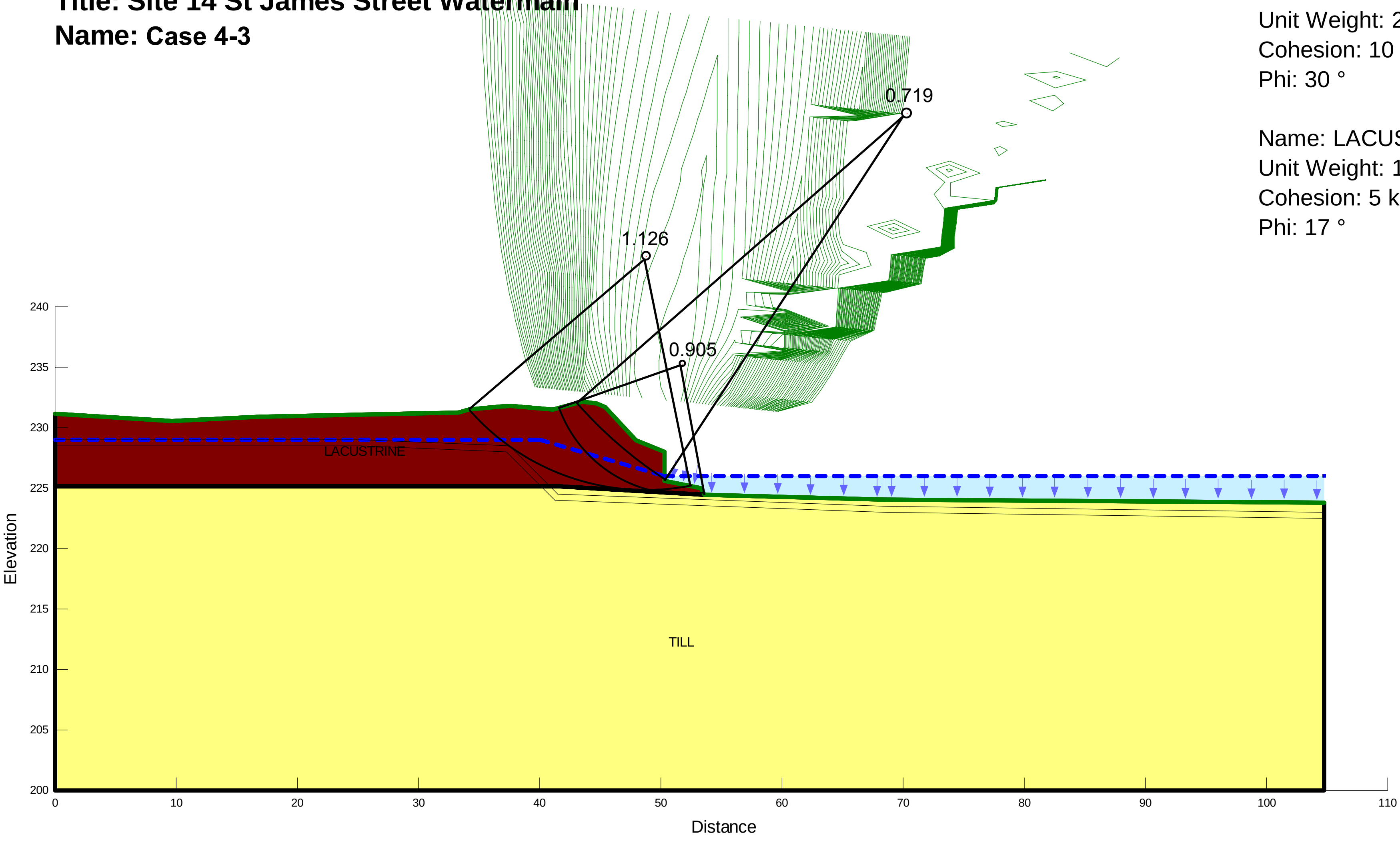
Name: TILL
Unit Weight: 21 kN/m³
Cohesion: 10 kPa
Phi: 30 °

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 E

Technology Selection Technical Memorandum

AECOM Imagine it. Delivered.

AECOM
99 Commerce Drive 204 477 5381 tel
Winnipeg, MB, Canada R3P 0Y7 204 284 2040 fax
www.aecom.com

To: Armand Delaurier, C.E.T. (City of Winnipeg)

Date: June 14, 2018
Project #: 60549028 (500)
From: Marshall Gibbons, C.E.T. and
Adam Braun, P. Eng.

cc: Marv McDonald, C.E.T., Chris Macey, P. Eng. and
Nathan Kehler, P. Eng., AECOM

Technical Memorandum

Subject: **High Risk River Crossing - Phase 2 – Inspection Technology Selection**

This technical memorandum details the rationale used for selecting appropriate non-destructive testing (NDT) pipe inspection technologies for use in assessing the physical condition of the river crossing pipes included in the above-noted project.

1. Background

The City has a regulatory requirement to quantify the condition of their potable water and sewer river crossings in an effort to prevent the inadvertent discharge of chlorinated or waste water into the environment. To meet these requirements, the City has undertaken a High Risk River Crossing (HRRC) inspection program, which was commenced in 2012 with Phase One of the HRRC inspection program (RFP 257-2012) and resulted in inspection of fourteen pipelines at ten sites. The inspection program yielded tangible results in the discovery of a severely deteriorated 500 mm St. James Interceptor Siphon which was subsequently repaired on an emergency basis using Cured-in-Place Pipe (CIPP) lining technology, identification of severely deteriorated water main on the Maryland crossing of the Assiniboine River, as well as identification of estimated remaining service life for the balance of the mains inspected.

The City of Winnipeg has again engaged AECOM under Phase Two of the program to inspect and assess the physical condition of six critical water and wastewater river crossing pipelines (Table 1).

Table 1 - List of Assets

Site	Name	River Crossing	Size (mm)	Material	Length (m)	Installation
1	Kildonan-Redwood Feeder Main	Red River	600	Steel	250	Buried
2	Charleswood-Assiniboia Feeder Main	Assiniboine River	600	Steel	185	Buried
3	St. Vital Bridge Force Main	Red River	500	Steel	202	Aerial
4	Newton Ave Force Main	Red River	350	HDPE	297	Buried
5	Heritage Park Force Main	Sturgeon Creek	250	PVC	250	Buried
6	Fort Garry - St Vital Feeder Main	Red River	600	Cast Iron	335	Tunneled

River crossings are typically critical to the operation of the City's regional distribution and collection systems. Their condition must be assessed to as high a degree of certainty based on the operational risks and environmental exposure to which the assets are subjected to. As shown in Table 1, the pipelines to be inspected vary in size, material, and installation. This memo will review procedures and technologies proposed for use in assessing the condition of both the ferrous metal and thermoplastic pipes identified as part of the program.

2. Assessment and Inspection Methodology

AECOM's approach to achieving the project goals will involve a process of review of the assets, their usage, assessing the risk associated with inspections, rationalizing the level of information required to understand condition in sufficient detail, reviewing the available inspection technologies, and selecting the most appropriate technology by weighing project requirements, risks and cost.

A first stage of choosing an inspection program is to assess how at risk from deterioration a particular asset might be, based on its original design, its known or assumed environment, and its operational parameters. As an example, assets with construction materials well matched to its environment and operational parameters, with a high design or safety factor, would not warrant the same level of attention and detail as an asset that is constructed of environmentally susceptible materials and an estimated marginal safety factor.

Another parameter in assessment of technologies is understanding the type of failure mechanisms that may be present for a particular asset and selecting inspection methodologies that are capable of detecting those types of defects. Other considerations in development of an inspection program might include;

- Mechanisms by which the pipes can deteriorate, and which of those mechanisms are most pertinent to evaluate;
- Assessing technologies capable of evaluating the state of pipe deterioration, and the accuracy and sensitivity of those technologies;
- Assessing deployment the inspection technologies through the pipes, and the operational requirements and deployment ranges of those tools;
- Details of pipeline construction such as pipe configuration / alignment, pipe jointing method, types and locations of fittings and valves, presence of protective coatings and linings (i.e. will the inspection tools pass through bends and other features to survey the entire pipeline segment without being damaged, and without damaging the line);
- Physical accessibility of the pipes (i.e. can the inspection tools be readily deployed into the pipes or must the piping or tools be modified in some way to enable the inspections);
- Operational accessibility of the pipes (i.e. can the inspection tools be readily deployed into the pipes without adversely impacting service, or must special operating conditions such as by-pass pumping, flow modulation or off-hours inspections be implemented);
- Presence of other infrastructure that may affect the inspection results, such as casing pipes, tunnel liners and high voltage power lines;
- Total cost of acquiring the pipe condition data, including the costs of modifying and cleaning the pipes, of performing the inspections, and of processing and assessing the data; and
- The value of the knowledge gained by performing the inspections.

Guidance regarding the selection and use of tools for evaluating pipe condition can be found in various documents, including NACE International Standard Practice SP0102 – In-Line Inspection of Pipelines.

Quantitative inspection and condition assessment of water and wastewater pressure pipelines are difficult and often challenging to undertake since most are buried or otherwise covered (insulated) and cannot be readily inspected from the exterior. Further, most municipal systems were not constructed with adequate access to enable insertion of inline tools to inspect them from the interior. Though visual inspections may be conducted externally at isolated excavations, or internally within larger-diameter pipes (confined entry), much of the deterioration that can occur within the pipe wall is not readily visible. River crossings, pose additional challenges due to unique geometry, limited access, and additional risks due to a lack of external access. Inspection and assessment of critical pipelines is further impeded because they often cannot be removed from service to prepare for and undertake the inspections without considerable planning and effort.

In some cases, the proper implementation of conventional inspection technologies such as CCTV and SONAR can provide valuable information for comparatively low cost when deployed from readily accessible locations such as manholes, pumping stations and valve chambers. Where more detailed data is required, advanced in-line inspection (ILI) technologies will be utilized, particularly in the ferrous metal pipelines. While these tools offer an excellent solution in terms of inspection, deployment of in-line inspection methods pose some risk to the pipelines, including

- The inspection tool becoming stuck or lodged in pipelines at unknown blockages, alignment deviations, or other obstructions;
- Damage to the pipeline during installation of pipeline modifications to facilitate deployment of the inspection tool; and
- Damage to the pipeline or pipeline coatings by deployment winches, cables tool contact, or cleaning processes.

AECOM has developed a separate risk assessment memo to address risks associated with pipeline inspections, dated June 2018.

3. Technology Selection

A variety of material-specific technologies and broad-use technologies (apply to virtually any pipe material) were reviewed for use inspecting the Winnipeg river crossing pipes, including:

- Electromagnetic (EM) tools which can detect wall loss in ferrous metal pipe
- Remote camera inspection (CCTV) which can detect visual defects
- SONAR which can detect debris accumulation, air pockets and general geometric shapes of any pipe material
- Leak Detection systems which can detect leakage in higher pressure systems
- Ultrasonic (UT) measurement that can detect wall thickness and defects within the pipe wall
- Pressure testing which can confirm if asset is actively leaking
- Opportunistic sampling which can confirm pipe material properties

A complete description of inspection technologies and specific vendor information is included in Appendix A

Broad-use technologies such as closed-circuit television (CCTV) and sonar technologies can be used for any pipe material to gather valuable information to supplement the condition assessment process, or to assess tool deployment risk. Their use in condition assessment, however, is limited to “visual” classification only of defects on the interior face of the pipe, and of the way the pipe is reacting to the soil stresses around it. In gravity pipes this is often an adequate level of assessment. In pressure pipe flow, however, 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) used on ferrous metal pipes, were developed to acquire quantitative physical information on residual pipe structure and/or to detect specific types of defects and defect geometry. Although deployed from only one side of the pipe they can obtain quantitative data of physical condition beyond the visible surface. 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 capabilities of various tools to detect and quantify 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.

Specifically, the Request for Proposal for the project suggested consideration of continuous electromagnetic inspection methods such as Remote Field Eddy Current (RFEC, or 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.

Presently there are no proven inspection platforms developed for continuous measurement of pipe wall condition for the non-metallic crossings. Therefore, the technical approach for condition assessment of the non-metallic pipes needs to be a balanced approach of alternative techniques, planned and/or opportunistic sampling and reviewing relevant mechanical properties over time, and a thorough understanding of the applied loads on the pipe to indirectly assess condition and material failure risk. The technical approaches for assessing the Newton Ave (HDPE) and Heritage Park (PVC) Force Mains are discussed separately in this section.

Pipelines included in the second phase of the HRRC assessment program include four ferrous metal pipelines (three steel and one CI) and two thermoplastic pipelines, (one HDPE and one PVC).

Defect and failure mechanisms for the various pipe types and technologies that can be used to detect these include;

Ferrous Metals (Steel, Cast Iron)

- External general corrosion - EM tools
- Pitting corrosion/pinhole corrosion usually the result of spot defects in protective coatings - EM Tools
- Graphitization of metal (in case of Cast Iron) – EM tools
- Splitting as a result of excessive internal pressure – Pressure Testing, EM tools
- Excessive deflection as a result of external loads – SONAR, CCTV, gauge pigging/mandrels
- Buckling as a result of excess external pressures -SONAR, CCTV, gauge pigging/mandrels
- Internal erosion as a result of high velocities -SONAR, CCTV, EM tools

PVC and HDPE

- Excessive deflection as a result of external loads - SONAR, CCTV, gauge pigging/mandrels
- Buckling as a result of excess external pressures -SONAR, CCTV, gauge pigging/mandrels
- Slow crack growth in HDPE pipe as a result of long term stress, particularly in non-modern high performance PE resins – Pressure testing, opportunistic sampling
- Cyclic fatigue stress in particularly in PVC pipe - Pressure testing, opportunistic sampling
- Hoop tension failures (splitting) as a result of excessive internal pressure -Pressure testing
- Internal erosion as a result of high velocities and debris- SONAR, CCTV

4. Site-by-Site Assessment Approach

Given the varying material types, installation condition, and access availability for each pipeline, no one methodology can be implemented across all sites. As outlined above, a balanced approach was used to select the most appropriate inspection technology for each pipeline, considering the cost of implementation, the risks associated with deployment, system operations, required access modifications, and the value of the information gained.

Site 1 – Kildonan-Redwood Feeder Main

The 600 mm Kildonan-Redwood Feeder Main is a buried crossing of the Red River located parallel to the Harry Lazarenko Bridge (formally the Redwood Bridge) between Main Street and Glenwood Crescent. This asset was installed in 1954.

The crossing consists of 600 mm steel pipeline installed via a tunnel shaft on the west bank and direct buried beneath the river and up the east bank. Fittings along the length of the crossing include a flanged cast iron 90 degree bend and welded steel mitered bends of various deflections, assumed to be less than 45 deg. Based on the pipe material, age of asset and configuration, the crossing has been identified for inspection using an inline RFT inspection platform. A high resolution platform is recommended to detect potential pinhole and pit corrosion areas.

Full size access into the crossing can be developed through disassembly and physical modification of the existing valve chambers. These modifications include:

- Removal of an existing side outlet 90 degree bend within the west tunnel shaft. The existing fitting is to be reinstalled upon completion of the inspection work.
- Disassembly of the piping within the east valve chamber. Chamber piping to be modified upon to reassembly to remove an existing hydrant lead and an abandoned 300 mm offtake.

Modification of assets, inspection, disinfection and restoration of the main is expected to take three to four weeks.

Site 2 – Charleswood-Assiniboia Feeder Main

The 600 mm Charleswood-Assiniboia Feeder Main is a buried crossing of the Assiniboine River located between Berkeley Street and Rouge Road. This asset was installed in 1965.

The crossing consists of a direct bury 600 mm steel pipeline with 30 degree mitered steel bends (3 piece). Based on the pipe material, age of asset and configuration the crossing has been identified for inspection using an inline RFT inspection platform. A high resolution platform is recommended to detect potential pinhole and pit corrosion areas

Full size access into the crossing can be developed through the installation of launch wye assemblies adjacent to the existing valve chambers.

Modification of assets, inspection, disinfection and restoration of the main is expected to take three to four weeks.

Site 3 – St. Vital Bridge Force Main

The 500 mm St Vital Bridge Force Main crosses the Red River along Osborne Street between Churchill Drive and Kingston Row. This asset was installed in 1989.

The force main, which conveys combined sewer flows from the Baltimore Road Pumping Station, was constructed by suspending 500 mm steel pipe from the underside of the St. Vital Bridge. 90 degree mitered steel bends were utilized to bring the pipeline above grade on either end of the force main. Exposed portions of the force main are covered with 50 mm of polyurethane and 22 gauge steel cladding.

While technically feasible, access into the pipeline to complete an inline inspection was not considered practical due to the following considerations:

- The upstream (Baltimore Road) pumping station operates without redundancy, as is typical with most pumping stations, and thus the pump station cannot be kept off line for long periods of time. As the force main crossing is non-redundant, the pipeline access modifications would need to be completed during short pump station shutdowns or under live flow conditions. Neither of which are ideal from a risk planning perspective.
- With respect to the inspection, the force main also contained 90 degree bends where the force main transitions onto the bridge. This could restrict traversing the pipeline with the inspection tool and necessitate installation of the launch points at an elevated location.
- Due to the short shutdown windows available, the inspection would have to be completed during live flows and tethered due to the high velocities within the force main during operation.

Based on work undertaken in Wastewater River Crossing Leak Detection Trials[1] program, it is known that the pipe did not leak and, being a suspended crossing, could be inspected by other less invasive methods. The age of this asset is only approximately 30 years, and its external environment would not be conducive to external corrosion problems, other than potential for bridge de-icing chemicals. The operating pressures of this asset are well below the capacity of this pipeline, thus expected stress levels are considered very low.

Based on the risks associated with internal inspection, it is recommended that an external inspection be conducted. The force main is suspended below the bridge deck and is accessible via the river bank and/or an under bridge crane (UBC), facilitating external access. Magnetic external inspection technologies require removal of the external ferrous metal cladding. It is proposed to undertake a targeted external inspection of the pipeline, selecting 3 to 4 representative locations where the external cladding and insulation will be removed,

---

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

wall thickness measurements taken using external magnetic or UT inspection technology. Upon completion of the inspection half shell insulation assemblies will be installed complete with new galvanized cladding.

AECOM is proposing a two-stage approach for the inspection of this pipeline.

Stage one involves a preliminary inspection of the pipeline in order to assess the most likely points of corrosion related wall loss. AECOM personnel performed a preliminary visual inspection of the force main utilizing the City's Under Bridge Crane (UBC) on February 8, 2018. This preliminary inspection focused on the following:

- A full length inspection of the pipeline and cladding in its current state.
- Identifying any visual locations where moisture (rain, road salts, etc.) could be impacting the pipeline.
- Identify any visible corrosion on the steel cladding that's indicative of leakage or externally driven corrosion.
- Inspection of the pipe supports, bends and expansion joints, and air release valves.

Figure 1 depicts one of the points of interest on the force main, where a drain port was installed without restoration of the exterior cladding.

Stage two will involve representative detailed inspection of the pipe, based on results of the preliminary inspection and a review of the force main's operational and environmental exposure to identify locations at a higher risk of experiencing internal and external corrosion. Recommendations for inspection locations will be provided with the draft support tender package. Inspection of 3 to 4 representative locations will provide adequate information to infer the condition of the crossing as a whole.

Figure 1: Under Bridge Inspection

Site 4 – Newton Ave Force Main

The 350 mm Newton Avenue Force Main crosses the Red River between Newton Avenue on the west side and Fraser's Grove Park, between Rossmere and Larchdale Crescents, on the east side.

The crossing consists of a direct buried 350 mm HDPE crossing conveying flows from the Hawthorne Pumping Station. The crossing runs in parallel to the 350 mm steel crossing servicing the Linden Pumping Station. Modifications in piping and valving completed under HRRC Phase 1 allows for a single force main to accommodate flows from both lift stations during dry weather periods, to permit inspection of either of the crossings.

There are no ILI tools currently available on the market to directly assess the structural deterioration of thermoplastic pipe. Thus our inspection is focused on assessing the pipeline circumferential geometry for evidence of structural distress (deflection) or conditions which could indicate high stress levels in the pipe wall.

A sample from the force main was obtained and tested under the HRRC Phase 1 program. Results from the testing were summarized in a technical memorandum dated July 14, 2017. The testing completed by NSF Canada identified that the HDPE force main exhibited low PENT values, which could indicate pipe is at risk of slow crack growth due to brittle failure of the pipe material. The lifespan of the pipe will likely be governed by the onset of slow crack growth.

HDPE pipe exhibiting low PENT test values are sensitive to long term, high applied stress operating conditions. This could include sustained high internal pressures (not a factor for this crossing) or excessive ring deflection caused by poor installation practices, high external soil loading, geotechnical conditions, or a combination of all three. A SONAR inspection will provide information on the pipelines deflection along its entire length which will allow us to predict pipe wall stress and assess its risk to slow crack growth failure. This inspection will be coupled with a low head pressure test to confirm the hydrostatic integrity of the pipeline and allow us to assess the condition and identify if there is an immediate risk of failure.

The cost of mobilizing SONAR equipment for a small quantity of inspection could be costly, thus AECOM is reviewing addition of this crossing inspection to other sewer condition assessment works projects.

Site 5 – Heritage Park Force Main

The Heritage Park force main is a 250 mm diameter PVC main that crosses Sturgeon Creek along Ness Avenue between Valley View Drive and School Road. Waste water flow is pumped in a westerly direction from the Heritage Sewer Pumping Station and discharges into a manhole at the intersection of Ness Ave and School Road. The river crossing pipe was replaced in 1989, and is constructed from 250 mm Class 150 AWWA C900-89 pipe with a Dimension Ratio (DR) of 18.

PVC, as with all thermoplastic materials, does not corrode and unless exposed to corrosive waste streams will not deteriorate over time due to environmental operating conditions. PVC, however, is susceptible to fatigue and physical loading conditions (i.e. excessive deflections) and cyclical internal pressures that could result in long term stress related failures. A desktop study was undertaken to determine the pipelines susceptibility to stress including excess ring deflection due to external loads and cyclical fatigue due to internal pressure operating conditions.

For any flexible pipe, proper installation (i.e. compaction of the bedding) plays a significant role in developing the full load carrying capacity of the pipe. Historical record drawings were reviewed to determine the age and installation conditions of the pipeline. The pipe was then analyzed using flexible pipe theory with an assumed “worst case” installation condition in order to develop a conservative estimate of the stress and deflection that the pipe would likely experience in the field. Utilizing the Modified-Iowa formula for flexible pipe deflection under full overburden load and hydrostatic pressure, the pipe was found to experience approximately 2% deflection under an assumed worst-case installation condition. As deflection limits of 5% to 7.5% are typically considered acceptable for PVC pipe[2], the risk of the pipeline being exposed to high stress loading conditions resulting from poor installation is low.

---

[2] *ASTM International, ASTM D3034, Standard Specification for Type PSM Poly (Vinyl Chloride) (PVC) Sewer Pipe and Fittings, November 1, 2016.*

Based on provided operating pressures for the pipeline (50’ or ~22 psi Total Dynamic Head), and a worst case assessment of pressure cycles, cycles to failure is beyond 1 x 10^8 , which would equate to several hundred years of design life based on cyclic fatigue.

Based on the assessment above, the pipeline has a high design safety factor, and should not be at significant risk of operational or environmental induced failure. The pipe overall is constructed of modern materials, and still well within its expected design life.

Options for in-line inspection for this pipeline were explored using a sonar platform, however, through discussion with vendors it became apparent that the relatively small diameter of the pipe and the presence of multiple 90 degree elbows would make it extremely difficult to complete a successful in-line inspection without extensive modifications to the force main in addition to the construction of tool launch assemblies. Given the analysis of the pipeline outlined above, AECOM does not propose an inline inspection and associated piping modifications at this time.

To satisfy regulatory reporting on condition, AECOM proposes that a pressure test be performed on the line to verify that there are no active leaks, at a pressure equivalent to the maximum assessed pressures plus a reasonable safety factor. AECOM also recommends that a sample of the PVC force main be collected and sent for testing at an accredited lab. This methodology will provide sufficient information for us to infer the condition of the crossing as a whole.

Site 6 – Fort Garry - St Vital Feeder Main

The Fort Garry - St Vital Feeder Main is a 600 mm grey cast iron pipeline crossing the Red River between the Fort Garry Bridges on Bishop Grandin Boulevard. The feeder main crossing was installed in conjunction with the Branch II Aqueduct which is installed within tunnel shafts and a tunnel within the underlying limestone bedrock crossing beneath the river. Subsequent to installation, the tunnel shafts and tunnel were filled with concrete, permanently encasing the pipelines. The pipe was installed in 1959 and put into service in approximately 1960, and has been in service approximately 60 years.

The concrete encasement of the cast iron piping within the underlying bedrock creates an ideal environment for the crossing as the pipe is generally shielded from all overburden loads and is protected from corrosion due to the high pH environment created by the concrete. Thus, exterior corrosion is not anticipated to be a factor in the deterioration of the cast iron pipeline. Interior corrosion of ferrous metal water mains is typically not a driver in pipeline deterioration in the City of Winnipeg due to both water quality and the nature of interior corrosion, which typically acts in more of a uniform manner than that of exterior driven corrosion. As excessive corrosion is not anticipated for the crossing, an in line leak detection survey combined with visual inspection utilizing tools such as Pure’s Sahara acoustic leak detection/CCTV inspection tool is proposed for this site. A visual inspection will allow us to identify a number of potential issues:
‘

- Presence of interior cementitious coating: To date, original specifications for the cast iron pipe have not been provided thus it is unknown if an interior cementitious coating was applied during manufacturing. The use of Sahara will permit a visual inspection of the pipe’s interior surface allowing us to confirm the presence of interior cementitious coating.
- Visual assessment of interior corrosion.
- Determine debris levels. Given the nature of the siphon with vertical drop shafts and 90 deg bends, there’s the potential for debris buildup within the siphon. An assessment of debris levels is prudent prior to undertaking cleaning and more advanced inspections.

To deploy the proposed inspection tool, a pipeline flow velocity of approximately 1 metre per second (m/s) is required. The City has identified through modelling that this could result in significant system wide impacts. AECOM and inspection vendors are evaluating alternate means of inducing the required velocities, or assessing the inspection geometric and length constraints should lesser flow velocities be utilized to reduce system impacts.

AECOM suggests that based on the pipeline environment that a visual inspection coupled with leak detection will be sufficient to evaluate the pipeline's remaining service life and satisfy regulatory reporting requirements. Should this inspection identify areas of concern warranting a more detailed inspection, an inline RFT tool capable of navigating the 90 degree cast iron bends can be deployed in a manner similar to that of the other ferrous metal feeder main crossings.

5. Conclusion

The proper selection of inspection technologies is critical to obtaining the correct information and subsequent condition assessment efforts. Unfortunately, there are both limited vendors for this type of inspection work making the development of accurate tender documents and imbedded qualifications critical to the procurement process. This is likely to the development of tenders geared towards single inspection vendors as noted herein. However, this is necessary so that the inspections are undertaken in a manner that permits accurate condition assessment and development of an asset management program for these critical assets.

We trust this information meets your requirements on this matter. Should you have any queries or require further information or clarification, please do not hesitate to contact either of the writers or Marv McDonald of this office.

Marshall Gibbons. C.E.T.
Senior Municipal Technologist
Conveyance
MG/NJK/ADB/pab

Adam Braun, P. Eng.
Municipal Engineer
Conveyance

Appendix A

Appendix A. Inspection Technologies Considered for the River Crossing Pipes

A variety of material-specific technologies and broad-use technologies (apply to virtually any pipe material) were reviewed for use inspecting the Winnipeg river crossing pipes, including:

- Electromagnetic (EM).
- Remote camera inspection (CCTV)
- Sonar
- Leak Detection systems
- Ultrasonic (UT)

A1.1 Electromagnetic (EM) Inspection Technologies

EM inspection technologies detect and quantify defects and thickness variations in ferromagnetic materials by sensing distortions caused by those defects/variations to a baseline magnetic field that the tool induces in the material. The EM technologies are currently the best option available for undertaking continuous wall condition inspections of the ferromagnetic river crossing pipes, where they can feasibly be deployed.

In-line EM pipe inspection tools were introduced in the petroleum industry in the 1950s to enable inspection of steel pipes. It was not until approximately 1990 that the technology was adapted for use on water mains by a predecessor firm of Russell NDE Systems Inc. of Edmonton. Today there are a variety of EM tools available for inspecting pipes comprised wholly or partly of ferromagnetic materials including steel, cast and ductile iron, reinforced concrete and concrete cylinder pipe.

EM inspection tools typically are multi-segment articulated tools that incorporate an emitter coil that induces the magnetic field in the pipe, detector coils that sense the induced magnetic field and its distortions, electronics modules that provide power, control and datalogging functions, and centralizers that keep the tool and its sensors centered within the pipe (i.e. maintain sensor stand-off, or gap, from the pipe surface within a desired range).

The ability of a tool to detect and accurately quantify defects and thickness variations in pipes hinges largely on maintaining sensor “stand-off” during the survey, and in calibrating the tool to the pipe being inspected. It is important to note that EM tool calibration and data analysis become more difficult if the pipeline being surveyed is comprised of multiple materials and/or wall thicknesses.

EM technologies considered for use on this project based on their ability to be applied in a continuous assessment mode to inspect the entire river crossing pipe, and to meet acceptable defect resolution objectives given the operating and configurational constraints of a particular crossing pipe, include the following:

- Remote Field Technology (RFT) from Pipeline Inspection and Condition Assessment Corporation (PICA) of Edmonton.
- EM Technology (formerly Enhanced EM) from Pure Technologies Ltd. (Pure) of Calgary
- Bracelet Probe from PICA
- Magnetic Flux Leakage (MFL)

A1.1.1 Remote Field Technology (RFT) from PICA

PICA presently offers three inline RFT tool platforms, all of which are available in tethered or free-swimming configurations with onboard data storage, with dedicated units for inspecting either water or wastewater pipelines:

- HydraSnake – sizes 150, 200 and 400mm.
- See Snake – sizes 75 through 350mm, and 600mm.
- Chimera (Figure A 1) – sizes 400mm through 900mm, with 1050mm tool being developed.

Figure A 1 - Chimera RFT Tool (courtesy of PICA).

PICA's inline RFT pipe inspection tools use a single emitter coil that is separated axially along the pipe from an array of detector coils that are spaced around the pipe circumference. When energized by a low frequency alternating current voltage, the emitter generates an electromagnetic field that travels longitudinally along the pipe via two paths, one inside the pipe and one outside, with the pipe wall serving as a wave guide. The internal field attenuates to almost nothing within about 2.5 pipe diameters from the exciter coil. Detector coils situated beyond that distance can detect amplitude (strength) and phase (time of flight) variations that occur in the outer (remote) field when it returns through the pipe wall (Figure A 2). Since the behaviour of the outer field is influenced by the condition and thickness of the pipe wall, the any variations from a "standard" reference field can be used to identify and quantify defects and wall loss caused by cracking, pitting, graphitization and erosion. The locations of welds, pipe joints, fittings, and ferromagnetic features in close proximity to the pipe can also be detected.

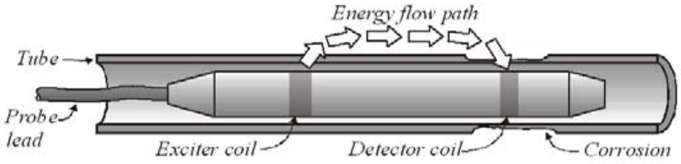


Figure A 2 - Sketch of Tethered RFT Tool Inspecting Pipeline.

The electronic components of PICA's RFT tools are all contained within sealed stainless steel modules that are connected to each other using flexible joints and centralizers that enable the multi-module inspection tools to negotiate the low-radius bends typically used in municipal piping systems (AWWA C153, for which the axial radius of bends is less than or equal to one pipe diameter), and which keep the tool centered in the pipeline with a small amount of stand-off that enables it to freely pass by sediment, corrosion tubercles, welds, pipe deflections, protrusions (dents) and other restrictions. Unlike some MFL tools, PICA's EM tools do not have to make direct contact with the pipe wall to sense its condition, and therefore can be used to inspect coated and lined pipes. Stand-off for various tools ranges from a minimum of 6.3 mm up to 38 mm depending on the particular tool and pipe diameter.

Tools can be deployed tethered or non-tethered and are configured for bidirectional movement. Tethered tools have a cable range of approx. 1000m (3300 feet), while the range of free-swimming tools is generally limited by pipeline flow velocity and on-board battery life, which can range from 12 hours to 72 hours. Free swimming tools are fitted with odometers, while cables of tethered tools run over odometers sheaves.

Speed of survey is such that pipe measurements can be acquired at 1.5mm intervals longitudinally along the pipeline. Typical surveys are performed at a speed of 3 to 5 m/min, to a maximum of 10 m/min, with data being recovered in both directions for redundancy and to permit comparison.

The ability of PICA's inspection tools to obtain accurate quantitative data has been proven through evaluation of many kilometres of inspection in studies and trials for the City of Calgary and others. Study by the City of Calgary [1] determined that PICA analysts were able to interpret pit depths using RFT technology to within 20% of physically measured pit depth 95% of the time, and that RFT could accurately detect very deep or full penetration pits over 90%

Operational parameters of PICA's EM inspection tools are listed in Table A 1.

Advantages to RFT technology include:

- Detection sensors do not have to be in direct contact with the pipe wall, so tool can inspect pipes that have internal linings, including the commonly-used cement-mortar and coat tar epoxy linings.
- Technology will operate with pipe in or out of service, and with pipe in watered or dewatered condition.
- Tool is equally sensitive to wall loss on both the interior and exterior of the pipe.
- Tool can acquire condition data around full circumference of the pipe and for the entire length of pipe surveyed.
- Tools can operate under pressures up to 700 psi.
- Tools capable of negotiating short radius bends that are commonly used in municipal piping systems (axial radius of curvature less than 1 pipe diameter).

Draw-backs to using PICA's RFT technology include:

---

[1] *Hartman, W.F., K. Karlson and R. Brander, Waterline Restoration Based on Condition Assessment, Sep. 2002, 10 pp.*

- Analyst may misinterpret the metal loss in a tight grouping of pits as a single large pit.
- Defect resolution decreases as pipe wall thickness increases above 12.7mm.

Table A 1 - Operational Parameters of PICA's EM Inspection Tools.

Parameter	Description
Pipe Material	Iron, Steel
Pipe Diameter	150mm to 900mm, and larger
Max. Pipe Wall Thickness	12.7mm steel; 25.4mm iron
Survey Speed	3m to 5m/min typical, up to 10m/min
Insertion Opening	Full Pipe Diameter
Sensor Spacing	12.7mm for Steel Pipe
Threshold for Defect Detection	20% Wall Loss
Accuracy of Defect Resolution	+/- 15% for pits; +/- 5% for general thinning
Accuracy of Defect Location	< 1.0 m

A1.1.2 EM Technology from Pure Technologies

Pure presently offers two inline RFT inspection platforms:

- PipeDiver (Figure A 3) – an inline tethered free-swimming platform that was developed for inspecting in-service pipelines. Variations of this platform include:
    - Mini PipeDiver for inspecting pipes ranging 400mm through 1200mm in diameter if the tool must pass through a butterfly valve, or as small as 300mm if the tool will not pass through butterfly valves.
    - Standard PipeDiver for inspecting pipes ranging 400mm through 1500mm.
    - Large Diameter PipeDiver for inspecting pipes 1500mm through 3000mm
- PureRobotics™ Crawler (Figure A 4) – a tethered tracked crawler platform that can inspect out-of-service pipelines 400mm diameter and larger in either dewatered or watered condition, to maximum pressure of 690 kPa (100 psi) with CCTV camera onboard, or 2415 mPa (350 psi) without the camera.

Figure A 3 - PipeDiver EM Inspection Platform (courtesy of Pure).

Both platforms utilize a single emitter coil and a multi-detector array for sensing defects and wall thinning of pipes. The number of detectors mounted in an array varies with pipe size range and is somewhat limited due to space restrictions in the assemblies. Since each detector covers a localized area of the wall immediately surrounding the sensor, all areas on the pipe circumference may not be covered (Figure A 5). Pure has recently developed a 1200mm walk-through tool that has 48 detectors in two staggered arrays, with a circumferential sensor spacing of approximately 78.5mm (Figure A 6), and is developing a tool with 256 sensors.

Figure A 4 - PureRobotics Crawler Inspection Platform (courtesy of Pure).

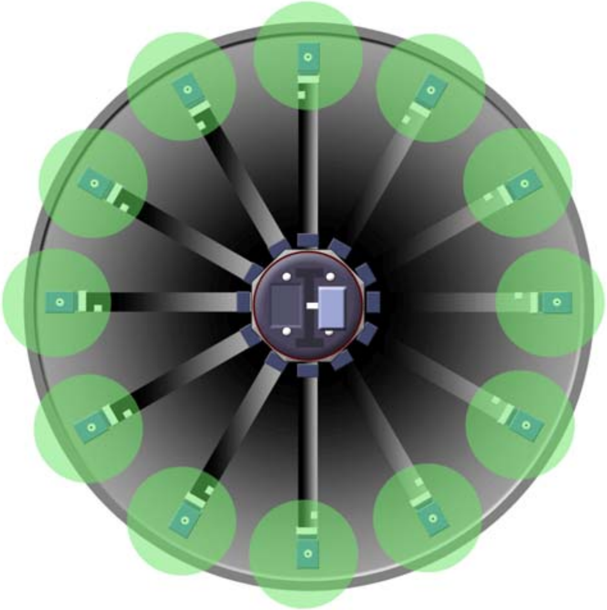


Figure A 5 - Defect Detection Zones for Pure's EM Tools.

Pure is continually developing and upgrading their inspection tools to achieve better defect resolution. Presently, all tools have a minimum of 24 sensors, and for various pipe sizes have sensor spacings approximately as follows:

- Crawler:
  - 400mm and larger pipe, with sensor spacing in 400mm pipe of approximately 49.9 mm
- PipeDiver:
  - 400 to less than 600mm, with approximate sensor spacing in 500mm pipe of 126mm
  - 600 to 900mm, with approximate sensor spacings of 78.5mm and 118mm in 600mm and 900mm pipe, respectively.

The threshold defect size (defect resolution) for earlier tools was 125mm-diameter (area 0.62m2) at 50% wall loss, which is considerably less than that afforded by PICA’s RFT tools, but Pure advises that resolution has since improved to 75 mm diameter at 30% wall loss. Additionally, the circumferential sensor spacing of Pure’s tools increases with pipe diameter, so the probability of missing defects in areas between sensors increases with pipe size. Decreased defect resolution could also be expected when the tool is not centered within the pipe.

It is also important to note that, at this time, the insertion and recovery of Pure’s EM tools requires pipelines to be temporarily taken out of service. Launching and retrieval tubes that would permit in-service insertion and recovery of the tool, similar to those used for PCCP pipeline inspections, are being developed.

Figure A 6 - Recently Developed 1200mm Pipe Inspection Tool (courtesy of Pure).

For inspection of metallic pipes, PipeDiver requires a flow velocity range of 0.15m/s to 0.45 m/s. Survey range is generally limited by pipeline flow velocity and on-board battery life, which can range from 12 hours to 72 hours. Crawler survey range is limited to cable length which typically is 1000m but can be as much as 2800m. Due to friction between the pipe and the crawler tether, crawler deployment is also limited by the cumulative change in pipeline direction that the crawler passes through: maximum is approximately 270 degrees.

The location of PipeDiver along the pipeline is typically tracked using stationary surface trackers that detect a signal emitted by the tool as it passes the tracker station. Since tracking can only be performed from the ends of the river crossing pipes, defect location would be averaged over the survey length and therefore, subject to error. Pure advises that defects can be located with an accuracy of within 0.3m longitudinally and 15 degrees circumferentially.

Operational parameters of Pure’s unmanned EM pipe inspection tools are listed in Table A 2.

Table A 2 - Operational Parameters of Pure’s Unmanned EM Inspection Tools.

Parameter	Description
Pipe Material	Steel, Ductile Iron
Pipe Diameter	300mm through 3000mm
Sensor Spacing	Varies with pipe diameter; 24 minimum
Max. Pipe Wall Thickness	12.7mm steel
Threshold for Defect Detection	30% Wall Loss
Defect Resolution	75 mm diameter at 30% Wall Loss

Parameter	Description
Accuracy of Defect Location	0.3m longitudinal, 15° circumferential
PipeDiver Requirements	
Minimum Insertion Opening	300mm for PipeDiver
Pressure Range	70 to 2070 kPa (10 to 300 psi)
Survey Speed	0.15m/s to 0.45m/s
Tool Length	3.7 to 4.6 m
Tool Weight	113 kg
Allowable Cumulative Change in Pipeline Direction (Bends)	N/A for PipeDiver; 270° for Crawler
Crawler Requirements	
Minimum Insertion Opening	450mm for Crawler
Pressure Range	0 to 2070 kPa (0 to 2415 psi)
Survey Speed	25 m/min max.
Tool Length	Varies with pipe size – 0.8 to 2.8 m

The advantages to the use of Pure’s EM technologies are similar to that of PICA’s RFT tool in that they can be deployed under either live or isolated conditions and do not require intimate contact with the pipe wall. However, the technology does not provide the accuracy and resolution of RFT technology.

A1.1.3 Bracelet Probe from PICA

The bracelet probe is an external inspection tool developed for use in detecting and quantifying wall loss in steel pipes covered with up to 50mm of pipe insulation, a condition termed “corrosion under insulation”. This technology was reviewed specifically for use on the 500 mm Baltimore Force Main installed on the St. Vital Bridge.

The Bracelet Probe Figure A 7 is a wheeled 16-sensor tool with a scan path 250mm wide (sensor spacing approximately 15.5mm), and can inspect the entire circumference of insulated pipes from 75mm diameter and larger, or bare pipes from 150mm diameter and larger, by performing multiple adjacent scans. Pipes can be scanned at a rate of up to 3.66m/min.

Force main drawings indicate the steel pipe on the bridge has a wall thickness of 9.525 mm (0.375 inch) and is insulated with 50mm of factory-applied closed-cell polyurethane insulation complete with spiral wound 22-gauge (0.86 mm) lock-seam steel cladding for moisture barrier and mechanical protection.

Unfortunately, the Bracelet Probe is unable to inspect through ferromagnetic cladding materials but can inspect through non-magnetic materials such as aluminum, some stainless steels and GRP. For areas where the ferromagnetic cladding can be removed, or that can be inspected with cladding and insulation removed, the defect resolution of the tool is as follows:

- For high lift-off application with 50 mm insulation on pipe, the Threshold of Detection (ToD) for localized wall loss (LWL) is a 38.1 mm diameter flat-bottom hole 1.905 mm deep (20% of wall thickness) on outer surface of the pipe. The sensitivity to wall loss on the inner surface of the pipe is extremely low in high lift-off applications.
- For bare pipe applications
  - ToD for LWL on the internal pipe surface is a 9.5 mm diameter flat bottom hole 1.905 mm deep (20% of wall thickness)
  - ToD for LWL on the exterior surface is 6.35 mm diameter flat bottom hole 1.905 mm deep.
  - ToD for General Wall Loss from the inner and outer surfaces is 1.43 mm (15% of wall thickness).

For bare pipe applications, pipe areas identified as having wall thinning defects could be further evaluated using a portable ultrasonic thickness gauge.

Figure A 7 - Bracelet Probe Pipe Wall Inspection System (courtesy of PICA).

A1.1.4 Magnetic Flux Leakage (MFL) Technology

MFL was the first EM technology developed to enable in-line inspection (ILI) of steel pipes for defects and wall thinning over their full circumference and surveyed length, and these are still the most commonly deployed ILI tools, primarily by the petroleum industry, for determining the physical condition of steel pipelines.

In MFL technology, a powerful magnet placed at the surface of the pipe induces a static magnetic field, or magnetic flux, within the pipe wall between the poles of the magnet (Figure A 8). 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 varies in thickness or contains surface-breaking cracks, fractures or pipe joints the magnetic field will become distorted and the flux will “leak” beyond the surface of the pipe and can be detected by sensors located near the surface between the poles of the magnet.

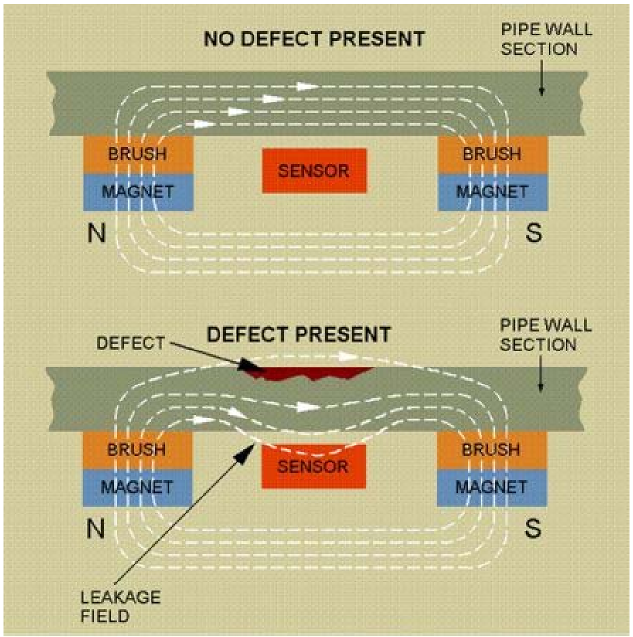


Figure A 8 - Sketch of Magnetic Flux Leakage Principle (courtesy of Pure).

The magnitude and shape of the leakage field, as measured by multiple adjoining sensors, can be used to characterize the type, size and shape of the defect. Primary and secondary sensors on the tool can discriminate between internal and external metal loss.

The sensitivity of MFL tools to detect corrosion-related wall loss and other defects in a pipe is dependent primarily on the tool’s ability to saturate the wall with magnetic flux, which is dependent on the strength of the magnets and their proximity to the pipe surface, and the thickness of the wall being magnetized. Any particular MFL tool will have an upper limit to the wall thickness that it can be used to inspect, above which the flux density in the wall, and hence the sensitivity of the tool to detect small defects, decreases.

Some tools are fitted with permanent magnets in which the strength cannot be adjusted while others are fitted with electromagnetic systems that are powered by on-board batteries. In conventional MFL tools, magnetic induction of the pipe wall is aided by stiff wire brushes that maintain contact between the magnets and the metallic pipe wall. Such tools cannot be used to inspect lined pipes; however, tools that use Hall Effect sensors can inspect internally coated and lined pipes with up to 25 mm of sensor “stand-off”, the distance of the sensors are positioned from the pipe wall.

Figure A 9 - Deployment of 600mm MFL Tool in Internally Coated Steel Watermain, Regina.

The flux may be induced in the wall in either the axial or transverse (circumferential) direction to enable detection of axial or transverse defects, such as stress- and corrosion-based cracks in girth or seam welds, and the narrow axial external corrosion (NAEC) that is associated with the failure of the external tape coatings along weld beads.

The sensitivity of the tool is also dependent on the type, orientation and number of sensors being used, and their proximity to the pipe surface, with the sensitivity of the tool decreasing as sensor stand-off increases. A variety of sensors have been used on ILI tools, including coils, Hall-effect sensors, magnetostrictive sensors, and others.

Due to the close proximity of the sensors to the pipe wall and the configuration of the magnet systems, MFL tools generally cannot pass through mitred or tight bends. Service providers advise that minimum axial radius that tools can pass is 1.5 diameters. Unfortunately, municipal watermains, force mains and sewers are typically constructed with segmented bends or cast bends having axial radii of one pipe diameter or less.

MFL surveys can locate various pipeline features and anomalies, and detect general and pitting corrosion on both internal and external pipe surfaces. MFL technology can be used to locate and quantify metal loss greater than 10% of original pipe wall thickness, but can only detect the presence of wall loss smaller than 10%.

MFL tools can effectively inspect steel pipe sizes with the following maximum wall thicknesses:

- 150mm diameter – 10mm wall
- 200mm-250mm – 12mm wall
- 300mm-1422mm – 25mm-38mm wall
- MFL pipeline inspections can be conducted under both dry and wet conditions. Conventional inspection tools have been developed for pipes ranging in size from 150mm to 1420mm. Transverse inspection tools have been developed for pipes ranging in size from 300mm to 900mm.

While MFL technology offers high accuracy in detecting internal and external defects in ferrous metal pipelines, its deployment in buried municipal pipes presents several challenges:

- The magnets require direct (or near direct) contact with the pipe surface in order to develop the magnetic flux field. Pipelines being inspected must therefore be completely free of debris and tuberculation. Conventional MFL tools that utilize brushes to contact the pipe cannot be used to inspect internally coated (lined) pipes.
- Tools require a minimum of 3m clear space in front of the insertion/extraction points to facilitate insertion and extraction. This generally would require significant civil works to construct tool access. A typical deployment is illustrated in Figure A 9.
- 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 pipe diameters; MFL tools cannot pass through mitered bends without damaging the tool and/or the pipeline.
- The mass of the magnet array and strong magnetic attraction between the tool and the pipeline make propelling tools through municipal pipelines difficult using normal line pressures. Also, since stronger magnets are needed to saturate thicker pipe walls, the force required to propel MFL tools generally increases as wall thickness increases.

Pure Technologies advised that their MFL tools are equipped with:

- Hall Effect sensors at 0.26” uniform circumferential spacing
- Sensor data sampling rates as frequent as 1403 times per second, or every 0.02 inches longitudinally at a flow rate of 2 fps
- Near-field sensors that can discriminate between Internal and External defects
- 6-axis Strap Down Inertial Sensor Integration for quality map and geographic location generation
- Geometry sensors for detecting dents, bulges, wrinkles, buckles, and other geometric anomalies.

A1.2 Remote Camera Inspection (CCTV)

When used in conjunction with pipeline cleaning equipment and procedures, remote camera inspection can provide valuable information of pipe condition, particularly when inspecting lined and submerged pipes. Visual data capture can provide excellent information on the extent of visual defects, and can utilize standardized defect coding systems such as PACP (Pipeline Assessment Certification Program) developed by WRc and NASSCO.

The visual data capture methods do not collect information beyond the visual range of the pipe wall, other than symptoms of possible structural distress such as deflection and cracking.

CCTV inspections may be carried out by either traditional tractor style deployments (e.g. sewer inspection) or through specialised inspection technologies allowing for inspection of in-service mains (e.g. Sahara). The Sahara inspection platform is discussed in further detail below.

A1.3 Sonar Technology

Since about 2011, sonar technology has been widely used in the City of Winnipeg for inspecting the interiors of sewer and force main piping for deflection defects and debris build-up. Sonar technologies are relatively easy and cost effective to deploy in live gravity wastewater applications. Deployment becomes more difficult in pressure pipes, but live deployments in conjunction with the correct pipeline access and system modifications are possible.

The technology involves physically passing a sonar transducer through the pipeline using, towed, robotic, or floating platforms. Tools can be deployed as standalone inspections or in conjunction with CCTV, LIDAR, or other technologies. Uses for sonar include:

- Snapshot of pipeline debris levels prior to cleaning and inspection with more invasive tools.
- Air pockets.
- Assessment of pipeline shape (deflection) and review for signs of structural distress, including excessive deflection, reverse curvature, and pipeline failures. See Figure A 10.

In thermoplastic pipes, which are not otherwise readily inspected for in service failure modes, SONAR can be utilized to provide insight into the pipe's in service stress levels and long term risk exposure to slow crack growth.

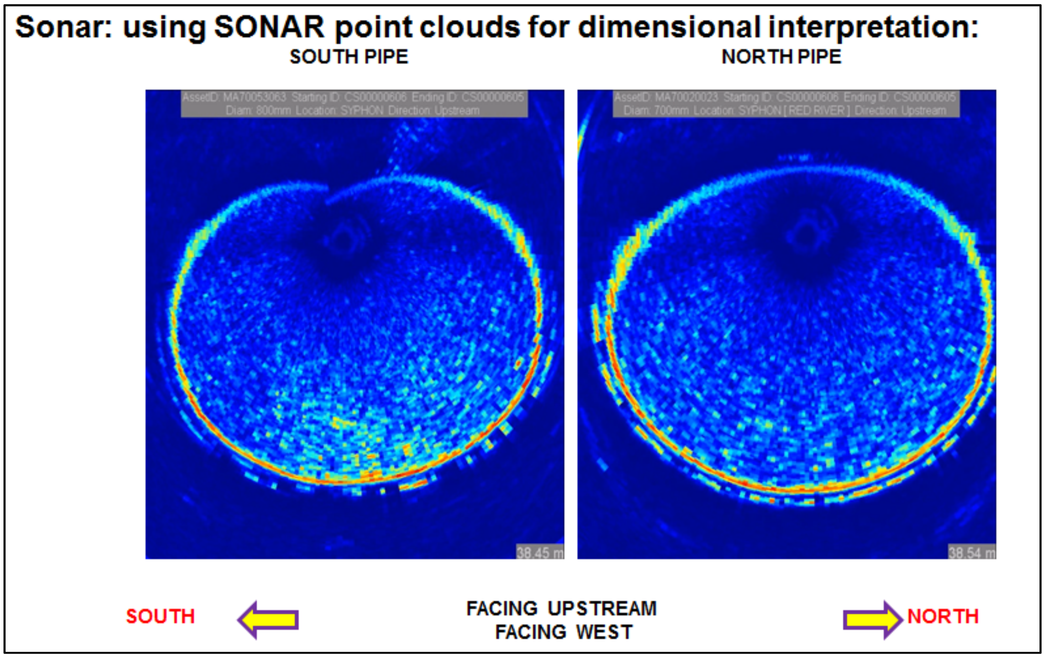


Figure A 10 - Sonar Imagery Depicting Reverse Curvature and Longitudinal Fracture of Plastic Pipe.

A1.4 Leak Detection

Several types of leak detection technologies were reviewed, including:

- SAHARA
- LDS 1000
- Investigator
- SmartBall

Both inline and online configurations of leak survey equipment are available for pipeline inspection. On-line equipment utilizes leak sensors that are attached to readily accessible features such as valves, pipes and hydrants, and uses radio equipment to convey leak signal data captured by the leak sensors to a processing unit or correlator. In-line leak survey equipment utilizes a free-swimming or tethered sensor head that travels through the inside of the pipeline, propelled by product flow. Leak data captured at the sensor head is conveyed to the analytical equipment on the surface via an electronic or fiber optic cable within the tether. On free-swimming systems, the inspection data is captured and stored at the sensor head, and is downloaded to the analytical equipment after the device is recovered from the pipeline.

Tethered in-line systems generally will be the most accurate method for pinpointing leak location in the river crossing pipes because the length of tether deployed (distance along the line from the launch location) can be accurately measured, and the leak sensor can be moved back-and-forth over the leak to pinpoint the site of greatest noise intensity. Free-swimming devices for which tool location is determined using over-the-line tracking equipment are not practical for inspecting river crossing pipes, and tools that use on-board inertial tracking systems may be subject to errors caused by tool slippage or excessive rolling. Leak detection using online systems can also be subject to error if the length pipeline between leak sensor positions is not accurately known, or if the pipeline is comprised of various and/or unknown materials.

Options for leak detection are briefly discussed below.

A1.4.1 Sahara from Pure Technologies

Sahara is an in-line multi-sensor pipe inspection system that combines high-definition CCTV, acoustic and sonde equipment in a single sensor head that is connected with an umbilical to truck-based analytical equipment. Since being introduced in 1998, Sahara has completed more than 5,600 kilometres of water pipeline inspections. Separate systems have been developed for inspecting watermains and force mains but Pure prefers to inspect the latter using their SmartBall platform.

Inspections are performed with the pipeline in service, and deploying the tool into the main is relatively simple. With the aid of a motorized insertion assembly, the sensor head and umbilical are fed into the main through a minimum 50mm (nominal) port, such as an existing air valve connection or a new purpose-installed port, and are pulled into the line by drag forces created by the fluid flowing into a drogue that is mounted on the sensor head behind the camera (Figure A 11). Acoustic and video data captured by the head as it travels through the line are relayed via the umbilical to the truck-mounted audio and video systems, which the operator continuously monitors for leak signals, and which records the CCTV data for record purposes.

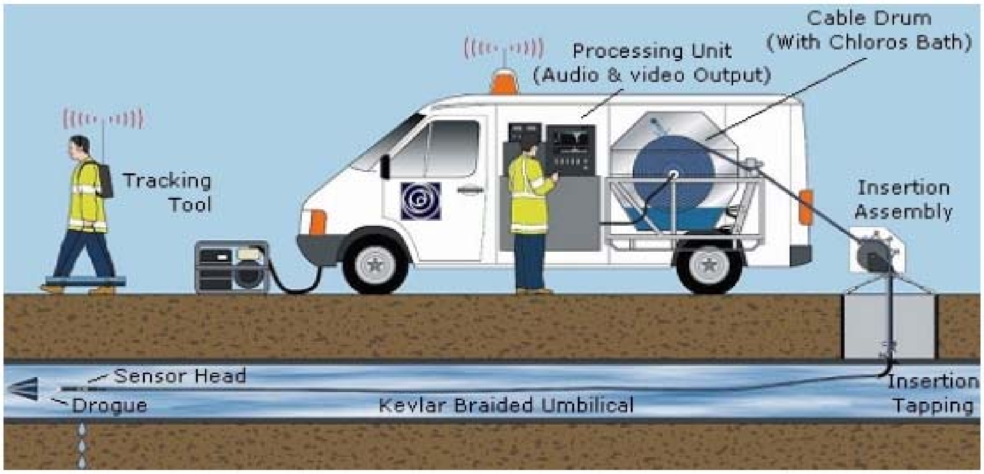


Figure A 11 - Sahara Inspection System (courtesy of Pure).

Locations of leaks are accurately determined by carefully advancing and retrieving the sensor across the leak area, zeroing-in on the intensity (loudness) of the leak signal. When the sensor is positioned exactly at the leak site, the length of umbilical deployed into the pipeline is determined from a distance counter attached to the cable winch and, where possible, the location is marked on the surface by a worker using a tracking device.

Typical umbilical length is approximately 1500m but deployment length can be increased to 3000m by connecting a second umbilical to the first. Immediately prior to inserting Sahara into the pipeline, flow velocity and direction are measured by Pure for the purposes of selecting an appropriate drogue size (diameter) for propelling the tool, and to confirm the requisite flow parameters (direction and velocity) have been achieved.

Operational parameters of the Sahara inspection system are listed in Table A 3.

Table A 3 - Various Operational Parameters of the Sahara Inspection System.

Parameter	Description
Pipe Material	All
Pipe Diameter	300mm and larger
Minimum Pipeline Pressure	21 kPa (3 psi)
Minimum Flow Velocity	0.3m per second
Insertion Port Opening	47mm (nominal 50mm-diameter)
Survey Distance per day	3.6 km/day
Allowable Cumulative Change in Pipeline Direction (Bends)	270 degrees
Leak Sensitivity	1 litre per hour at 600 kPa (87 psi) pressure
Accuracy of Leak Location	Typically within 0.5m for surface trackable inspections, but 0.1m has been achieved

A1.4.2 LDS 1000 from GAME Trenchless Consultants

LDS 1000, Figure A 12 and Figure A 13 developed by JD7 in the UK and licensed in Canada by Game Trenchless Consultants of Quebec, is a multi-sensor inspection system that is almost identical to the system Sahara except the length of tether cable available for inspection is somewhat less: 1000 metres.

GAME, therefore, is a direct competitor to Pure for inline tethered leak detection, video inspection and line location (tracing) work.

Figure A 12 - LDS 1000 Inspection System (courtesy of GAME).

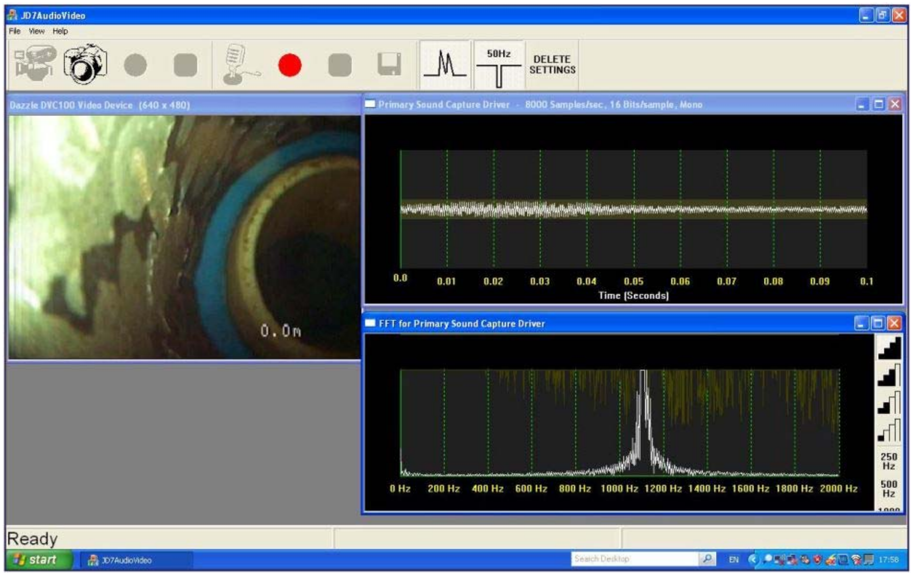


Figure A 13 - User Interface for LDS 1000 Inspection Platform (courtesy of GAME).

A1.4.3 Investigator from GAME Trenchless Consultants

Investigator, also developed by JD7 and licensed in Canada by Game Trenchless Consultants, is similar to LDS 1000 but is more portable, has a more rigid tether similar to a sewer push camera, and has a maximum deployment range of only 100 metres (Figure A 14).

This platform, therefore, is more cost effective to deploy for short inspections than either Sahara or LDS 1000.

Figure A 14 - Investigator Leak Detection System (courtesy of GAME).

A1.4.4 SmartBall from Pure Technologies

SmartBall is a free-swimming in-line leak detection system that is deployed from a launch location on the pipeline, and can travel several kilometres downstream before being retrieved at a convenient location. The device is essentially a 66mm diameter spherical aluminum sensor core that placed inside a lightweight and permeable foam ball (Figure A 15). The core contains a variety of electronic systems including the acoustic sensor, pressure sensors for detecting pressure changes, inertial tracking system for determining ball location and for pipeline mapping, a datalogger and the power cell. The foam cover is sized to suit the diameter and flow of the pipeline being surveyed, and its diameter typically will be less than one-third of the pipe diameter.

SmartBall requires a minimum 100mm-diameter port or opening for both insertion and retrieval, and clearance is required for the attachment of the launch and retrieval equipment. For the river crossing pipes it may be possible to insert this device into the pipeline at a pump station or valve chamber, and retrieve it from a downstream valve chamber or force main discharge manhole.

SmartBall gains its mobility from the fluid that flows through the pipeline and moves the ball along the invert (Figure A 16). The device can negotiate valves and inclines, and at a flow velocity of 5.5 m/s can traverse vertical piping. Due to the size of the core, the device is limited to surveying pipes 300mm-diameter and larger.

As the ball travels through the pipeline, the electronics sample acoustic signals thousands of times per second and store the data in the on-board datalogger. Other data recorded includes time, ball rotation (inertial tracking system) and acoustic signals received from transponders attached to the exterior of the pipeline along the survey route.

After the ball is recovered from the pipeline, the data is downloaded and analyzed to determine the presence and location of significant acoustic events. Company literature indicates the device is capable of detecting leakage as little as 0.5 litres per minute, but Pure advises the device can detect smaller leaks. Pure claims the accuracy of leak positioning is within 1m when using transponders, and they generally recommend that transducers be installed at 1km intervals. The use of transponders at closer intervals would tend to improve survey accuracy.

Operational parameters of the SmartBall leak detection system are listed in Table A 4.

Figure A 15 - SmartBall Leak Detection System (courtesy of Pure).

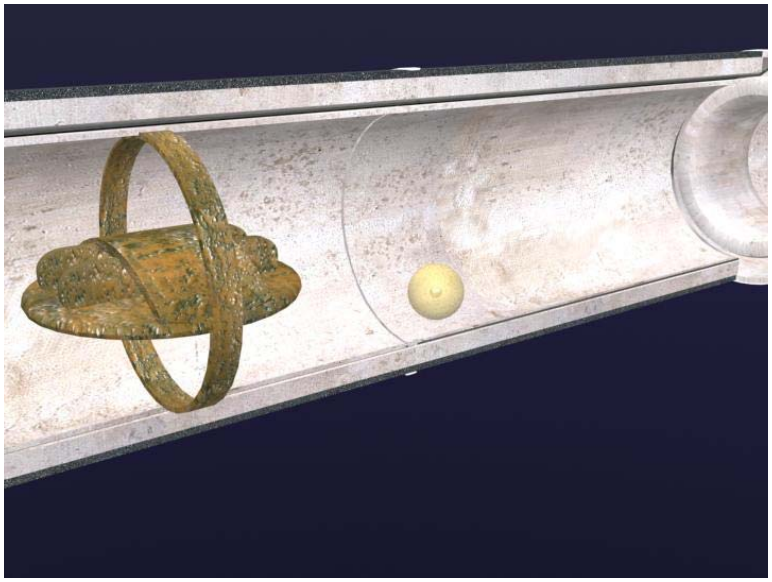


Figure A 16 - Rendering of SmartBall Leak Detection System (courtesy of Pure).

Table A 4 - Various Operational Parameters of the SmartBall Leak Detection System.

Parameter	Description
Pipeline Material	All Types
Pipeline Diameter	300mm and Greater
Pipeline Pressure	140 kPa to 2000 kPa (20 psi to 290 psi)
Flow Velocity	0.5 m/s to 5.5 m/s
Insertion Point Bore	100mm Diameter Port or Greater
Survey Distance per Insertion	Varies with flow; battery life of 12 hours At velocity of 0.5m/s, almost 20km
Allowable Cumulative Change in Pipeline Direction	Not applicable
Leak Sensitivity	0.5 L/minute at a line pressure of 413 kPa (60 psi)
Accuracy of Leak Location	Using transponders, 1 metre Without transponders, 1% of total survey distance

A1.5 Ultrasonic Technology (UT)

Developed for inspecting oil and gas pipelines (Figure A 17), inline ultrasonic tools have yet to be adapted for municipal pipe inspections; however, a consortium of water agencies including Yorkshire Water is currently working with an inspection contractor in the UK (JD7) to develop an inline ultrasonic tool for accurately detecting and measuring pipe wall thickness, dents and deflection for watermains. The greatest setback to the development of this tool is reportedly an inability to maintain the tool position in the center of the pipeline. Pure Technologies is also developing an ultrasonic inspection tool, and is presently negotiating with the City of Hamilton to perform a pilot inspection when the tool is ready.

Figure A 17 - Inline Ultrasonic Inspection Tool for Gas Pipelines (courtesy of Dacon).

In ultrasonic pipe inspection tools, a transducer (combined transmitter / receiver of ultrasound frequencies) directs ultrasonic pulses into the pipe being tested. The pulses are reflected back from the front and rear pipe wall surfaces and from any discontinuities that may be encountered, such as linings. The time taken for the various echoes to return to the transducer can be used to determine the distances between the reflecting surfaces and the transducer. Pipe inspection tools often have several transducers spaced around the circumference of the pipe.

Ultrasonic pipe inspection tools can accurately detect and measure metal loss (internal and external) and mid-wall anomalies such as laminations, inclusions and cracks. Achieving high accuracy of inspection requires that these transducers remain a fixed distance from the pipe surface, no matter how rough the interior surface of the pipe since an increase over the nominal tool stand-off would indicate that metal has been lost from the inner surface. Tool resolution for steel pipes is typically 10% of pipe wall thickness.

Although inline platforms for inspecting municipal piping are currently under development, portable ultrasonic thickness gauges are available for accurately measuring wall thickness from the exterior (or interior) of an exposed pipe. This type of inspection can be used to inspect large areas of pipeline by means of taking individual spot measurements throughout the inspection area. Although the measurement of wall thickness can be very precise, the process is time consuming and the accuracy of the condition assessment is dependent on the number of measurements made. Thus, they are more conducive to spot checks for confirmation of defects identified by other tools. Figure A 18 depicts the process of inspecting large pipe areas using portable a UT inspection gauge.

Few tools are available for assessing the material properties of HDPE pipes in-situ. Ultrasonic pipe inspection tools can be used to measure pipe wall thickness, but they provide no information regarding the quality of the HDPE material and its degradation. Ultrasonic flaw detection tools can be used to examine HDPE butt-fusion welds for defects, but these are typically used from the exterior of the pipeline during construction, and in-line tools for in-situ inspection of pipelines have not been developed.

Figure A 18 - UT Inspection of 400mm Iron Pipe.

Appendix F

Pipeline Modification Record Drawings

THE CITY OF WINNIPEG

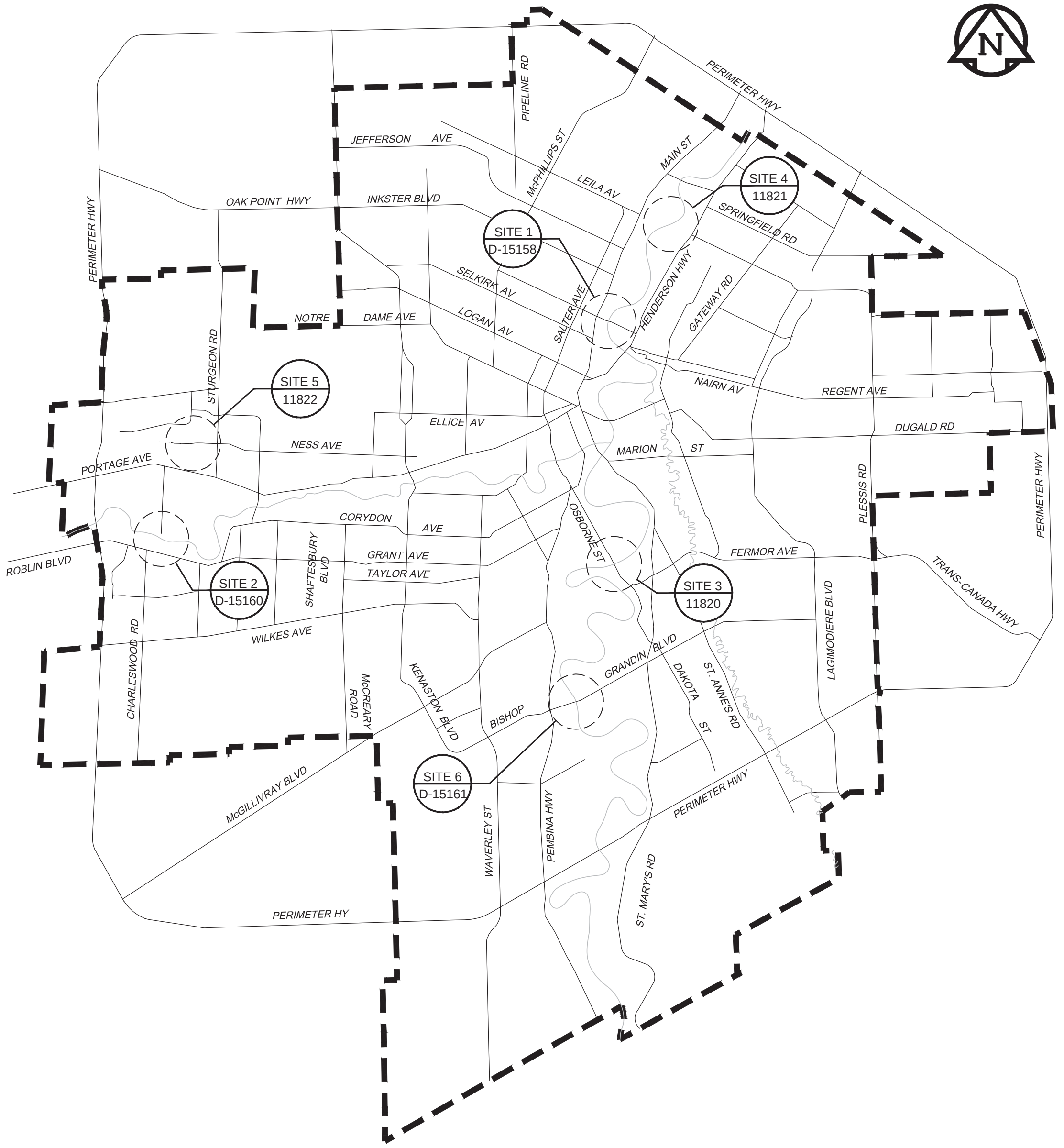
WATER AND WASTE DEPARTMENT

PROVISION OF PIPELINE ACCESS MODIFICATIONS, CLEANING AND SUPPORT SERVICES FOR RIVER CROSSING INSPECTIONS - PHASE TWO

BID OPPORTUNITY NO. 492-2018

LIST OF PROJECT DRAWINGS

CITY DRAWING No.	CONSULTANT DRAWING No.	DESCRIPTION
D-15157	G-0001	COVER SHEET
D-15158	C-1001	SITE 1 - KILDONAN - REDWOOD FEEDER MAIN - WEST CHAMBER
D-15159	C-1001	SITE 1 - KILDONAN - REDWOOD FEEDER MAIN - EAST CHAMBER
D-15160	C-1002	SITE 2 - CHARLESWOOD - ASSINIBOIA FEEDER MAIN
11820	C-1003	SITE 3 - ST. VITAL BRIDGE FORCE MAIN
11821	C-1004	SITE 4 - NEWTON AVENUE FORCE MAIN
11822	C-1005	SITE 5 - HERITAGE PARK FORCE MAIN
D-15161	C-1006	SITE 6 - FORT GARRY - ST. VITAL FEEDER MAIN



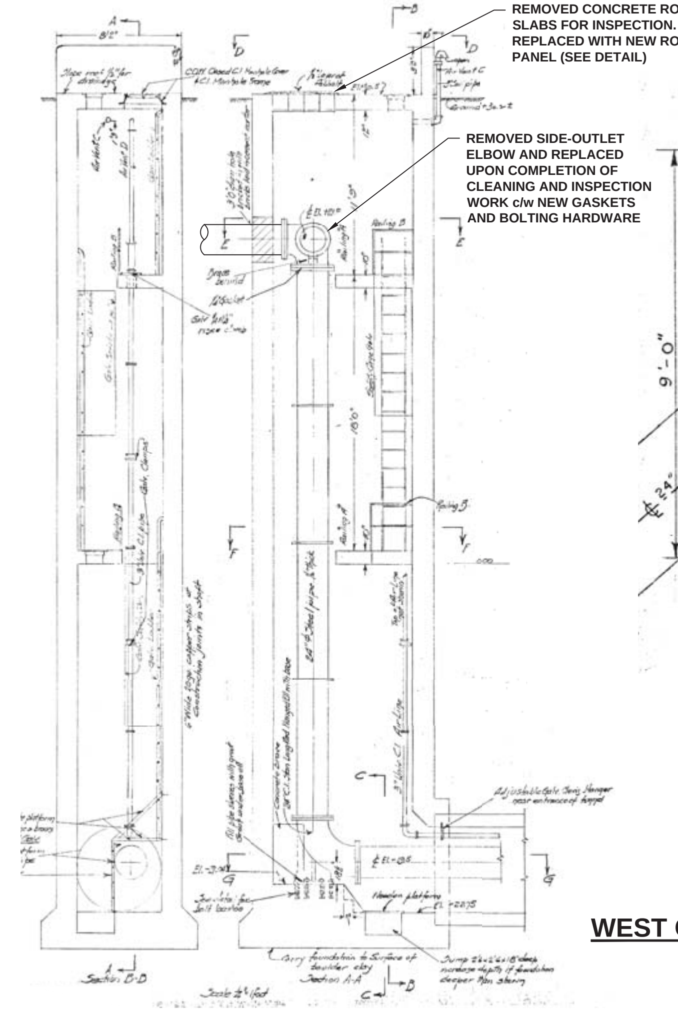
RECORD DRAWING

City Drawing Number D-15157

D SIZE 22" x 34" (559mm x 864mm)

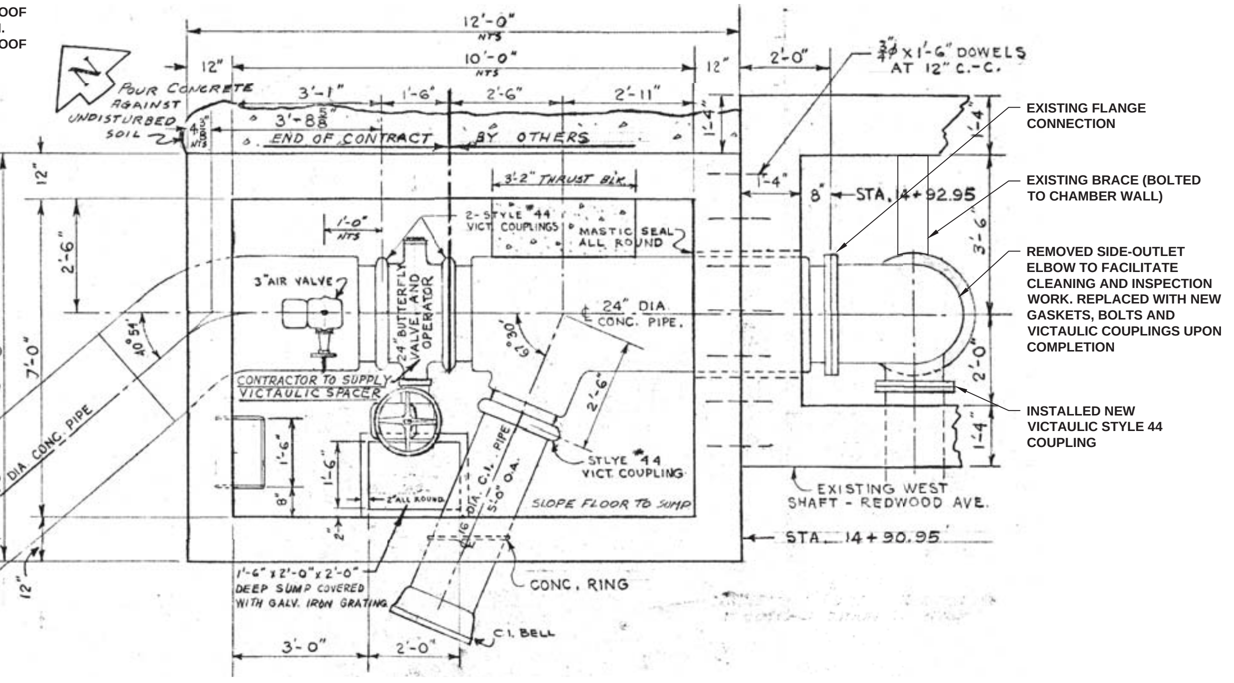
PLOT: 11/14/2019 10:04:07 AM

AECOM FILE NAME: 60549028-SHT-30-0000-G-0001.dwg Saved By: cooperkl



WEST CHAMBER SECTION

Scale 1:75



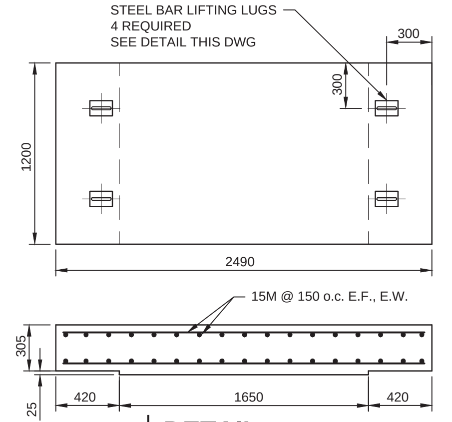
WEST CHAMBER PLAN

Scale 1:25

SITE PLAN

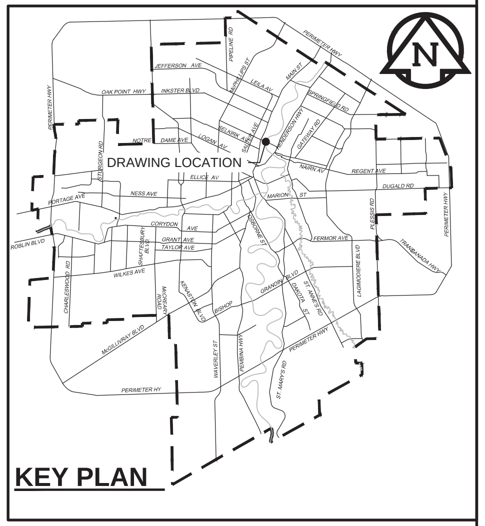
Scale 1:1500

0 37.5 75 m

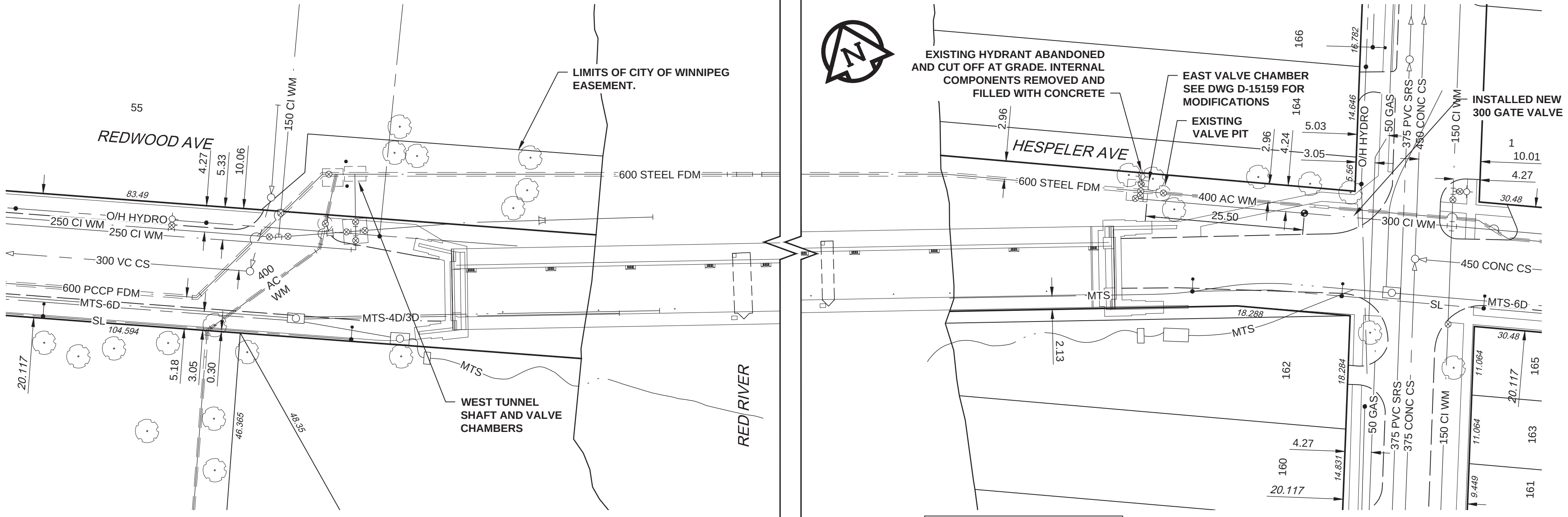


DETAIL

REMOVABLE CONCRETE ROOF PANEL PLAN & SECTION SCALE 1:25



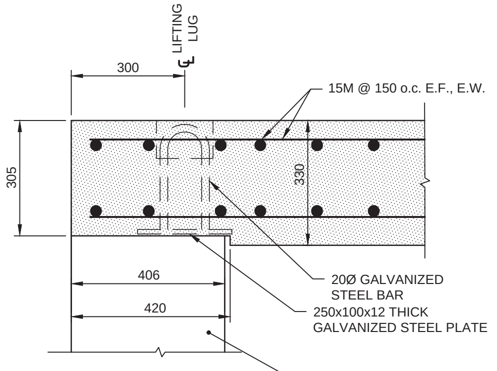
KEY PLAN



PLAN

Scale 1:500

0 12.5 25 m



DETAIL

REMOVABLE ROOF PANEL LIFTING LUGS (4 REQUIRED) SCALE 1:10

NOTES:

SEE NEXT SHEET, DWG D-15159 FOR SEQUENCE OF WORK AND CONSTRUCTION NOTES

ENGINEERS GEOSCIENTISTS MANITOBA
Certificate of Authorization
AECOM Canada Ltd.
No. 4671 Date:____________

CONSTRUCTION COMPLETION DATE:
NOVEMBER 8, 2019

METRIC
WHOLE NUMBERS INDICATE MILLIMETRES
DECIMALIZED NUMBERS INDICATE METRES

RECORD DRAWING

BID OPPORTUNITY NO. 492-2018

EXISTING	LEGEND - PLAN	NEW
150 WM	WATERMAIN	150 WM
	HYDRANT	
⊗	VALVE	
300 LDS	LAND DRAINAGE SEWER	300 LDS
250 WWS	WASTE WATER SEWER	250 WWS
○	MANHOLE	●
□	CATCH BASIN	■
△	CURB INLET	▲
	CULVERT	
	PIPE ABANDONMENTS	
	SURVEY BAR	

EXISTING	LEGEND - PLAN	NEW
	CURB STOP	
	REDUCER	
	COUPLING	
	ANODE	
	HYDRO	
	MTS	
	GAS	
	TESTHOLE	
	LAMP STANDARD	
	TREE	

EXISTING	LEGEND - PROFILE	NEW
150 WM	WATERMAIN	150 WM
	HYDRANT	
	VALVE	
300 LDS	LAND DRAINAGE SEWER	300 LDS
250 WWS	WASTE WATER SEWER	250 WWS
	PAVEMENT CROWN	
	N/W PROPERTY LINE	
	S/E PROPERTY LINE	
	N/W GUTTER	
	S/E GUTTER	

LOCATION APPROVED
UNDERGROUND STRUCTURES

SUPR. U/G STRUCTURES COMMITTEE DATE

NOTE:
LOCATION OF UNDERGROUND STRUCTURES AS SHOWN ARE BASED ON THE BEST INFORMATION AVAILABLE. BUT NO GUARANTEE IS GIVEN THAT ALL EXISTING UTILITIES ARE SHOWN OR THAT THE GIVEN LOCATIONS ARE EXACT. CONFIRMATION OF EXISTENCE AND EXACT LOCATION OF ALL SERVICES MUST BE OBTAINED FROM THE INDIVIDUAL UTILITIES BEFORE PROCEEDING WITH CONSTRUCTION.

NO.	REVISIONS	YY/MM/DD	BY
1	ISSUED FOR RECORD	19/11/28	KLC
0	ISSUED FOR CONSTRUCTION	18/06/01	KLC

AECOM

DESIGNED BY NJK
CHECKED BY
DRAWN BY KLC
APPROVED BY
HOR. SCALE AS NOTED
VERT. SCALE N/A
RELEASED FOR CONSTRUCTION
DATE
DATE

PROFESSIONAL'S SEAL
A. D. BRAUN Member 24453

CONSULTANT DRAWING NO.
60549028-SHT-30-0000-C-1001

Winnipeg
THE CITY OF WINNIPEG
WATER AND WASTE DEPARTMENT

PROVISION OF PIPELINE ACCESS MODIFICATIONS, CLEANING AND SUPPORT SERVICES FOR RIVER CROSSING INSPECTIONS - PHASE TWO

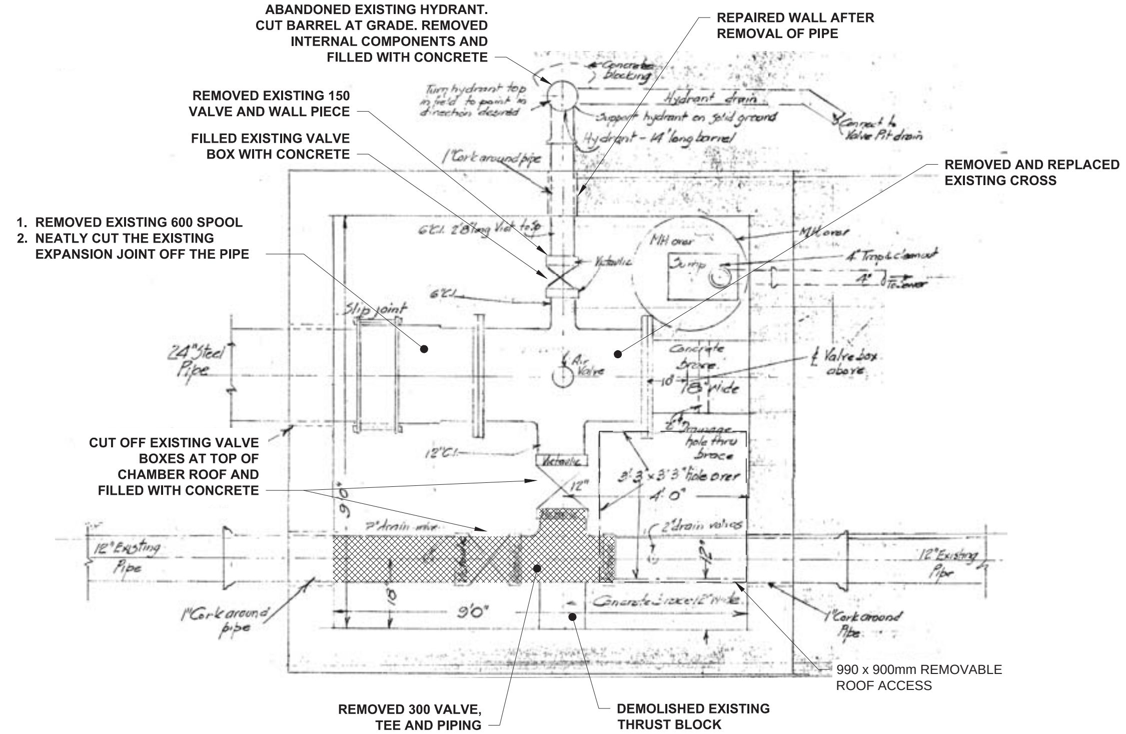
SITE 1

KILDONAN - REDWOOD
FEEDER MAIN - WEST CHAMBER

SHEET 1 OF 7

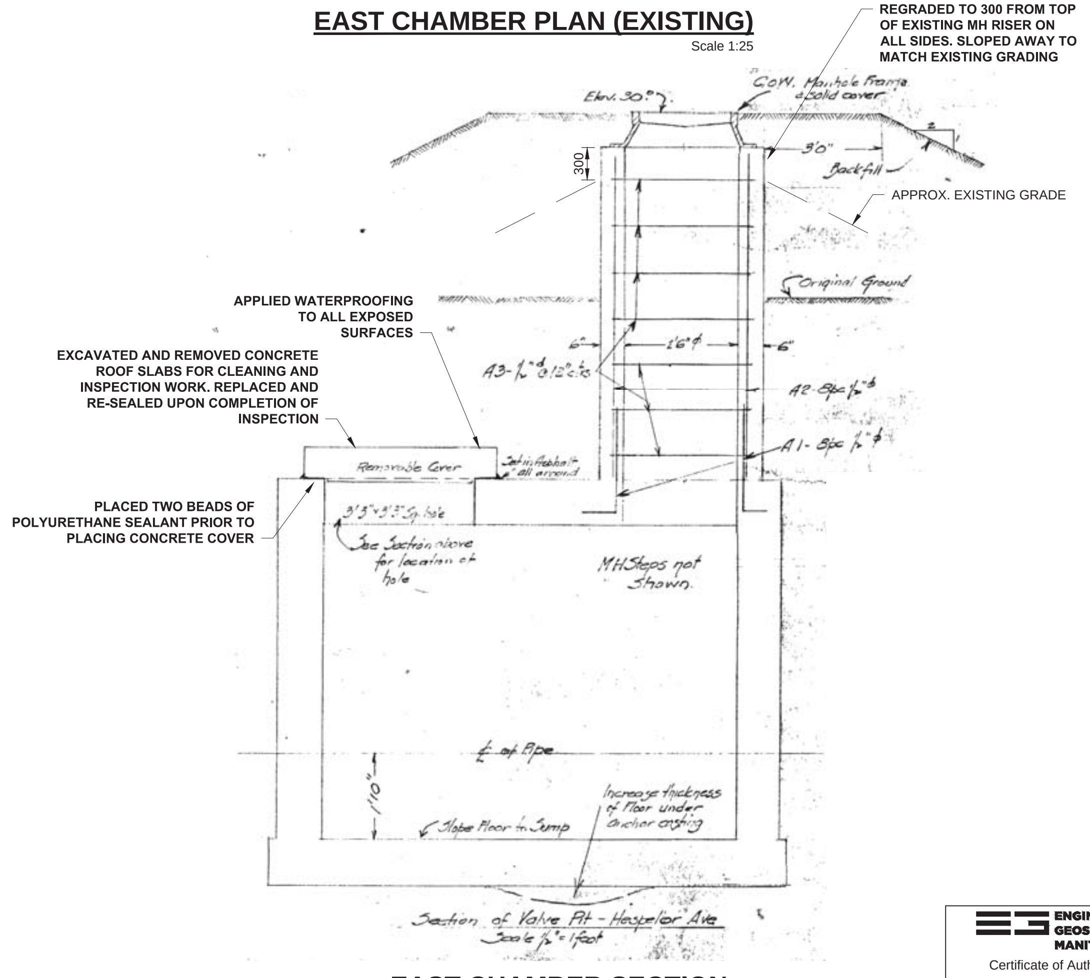
CITY DRAWING NUMBER
D-15158
REV 1

AECOM REVIEW DRFT CHK



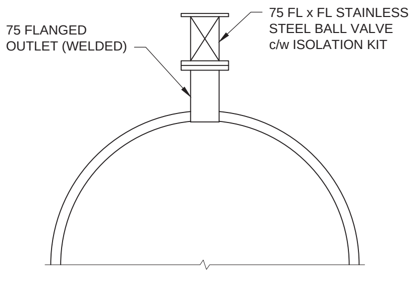
EAST CHAMBER PLAN (EXISTING)

Scale 1:25



EAST CHAMBER SECTION

Scale 1:25

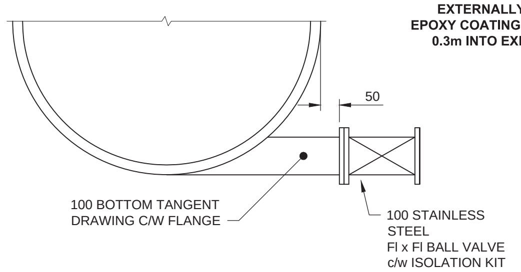


1 | DETAIL

BLOW OFF ASSEMBLY

0 250 500 mm

1:10

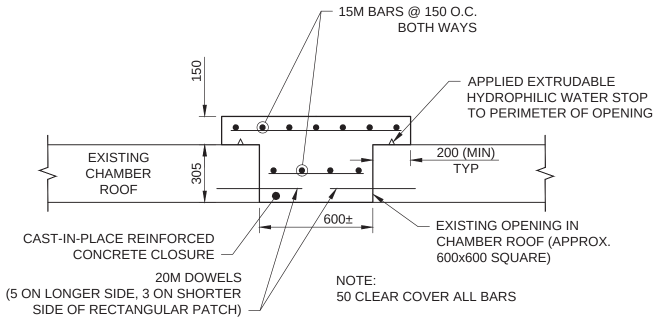


2 | DETAIL

DRAIN ASSEMBLY

0 250 500 mm

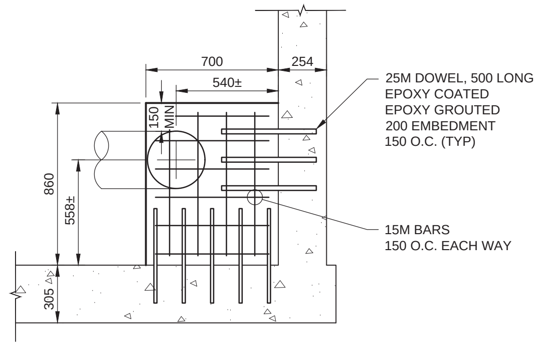
1:10



3 | ROOF CLOSURE DETAIL

-

Scale 1:20

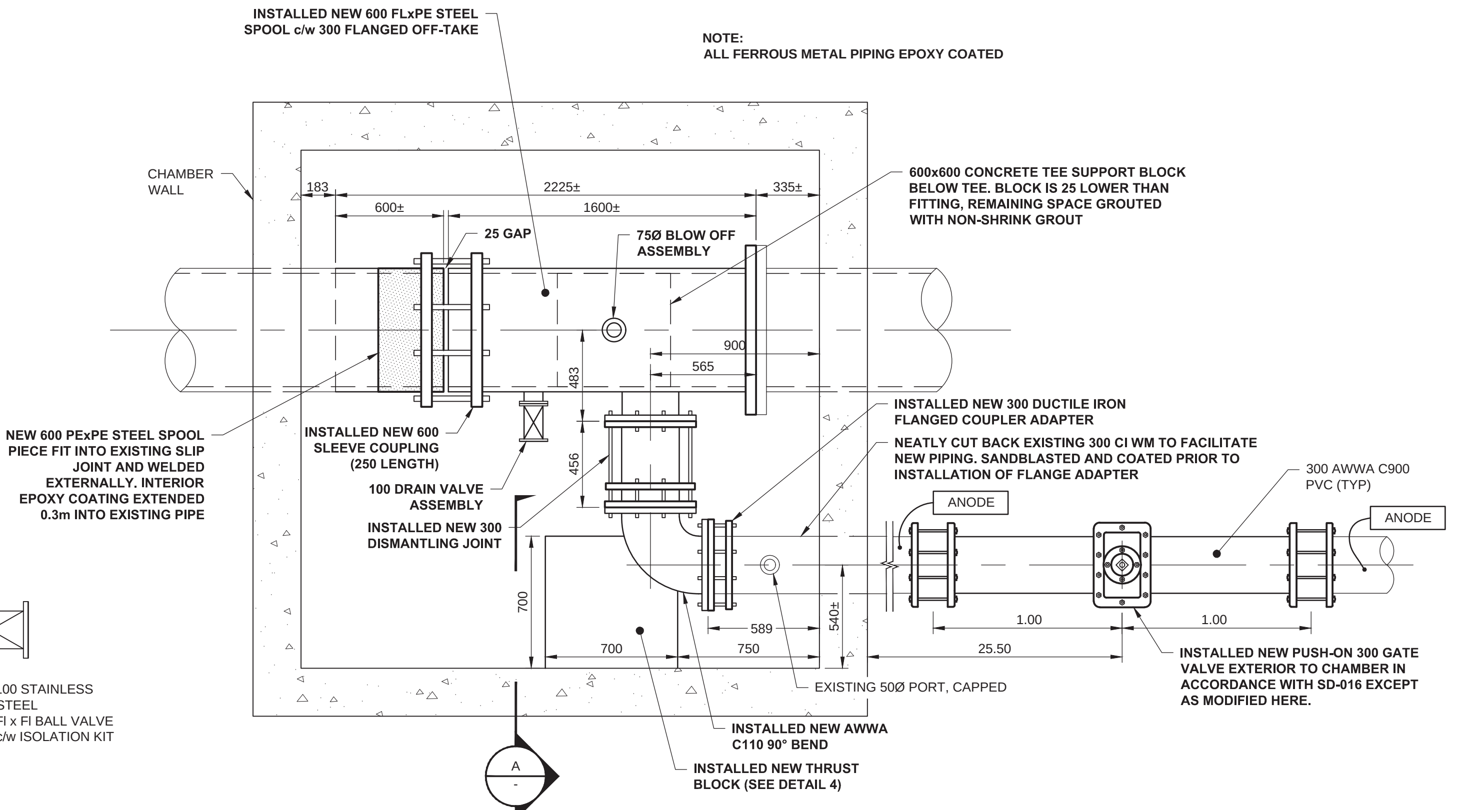


4 | THRUST BLOCK DETAIL

-

SECTION A

SCALE 1:20



EAST CHAMBER PLAN (NEW)

Scale 1:20

MATERIAL LIST

ITEM	MANUFACTURER	MATERIAL SPECIFICATION	SPECIAL FEATURES
600 STEEL SPOOL PIECE	ABCO	ASTM A53, AWWA C200 FIELD WELDED TO AWWA C206-17 INT/EXT EPOXY COATED TO AWWA C210-15	
600 STEEL SLEEVE COUPLING	ROBAR INDUSTRIES LTD	ASTM A36 End rings: ASTM A576, Grade 1020 / A36 NRB gaskets: ASTM D2000, NSF 61 Approved Fasteners: AWWA C111 Coatings: Fusion bonded epoxy coated per AWWA C-213, NSF 61 Approved	
600 STEEL FL-PE SPOOL PIECE	ABCO	ASTM A53, AWWA C200 75 mm flanged outlet – ASTM A105 (flanges to ANSI B16.5) 100 mm flanged off-take – ASTM A105 (flanges to ANSI B16.5) 300 mm flanged off-take – ASTM A105 (flanges to AWWA C207-18) Interior and exterior of spool piece epoxy coated to AWWA C210-15	
300 DUCTILE IRON DISMANTLING JOINT	ROBAR INDUSTRIES LTD	AWWA C219 Flanged body: ASTM A536-05 65-45-12 End ring: ASTM A536 60-40-18 Gasket: SBR per ASTM D2000 Tie rods: ASTM A193 B8M Class 2 Coating: fusion bonded epoxy per AWWA C213, NSF 61 Approved	
300 90-DEGREE CAST IRON ELBOW	METALFIT INC	ANSI/AWWA C110/A21.10 ANSI B16.1, ASTM A 536	
300 DUCTILE IRON FLANGE COUPLER ADAPTER	ROBAR INDUSTRIES LTD	ASTM A536 Grade 65-45-12, AWWA C219 End ring: Cast ductile iron per ASTM A536 Grade 65-45-12 Coating: fusion bonded epoxy per AWWA C213, NSF 61 Approved	
PVC PIPE (300)	IPEX INC	AWWA C900 CSA B137.3	"BLUE BRUTE" DR-18
300 GATE VALVE	MUELLER	AWWA C509 MODEL A-2360-40	RESILIENT WEDGE GATE VALVE CW TO OPEN (RED TOP)
ANODES - ZINC	BREN TECHNOLOGIES INC	ASTM B418	24 LB ZINC ANODE
300 IRON COUPLING	ROMAC INDUSTRIES INC	ASTM A 536 GRADE 65-45-12 AWWA C219, AWWA C213 MODEL "MACRO"	FBE COATED D.I. SLEEVE & END RINGS TYPE 30455 NUTS, BOLTS, WASHERS AND DI-ELECTRIC WASHERS
VALVE BOX UPPER CASING	TITAN FOUNDRY LTD.	C.I. - ASTM A-48 CLASS 30B CONFORMS TO AP-001	CASING MARKED "TITAN" & "VB4" LID MARKED "TITAN & "W"
VALVE STEM EXTENSION	TITAN FOUNDRY LTD.	CONFORMS TO AP-002	
VALVE BOX LOWER CASING	IPEX INC	AWWA C900 CSA B137.3 CONFORMS TO AP-001	PVC DR-18
100 STAINLESS STEEL BALL VALVE	M.A. STEWART & SONS LTD	ANSI B16.34 CLASS 150	
75 STAINLESS STEEL BALL VALVE	M.A. STEWART & SONS LTD	ANSI B16.34 CLASS 150	

CONTRACTOR: J-CON CIVIL LTD

CONSTRUCTION PERIOD: SEPT. 14, 2018 to NOV. 8, 2019

CONSTRUCTION COMPLETION DATE:
NOVEMBER 8, 2019

RECORD DRAWING

BID OPPORTUNITY NO. 492-2018

METRIC
WHOLE NUMBERS INDICATE MILLIMETRES
DECIMALIZED NUMBERS INDICATE METRES

ENGINEERS GEOSCIENTISTS MANITOBA
Certificate of Authorization
AECOM Canada Ltd.
No. 4671 Date:____________

EXISTING	LEGEND - PLAN	NEW	EXISTING	LEGEND - PLAN	NEW	EXISTING	LEGEND - PROFILE	NEW
150 WM	WATERMAIN	150 WM	♂	CURB STOP	●	150 WM	WATERMAIN	150 WM
⌖	HYDRANT	●	⊳	REDUCER	▶	†	HYDRANT	+
⊗	VALVE	●	×	COUPLING	×	X	VALVE	X
300 LDS	LAND DRAINAGE SEWER	300 LDS	⊋	ANODE	⊋	300 LDS	LAND DRAINAGE SEWER	300 LDS
250 WWS	WASTE WATER SEWER	250 WWS	—	HYDRO		250 WWS	WASTE WATER SEWER	250 WWS
○	MANHOLE	●	—	MTS		—	PAVEMENT CROWN	—
□	CATCH BASIN	■	—	GAS		◇	N/W PROPERTY LINE	
△	CURB INLET	▲	⊕	TESTHOLE		○	S/E PROPERTY LINE	
⊏ ═ ═ ⊐	CULVERT	⊏ ═ ═ ⊐	•—•	LAMP STANDARD		□	N/W GUTTER	
—∃	PIPE ABANDONMENTS	—∃	⊙	TREE		—○—	S/E GUTTER	
⌖	SURVEY BAR							

LOCATION APPROVED
UNDERGROUND STRUCTURES

SUPR. U/G STRUCTURES COMMITTEE DATE

NOTE:
LOCATION OF UNDERGROUND STRUCTURES AS SHOWN ARE BASED ON THE BEST INFORMATION AVAILABLE. BUT NO GUARANTEE IS GIVEN THAT ALL EXISTING UTILITIES ARE SHOWN OR THAT THE GIVEN LOCATIONS ARE EXACT. CONFIRMATION OF EXISTENCE AND EXACT LOCATION OF ALL SERVICES MUST BE OBTAINED FROM THE INDIVIDUAL UTILITIES BEFORE PROCEEDING WITH CONSTRUCTION.

NO.	REVISIONS	YY/MM/DD	BY
1	ISSUED FOR RECORD	19/11/28	KLC
0	ISSUED FOR CONSTRUCTION	18/06/01	KLC

AECOM

DESIGNED BY	NJK	CHECKED BY	
DRAWN BY	KLC	APPROVED BY	
HOR. SCALE	AS NOTED	RELEASED FOR CONSTRUCTION	
VERT. SCALE	N/A		
DATE		DATE	

PROFESSIONAL'S SEAL
A. D. BRAUN
Member 24453

CONSULTANT DRAWING NO.
60549028-SHT-30-0000-C-1001

THE CITY OF WINNIPEG
WATER AND WASTE DEPARTMENT

PROVISION OF PIPELINE ACCESS MODIFICATIONS, CLEANING AND SUPPORT SERVICES FOR RIVER CROSSING INSPECTIONS - PHASE TWO

SITE 1

KILDONAN - REDWOOD
FEEDER MAIN - EAST CHAMBER

SHEET 2 OF 7

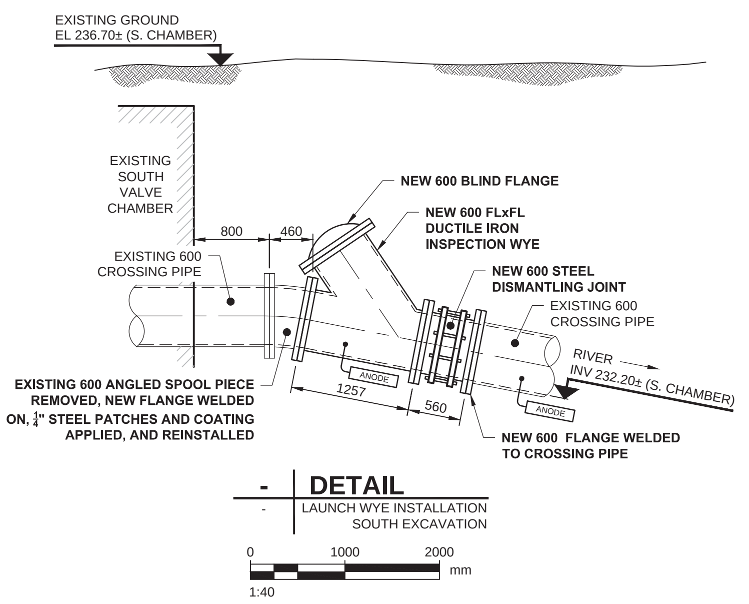
CITY DRAWING NUMBER
D-15159

REV 1

D SIZE 22" x 34" (559mm x 864mm)

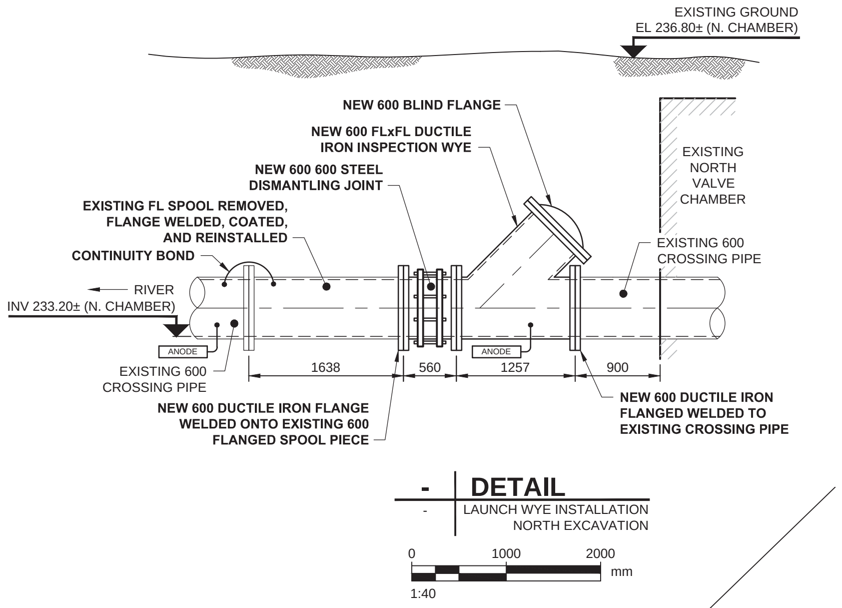
PLOT: 11/27/2019 4:38:19 PM

AECOM FILE NAME: 60549028-SHT-30-0000-C-1002.dwg Saved By: cooperkl



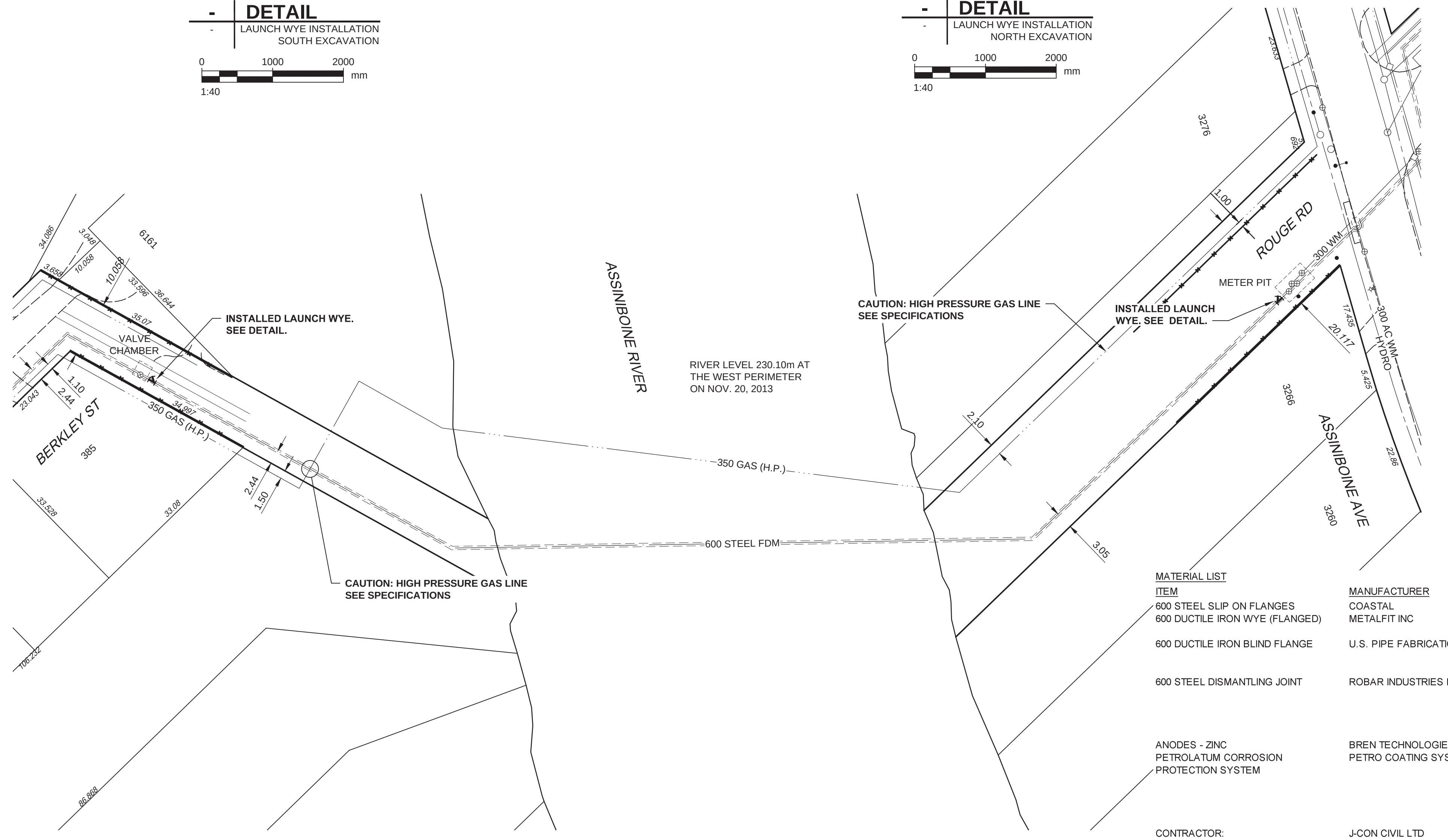
- DETAIL

- LAUNCH WYE INSTALLATION SOUTH EXCAVATION



- DETAIL

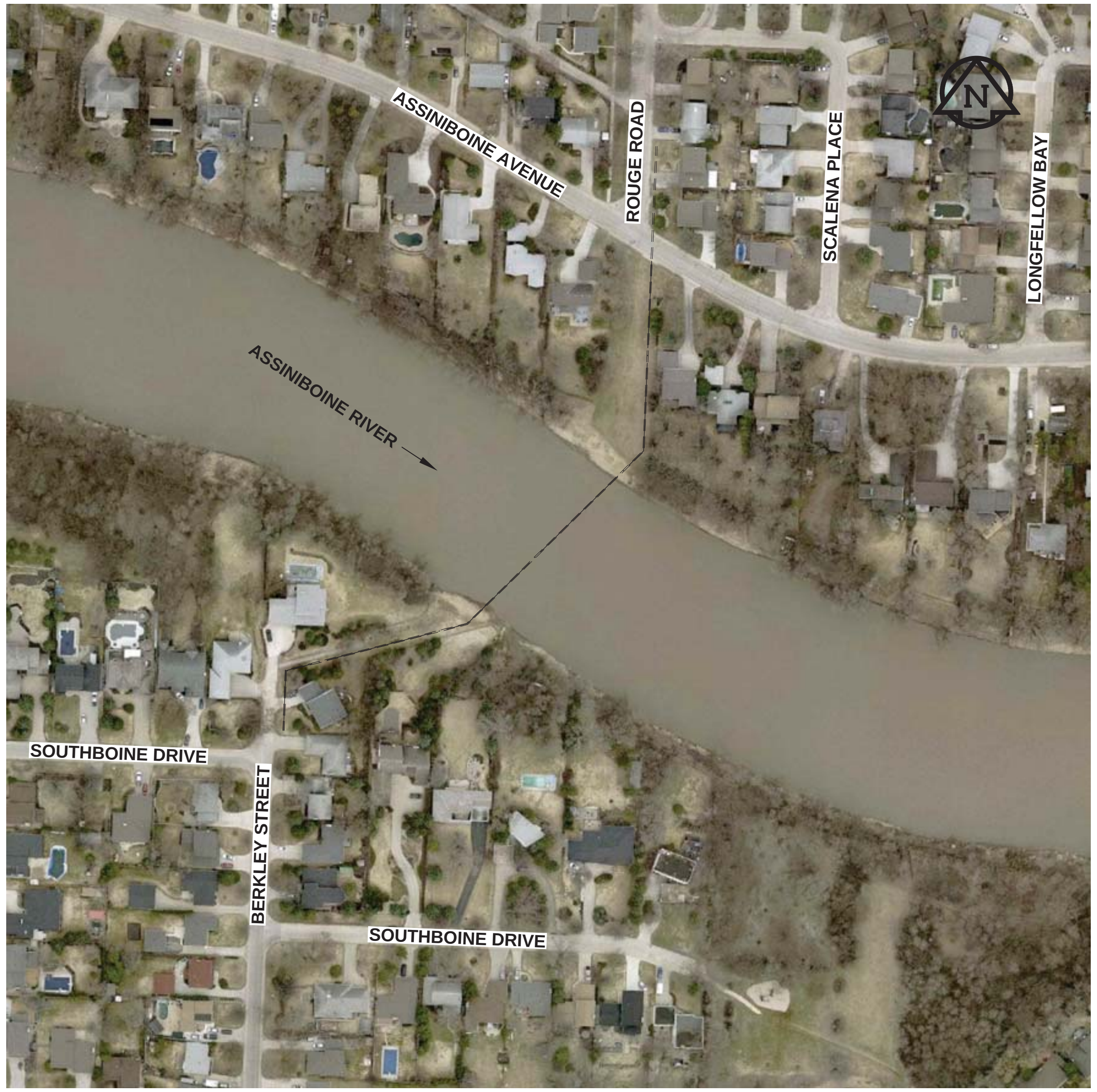
- LAUNCH WYE INSTALLATION NORTH EXCAVATION



PLAN

Scale 1:500

0 12.5 25 m



SITE PLAN

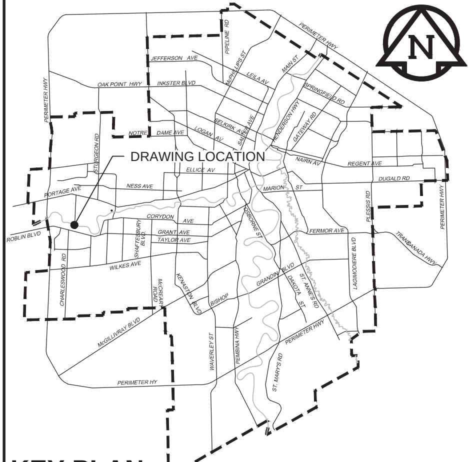
Scale 1:1500

0 37.5 75 m

MATERIAL LIST

ITEM	MANUFACTURER	MATERIAL SPECIFICATION	SPECIAL FEATURES
600 STEEL SLIP ON FLANGES	COASTAL	AWWA C207	CLASS D
600 DUCTILE IRON WYE (FLANGED)	METALFIT INC	ANSI/AWWA C110/A21.10 ANSI B16.1, ASTM A 536	
600 DUCTILE IRON BLIND FLANGE	U.S. PIPE FABRICATION	ASME/ANSI B16.1 Class 125 Fusion bonded epoxy coating in accordance with AWWA C116	
600 STEEL DISMANTLING JOINT	ROBAR INDUSTRIES LTD	Meets AWWA C207 & C219 Barrel: ASTM A36 End rings: ASTM A576, Grade 1020 / A36 Gaskets: NBR per ASTM D2000 Fusion epoxy coated per AWWA C213	
ANODES - ZINC	BREN TECHNOLOGIES INC	ASTM B418	24 LB ZINC ANODE
PETROLATUM CORROSION PROTECTION SYSTEM	PETRO COATING SYSTEMS LTD	AWWA C217	INCLUDES: LOW TEMPERATURE PETROLATUM TAPE VOID-FILLING MASTIC FILLER PETROLATUM PASTE PRIMER PROTECTIVE OUTERWRAP

CONTRACTOR: J-CON CIVIL LTD
CONSTRUCTION PERIOD: SEPT. 14, 2018 to NOV. 8, 2019



KEY PLAN

WARNING

IF POWER EQUIPMENT OR EXPLOSIVES ARE TO BE USED
FOR EXCAVATION ON THIS PROJECT, THE CONTRACTOR MUST:
1) NOTIFY THE GAS COMPANY OF THE PROPOSED LOCATION OF EXCAVATION.
2) TAKE PRECAUTION TO AVOID DAMAGE TO GAS COMPANY INSTALLATIONS.
SEE PROVINCIAL REGULATION 140/92 FOR DETAILS

ENGINEERS GEOSCIENTISTS MANITOBA
Certificate of Authorization
AECOM Canada Ltd.
No. 4671 Date:____________

METRIC
WHOLE NUMBERS INDICATE MILLIMETRES
DECIMALIZED NUMBERS INDICATE METRES

CONSTRUCTION COMPLETION DATE:
NOVEMBER 8, 2019

RECORD DRAWING

BID OPPORTUNITY NO. 492-2018

EXISTING	LEGEND - PLAN	NEW
150 WM	WATERMAIN	150 WM
⌖	HYDRANT	●
⊗	VALVE	●
300 LDS	LAND DRAINAGE SEWER	300 LDS
250 WWS	WASTE WATER SEWER	250 WWS
○	MANHOLE	●
□	CATCH BASIN	■
△	CURB INLET	▲
⊏-----⊐	CULVERT	⊏-----⊐
——⊐	PIPE ABANDONMENTS	——⊐
⌖	SURVEY BAR	

EXISTING	LEGEND - PLAN	NEW
♂	CURB STOP	●
— · ▷ · —	REDUCER	—▶—
— · ✕ · —	COUPLING	—✕—
⊋	ANODE	⊋
— – ——	HYDRO	
——··——	MTS	
——···——	GAS	
⊕	TESTHOLE	
•—•	LAMP STANDARD	
○	TREE	

EXISTING	LEGEND - PROFILE	NEW
150 WM	WATERMAIN	150 WM
┼	HYDRANT	┼
X	VALVE	X
300 LDS	LAND DRAINAGE SEWER	300 LDS
250 WWS	WASTE WATER SEWER	250 WWS
—— · ——	PAVEMENT CROWN	—— · ——
—◇—	N/W PROPERTY LINE	
—○—	S/E PROPERTY LINE	
—□—	N/W GUTTER	
— — ○ — —	S/E GUTTER	

LOCATION APPROVED
UNDERGROUND STRUCTURES

SUPR. U/G STRUCTURES COMMITTEE ________ DATE ________

NOTE:
LOCATION OF UNDERGROUND STRUCTURES AS SHOWN ARE BASED ON THE BEST INFORMATION AVAILABLE. BUT NO GUARANTEE IS GIVEN THAT ALL EXISTING UTILITIES ARE SHOWN OR THAT THE GIVEN LOCATIONS ARE EXACT. CONFIRMATION OF EXISTENCE AND EXACT LOCATION OF ALL SERVICES MUST BE OBTAINED FROM THE INDIVIDUAL UTILITIES BEFORE PROCEEDING WITH CONSTRUCTION.

NO.	REVISIONS	YY/MM/DD	BY
2	ISSUED FOR RECORD	19/11/28	KLC
1	REVISED GAS MAIN ALIGNMENT	18/08/02	KLC
0	ISSUED FOR CONSTRUCTION	18/06/01	KLC

AECOM

DESIGNED BY	NJK	CHECKED BY	MGM
DRAWN BY	KLC	APPROVED BY	
HOR. SCALE	AS NOTED	RELEASED FOR CONSTRUCTION	
VERT. SCALE	N/A		
DATE		DATE	

PROFESSIONAL'S SEAL
A.D. BRAUN
Member 24453

CONSULTANT DRAWING NO.
60549028-SHT-30-0000-C-1002

Winnipeg

THE CITY OF WINNIPEG
WATER AND WASTE DEPARTMENT

PROVISION OF PIPELINE ACCESS MODIFICATIONS, CLEANING AND SUPPORT SERVICES FOR RIVER CROSSING INSPECTIONS - PHASE TWO

SITE 2

CHARLESWOOD - ASSINIBOIA
FEEDER MAIN

SHEET 3 OF 7

CITY DRAWING NUMBER
D-15160

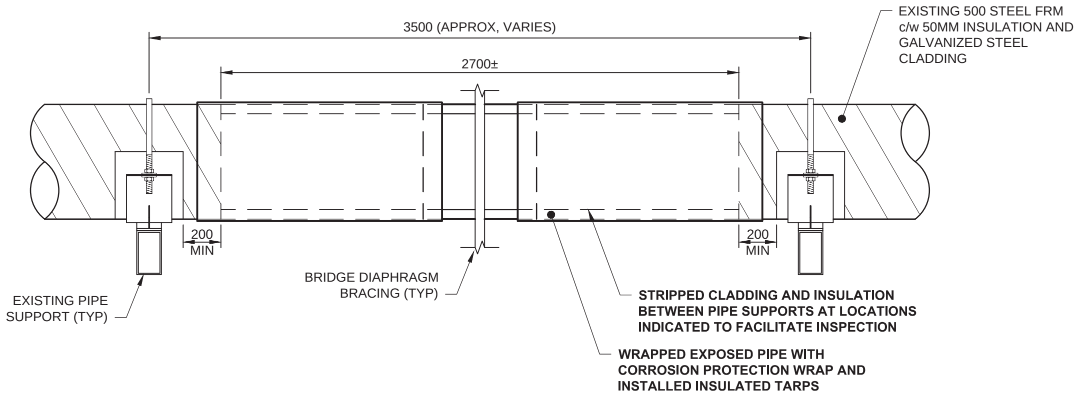
REV 2

AECOM REVIEW DRFT CHK

D SIZE 22" x 34" (559mm x 864mm)

PLOT: 11/27/2019 4:42:30 PM

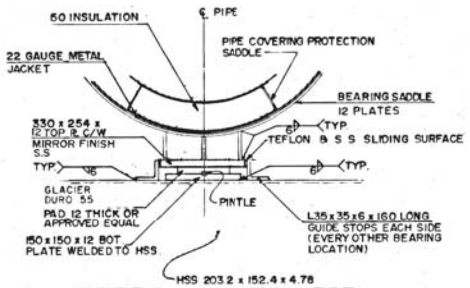
AECOM FILE NAME: 60549028-SHT-30-0000-C-1003.dwg Saved By: cooperkl



- TYPICAL CLADDING REMOVAL DETAIL

-

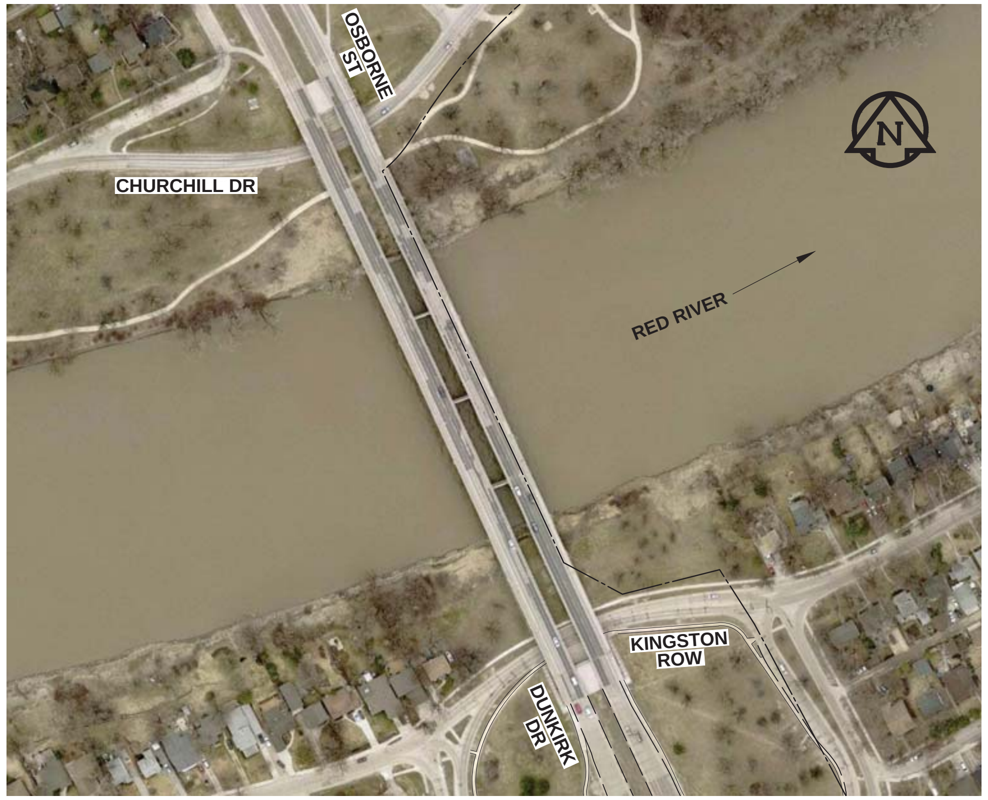
Scale 1:20



- EXISTING PIPE X-SECTION

-

Scale N.T.S.



SITE PLAN

Scale 1:1750

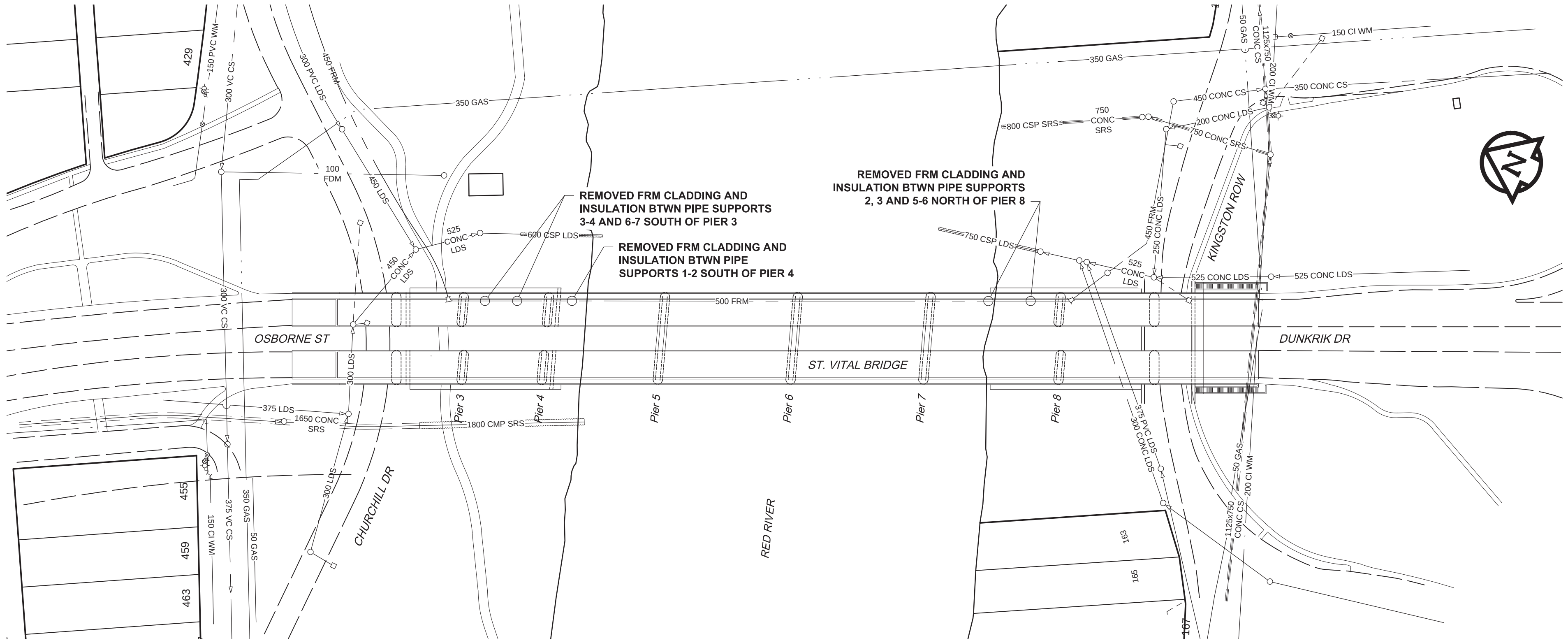
0 43.75 87.50 m

MATERIAL LIST

ITEM	MANUFACTURER	MATERIAL SPECIFICATION	SPECIAL FEATURES
CORROSION PROTECTION WRAP - WRAPID COAT PE	CAN/USA-CPS	AWWA C217	

CONTRACTOR: J-CON CIVIL LTD

CONSTRUCTION PERIOD: SEPT. 14, 2018 to NOV. 8, 2019



PLAN

Scale 1:1000

0 25 50 m

KEY PLAN

ENGINEERS GEOSCIENTISTS MANITOBA

Certificate of Authorization

AECOM Canada Ltd.

No. 4671 Date:____________

CONSTRUCTION COMPLETION DATE:
NOVEMBER 8, 2019

RECORD DRAWING

BID OPPORTUNITY NO. 492-2018

METRIC

WHOLE NUMBERS INDICATE MILLIMETRES
DECIMALIZED NUMBERS INDICATE METRES

EXISTING	LEGEND - PLAN	NEW	EXISTING	LEGEND - PLAN	NEW	EXISTING	LEGEND - PROFILE	NEW
150 WM	WATERMAIN	150 WM	♂	CURB STOP	♂	150 WM	WATERMAIN	150 WM
⌖	HYDRANT	⌖	⊳	REDUCER	⊳	†	HYDRANT	†
⊗	VALVE	●	✕	COUPLING	✕	X	VALVE	X
300 LDS	LAND DRAINAGE SEWER	300 LDS	⊃	ANODE	⊃	300 LDS	LAND DRAINAGE SEWER	300 LDS
250 WWS	WASTE WATER SEWER	250 WWS	—	HYDRO		250 WWS	WASTE WATER SEWER	250 WWS
○	MANHOLE	●	—	MTS		—	PAVEMENT CROWN	—
□	CATCH BASIN	■	—	GAS		◇	N/W PROPERTY LINE	
△	CURB INLET	▲	⊕	TESTHOLE		⬡	S/E PROPERTY LINE	
⊏ ⊐	CULVERT	⊏ ⊐	•—•	LAMP STANDARD		□	N/W GUTTER	
—⊐	PIPE ABANDONMENTS	—⊐	◌	TREE		⊖	S/E GUTTER	
⌖	SURVEY BAR							

AECOM REVIEW DRFT CHK

LOCATION APPROVED
UNDERGROUND STRUCTURES

SUPR. U/G STRUCTURES COMMITTEE DATE

NOTE:
LOCATION OF UNDERGROUND STRUCTURES AS SHOWN ARE BASED ON THE BEST INFORMATION AVAILABLE. BUT NO GUARANTEE IS GIVEN THAT ALL EXISTING UTILITIES ARE SHOWN OR THAT THE GIVEN LOCATIONS ARE EXACT. CONFIRMATION OF EXISTENCE AND EXACT LOCATION OF ALL SERVICES MUST BE OBTAINED FROM THE INDIVIDUAL UTILITIES BEFORE PROCEEDING WITH CONSTRUCTION.

NO.	REVISIONS	YY/MM/DD	BY
1	ISSUED FOR RECORD	19/11/28	KLC
0	ISSUED FOR CONSTRUCTION	18/06/01	KLC

AECOM

DESIGNED BY	NJK	CHECKED BY	
DRAWN BY	KLC	APPROVED BY	
HOR. SCALE	AS NOTED	RELEASED FOR CONSTRUCTION	
VERT. SCALE	N/A		
DATE		DATE	

PROFESSIONAL'S SEAL

A. D. BRAUN Member 24453

CONSULTANT DRAWING NO.
60549028-SHT-30-0000-C-1003

Winnipeg

THE CITY OF WINNIPEG
WATER AND WASTE DEPARTMENT

PROVISION OF PIPELINE ACCESS MODIFICATIONS, CLEANING AND SUPPORT SERVICES FOR RIVER CROSSING INSPECTIONS - PHASE TWO

SITE 3

ST. VITAL BRIDGE
FORCE MAIN

SHEET 4 OF 7

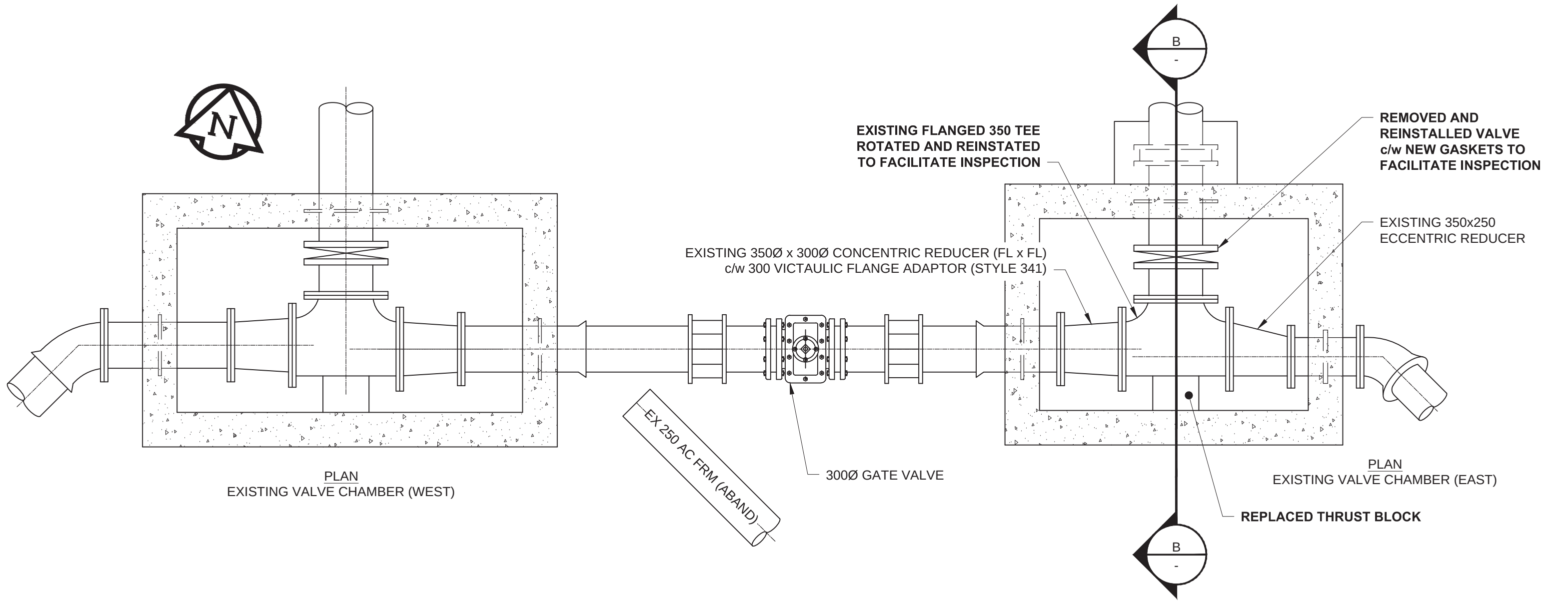
CITY DRAWING NUMBER 11820

REV 1

D SIZE 22" x 34" (559mm x 864mm)

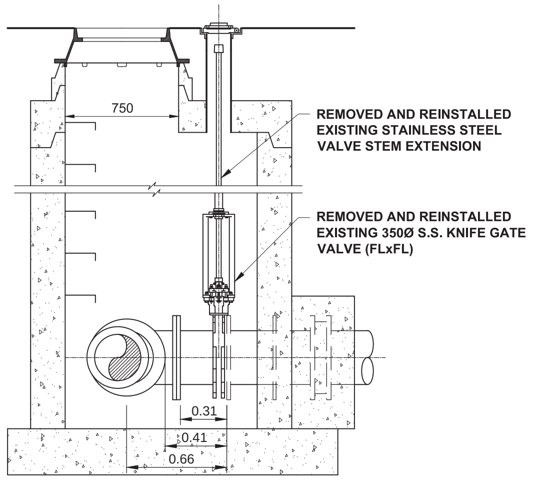
PLOT: 11/27/2019 4:51:37 PM

AECOM FILE NAME: 60549028-SHT-30-0000-C-1004.dwg Saved By: cooperkl



CHAMBER MODIFICATIONS

Scale 1:25



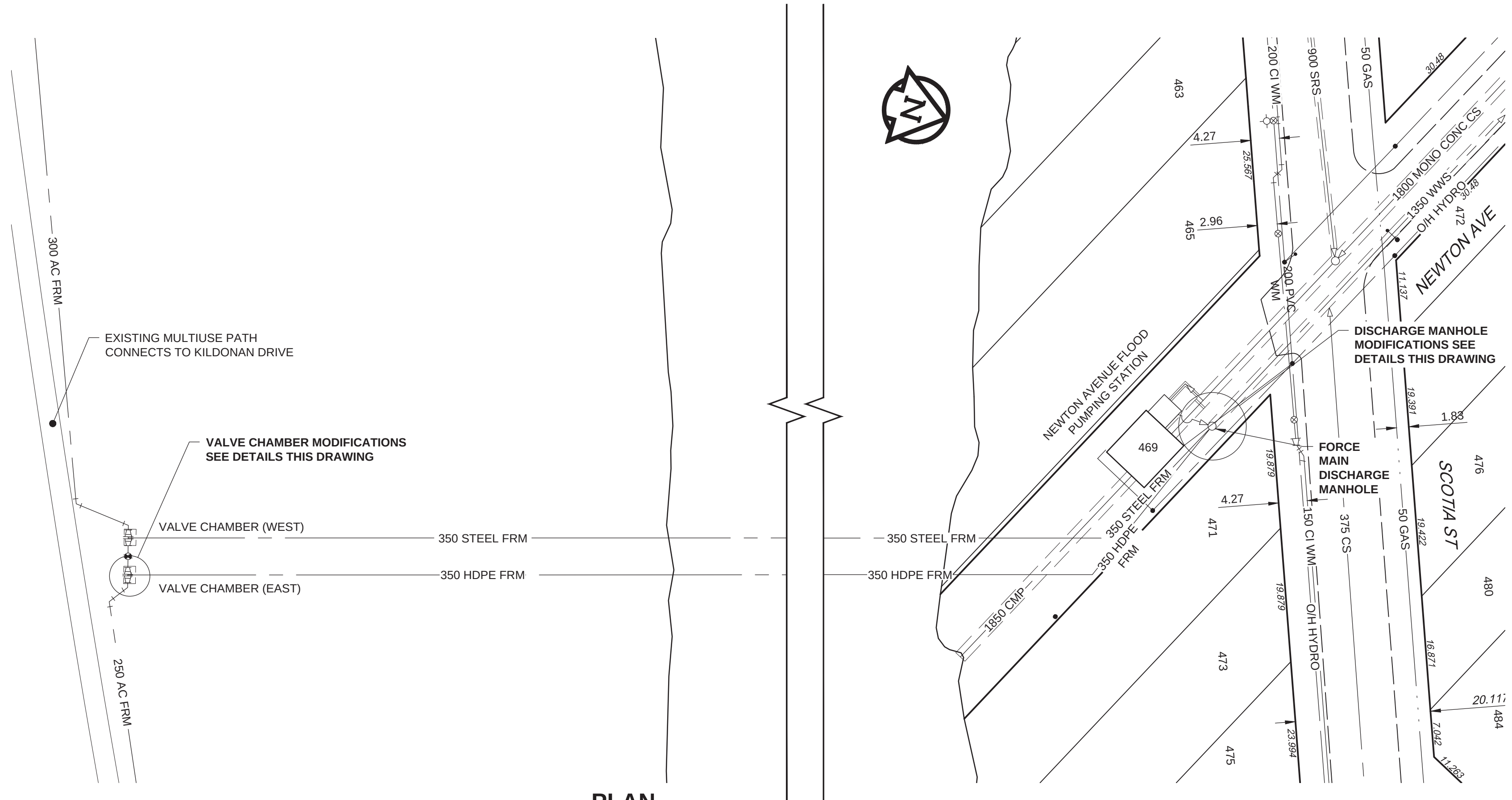
SECTION B-B

Scale 1:25

SITE PLAN

Scale 1:2000

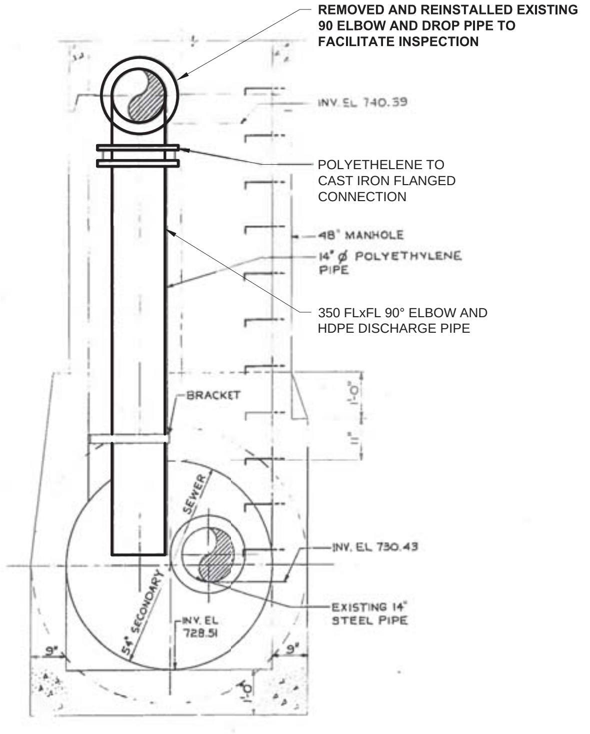
0 50 100 m



PLAN

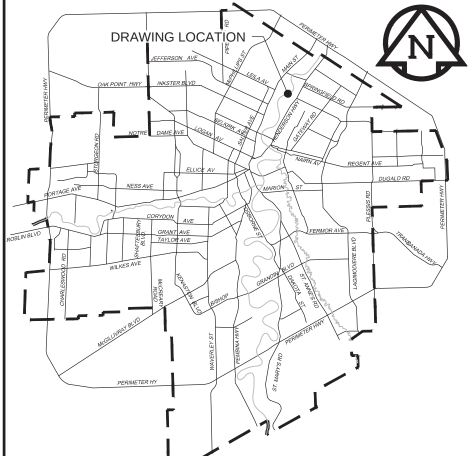
Scale 1:500

0 12.5 25 m



DISCHARGE MANHOLE MODIFICATIONS

Scale 1:25



KEY PLAN

ENGINEERS GEOSCIENTISTS MANITOBA
Certificate of Authorization
AECOM Canada Ltd.
No. 4671 Date:____________

CONSTRUCTION COMPLETION DATE:
NOVEMBER 8, 2019

RECORD DRAWING

BID OPPORTUNITY NO. 492-2018

METRIC

WHOLE NUMBERS INDICATE MILLIMETRES
DECIMALIZED NUMBERS INDICATE METRES

EXISTING	LEGEND - PLAN	NEW	EXISTING	LEGEND - PLAN	NEW	EXISTING	LEGEND - PROFILE	NEW
150 WM	WATERMAIN	150 WM	♂	CURB STOP	●	150 WM	WATERMAIN	150 WM
-○-	HYDRANT	-●-	-▷-	REDUCER	-▶-	†	HYDRANT	+
⊗	VALVE	●	-✕-	COUPLING	-✕-	X	VALVE	X
300 LDS	LAND DRAINAGE SEWER	300 LDS	⊋	ANODE	⊋	300 LDS	LAND DRAINAGE SEWER	300 LDS
250 WWS	WASTE WATER SEWER	250 WWS	—	HYDRO		250 WWS	WASTE WATER SEWER	250 WWS
○	MANHOLE	●	—	MTS		—	PAVEMENT CROWN	—
□	CATCH BASIN	■	—	GAS		-◇-	N/W PROPERTY LINE	
△	CURB INLET	▲	⊕	TESTHOLE		-⬡-	S/E PROPERTY LINE	
⊏ ═ ═ ⊐	CULVERT	⊏ ═ ═ ⊐	•—•	LAMP STANDARD		-□-	N/W GUTTER	
—∃	PIPE ABANDONMENTS	—∃	⊙	TREE		-⊖-	S/E GUTTER	
⌖	SURVEY BAR							

AECOM REVIEW
DRFT CHK

LOCATION APPROVED
UNDERGROUND STRUCTURES

SUPR. U/G STRUCTURES COMMITTEE DATE

NOTE:
LOCATION OF UNDERGROUND STRUCTURES AS SHOWN ARE BASED ON THE BEST INFORMATION AVAILABLE. BUT NO GUARANTEE IS GIVEN THAT ALL EXISTING UTILITIES ARE SHOWN OR THAT THE GIVEN LOCATIONS ARE EXACT. CONFIRMATION OF EXISTENCE AND EXACT LOCATION OF ALL SERVICES MUST BE OBTAINED FROM THE INDIVIDUAL UTILITIES BEFORE PROCEEDING WITH CONSTRUCTION.

NO.	REVISIONS	YY/MM/DD	BY
1	ISSUED FOR RECORD	19/11/28	KLC
0	ISSUED FOR CONSTRUCTION	18/06/01	KLC

AECOM

DESIGNED BY	NJK	CHECKED BY	
DRAWN BY	KLC	APPROVED BY	
HOR. SCALE	AS NOTED	RELEASED FOR CONSTRUCTION	
VERT. SCALE	N/A		
DATE		DATE	

PROFESSIONAL'S SEAL
A. D. BRAUN Member 24453

CONSULTANT DRAWING NO.
60549028-SHT-30-0000-C-1004

THE CITY OF WINNIPEG
WATER AND WASTE DEPARTMENT

PROVISION OF PIPELINE ACCESS MODIFICATIONS,
CLEANING AND SUPPORT SERVICES FOR
RIVER CROSSING INSPECTIONS - PHASE TWO
SITE 4
NEWTON AVENUE
FORCE MAIN

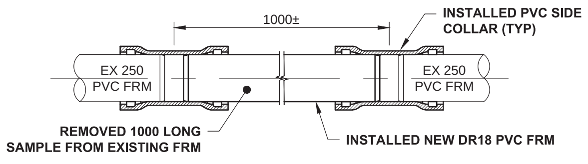
SHEET 5 OF 7

CITY DRAWING NUMBER 11821

REV 1

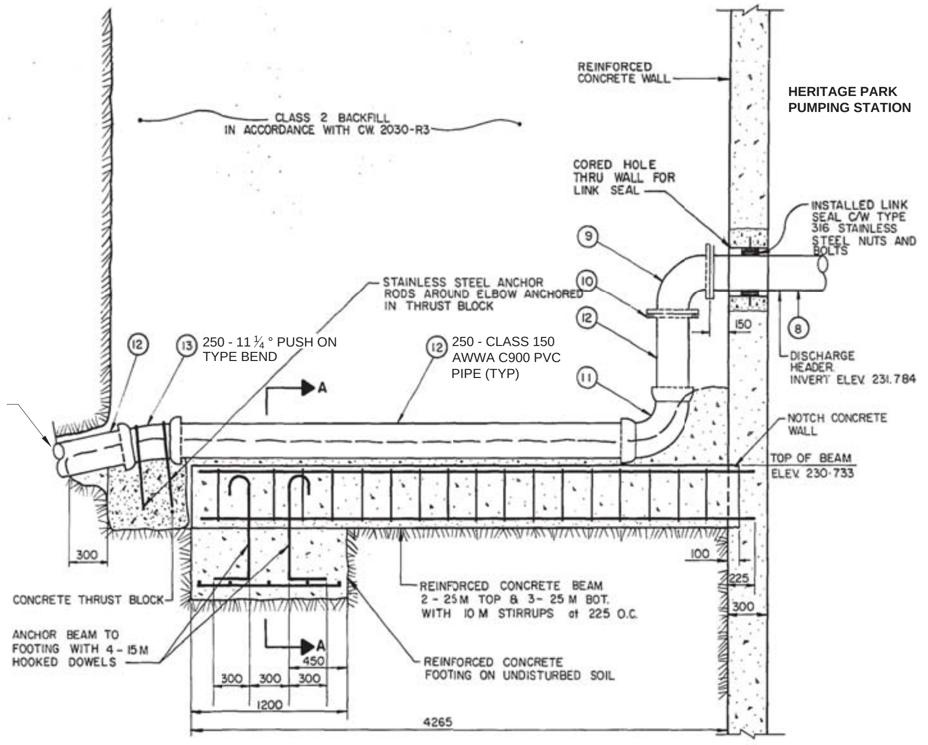
D SIZE 22" x 34" (559mm x 864mm)

PLOT: 11/27/2019 5:00:18 PM



1 PVC SAMPLE REMOVAL

1:20



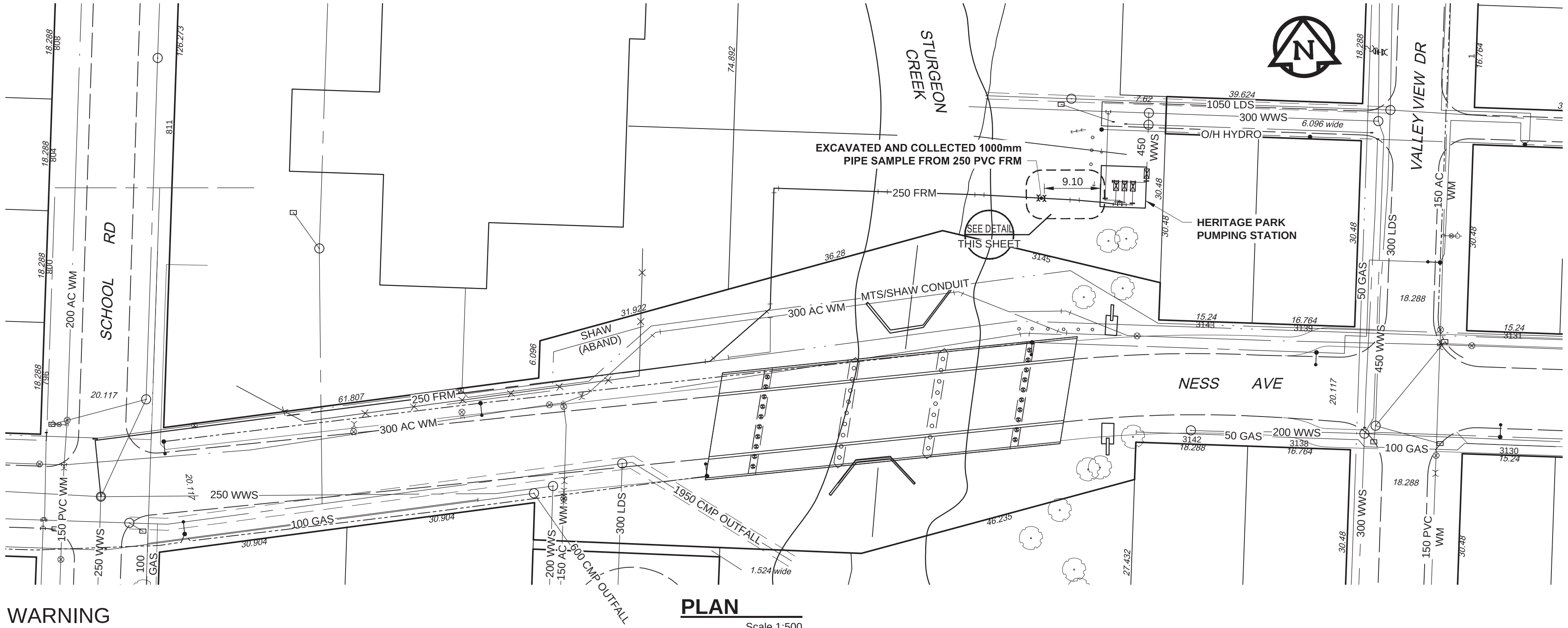
EXISTING SUPPORT BEAM DETAIL

Scale N.T.S.

SITE PLAN

Scale 1:1500

0 37.5 75 m



PLAN

Scale 1:500

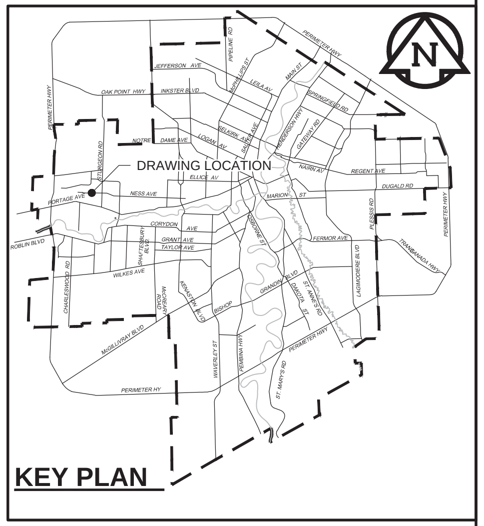
0 12.5 25 m

MATERIAL LIST

ITEM	MANUFACTURER	MATERIAL SPECIFICATION	SPECIAL FEATURES
250 PVC SLIDE COLLAR	IPEX	AWWA C900	
PVC PIPE (150 TO 300)	NORTHERN PIPE PRODUCTS	AWWA C900	DR-18

CONTRACTOR: J-CON CIVIL LTD

CONSTRUCTION PERIOD: SEPT. 14, 2018 to NOV. 8, 2019



KEY PLAN

WARNING

IF POWER EQUIPMENT OR EXPLOSIVES ARE TO BE USED FOR EXCAVATION ON THIS PROJECT, THE CONTRACTOR MUST:

1) NOTIFY THE GAS COMPANY OF THE PROPOSED LOCATION OF EXCAVATION.

2) TAKE PRECAUTION TO AVOID DAMAGE TO GAS COMPANY INSTALLATIONS.

SEE PROVINCIAL REGULATION 140/92 FOR DETAILS

METRIC

WHOLE NUMBERS INDICATE MILLIMETRES
DECIMALIZED NUMBERS INDICATE METRES

ENGINEERS GEOSCIENTISTS MANITOBA
Certificate of Authorization
AECOM Canada Ltd.
No. 4671 Date:____________

CONSTRUCTION COMPLETION DATE:
NOVEMBER 8, 2019

RECORD DRAWING

BID OPPORTUNITY NO. 492-2018

EXISTING	LEGEND - PLAN	NEW	EXISTING	LEGEND - PLAN	NEW	EXISTING	LEGEND - PROFILE	NEW
150 WM	WATERMAIN	150 WM	♂	CURB STOP	●	150 WM	WATERMAIN	150 WM
-○-	HYDRANT	-●-	⊳	REDUCER	▶	†	HYDRANT	✝
⊗	VALVE	●	✕	COUPLING	—	X	VALVE	X
300 LDS	LAND DRAINAGE SEWER	300 LDS	⊋	ANODE	⊋	300 LDS	LAND DRAINAGE SEWER	300 LDS
250 WWS	WASTE WATER SEWER	250 WWS	—	HYDRO		250 WWS	WASTE WATER SEWER	250 WWS
○	MANHOLE	●	—	MTS		—	PAVEMENT CROWN	—
□	CATCH BASIN	■	—	GAS		◇	N/W PROPERTY LINE	
△	CURB INLET	▲	⊕	TESTHOLE		○	S/E PROPERTY LINE	
⊏ ═ ═ ⊐	CULVERT	⊏ ═ ═ ⊐	•—•	LAMP STANDARD		□	N/W GUTTER	
—∃	PIPE ABANDONMENTS	—∃	○	TREE		—○—	S/E GUTTER	
⊕	SURVEY BAR							

LOCATION APPROVED
UNDERGROUND STRUCTURES

SUPR. U/G STRUCTURES COMMITTEE ____ DATE ____

NOTE:
LOCATION OF UNDERGROUND STRUCTURES AS SHOWN ARE BASED ON THE BEST INFORMATION AVAILABLE. BUT NO GUARANTEE IS GIVEN THAT ALL EXISTING UTILITIES ARE SHOWN OR THAT THE GIVEN LOCATIONS ARE EXACT. CONFIRMATION OF EXISTENCE AND EXACT LOCATION OF ALL SERVICES MUST BE OBTAINED FROM THE INDIVIDUAL UTILITIES BEFORE PROCEEDING WITH CONSTRUCTION.

NO.	REVISIONS	YY/MM/DD	BY
1	ISSUED FOR RECORD	19/11/28	KLC
0	ISSUED FOR CONSTRUCTION	18/06/01	KLC

AECOM

DESIGNED BY	NJK	CHECKED BY	
DRAWN BY	KLC	APPROVED BY	
HOR. SCALE	AS NOTED	RELEASED FOR CONSTRUCTION	
VERT. SCALE	N/A		
DATE		DATE	

PROFESSIONAL'S SEAL
A. D. BRAUN
Member
24453

CONSULTANT DRAWING NO.
60549028-SHT-30-0000-C-1005

THE CITY OF WINNIPEG
WATER AND WASTE DEPARTMENT

PROVISION OF PIPELINE ACCESS MODIFICATIONS, CLEANING AND SUPPORT SERVICES FOR RIVER CROSSING INSPECTIONS - PHASE TWO

SITE 5

HERITAGE PARK
FORCE MAIN

SHEET 6 OF 7

CITY DRAWING NUMBER 11822 REV 1

AECOM FILE NAME: 60549028-SHT-30-0000-C-1005.dwg Saved By: cooperkl

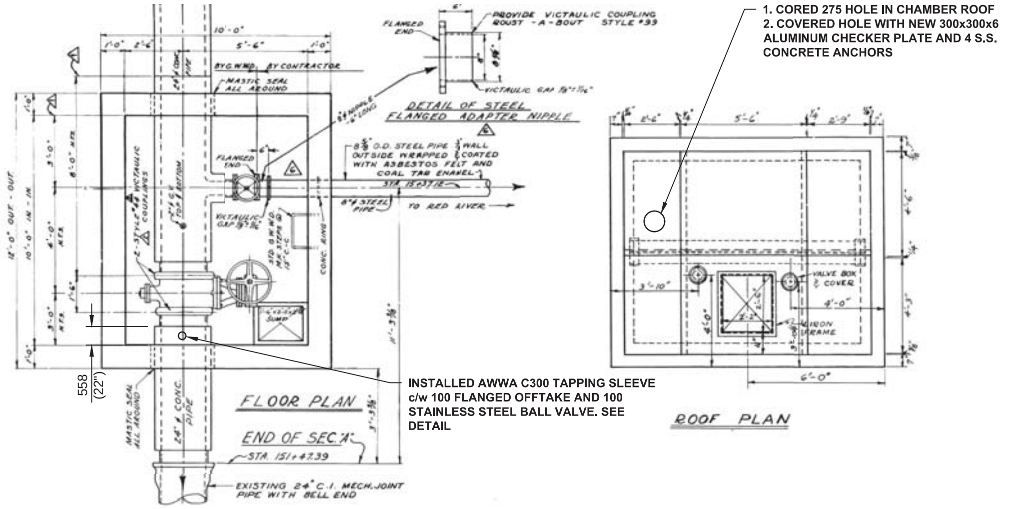
AECOM REVIEW DRFT CHK

D SIZE 22" x 34" (559mm x 864mm)

PLOT: 11/27/2019 5:07:58 PM

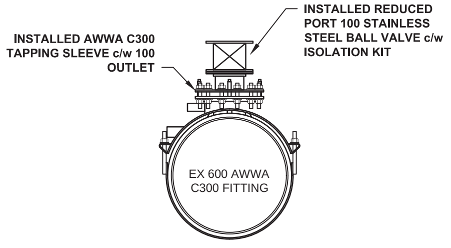
AECOM FILE NAME: 60549028-SHT-30-0000-C-1006.dwg Saved By: cooperkl

AECOM REVIEW DRFT CHK



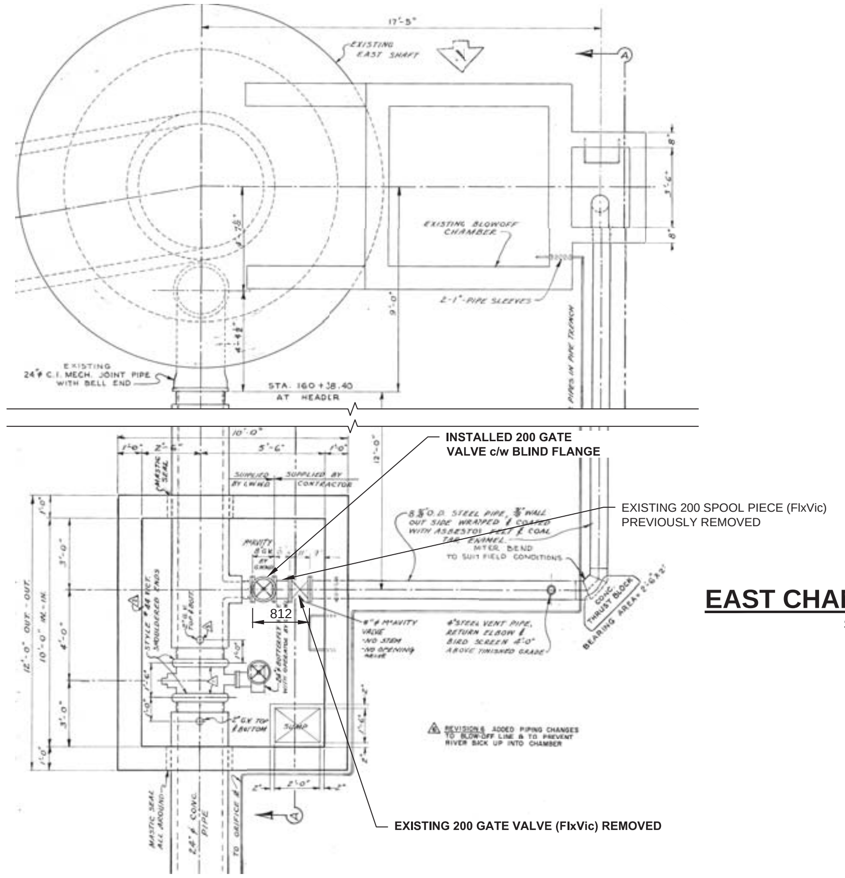
WEST CHAMBER

Scale N.T.S.



1 NEW 100Ø BLOW OFF VALVE DETAIL

Scale: N.T.S.



EAST CHAMBER

Scale N.T.S.

SITE PLAN

Scale 1:1750

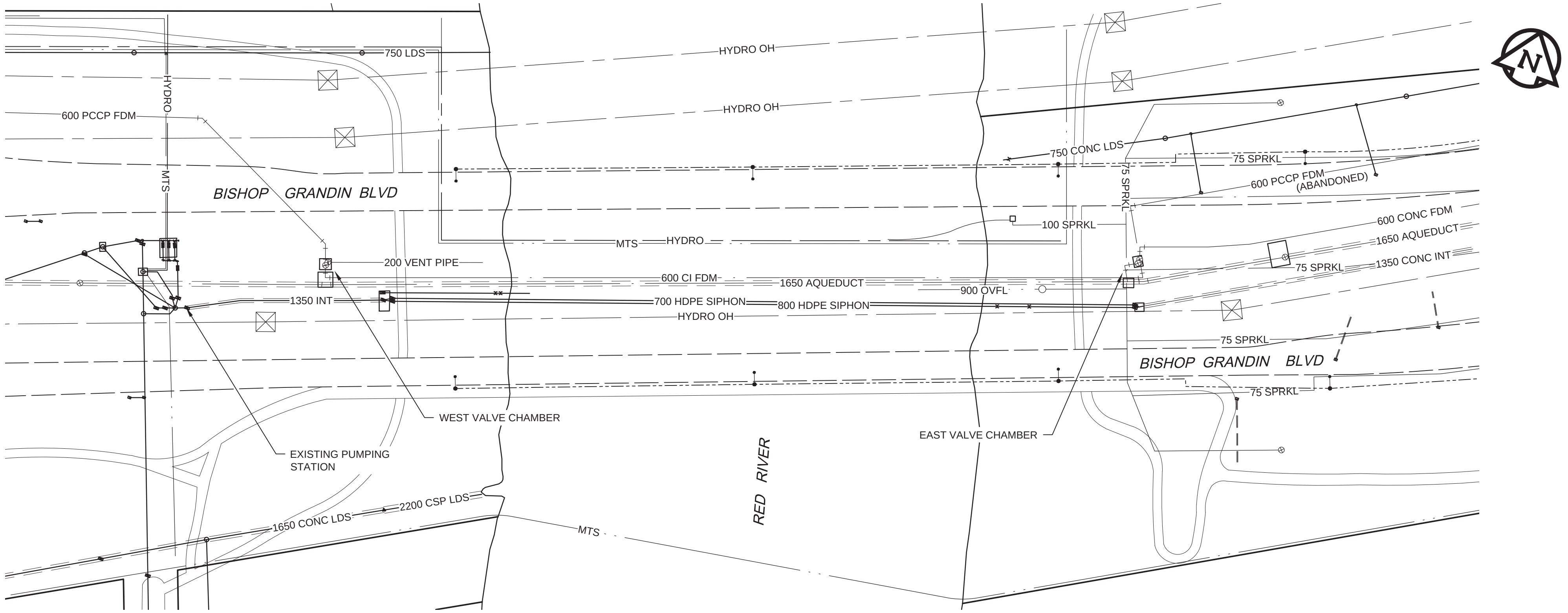
0 43.75 87.50 m

MATERIAL LIST

ITEM	MANUFACTURER	MATERIAL SPECIFICATION	SPECIAL FEATURES
200 BLIND FLANGE	U.S. PIPE FABRICATION	ASME/ANSI B16.5 12516	
200 DRAIN VALVE	MUELLER	ANSI/AWWA C509	
600 TAPPING SLEEVE	ROBAR INDUSTRIES LTD	Flange: AWWA C207 CLASS D Steel: ASTM C36 Straps & Fasteners: SS 316 Gasket: NBR ASTM D2000 Coatings: AWWA C213	100mm OUTLET
100 REDUCED PORT STAINLESS STEEL BALL VALVE	M.A. STEWART & SONS LTD	ANSI B16.34 CLASS 150	75mm OPENING
400 MM X 350 MM X 12.5 MM CHECKERPLATE ALUMINUM	J-CON		

CONTRACTOR: J-CON CIVIL LTD

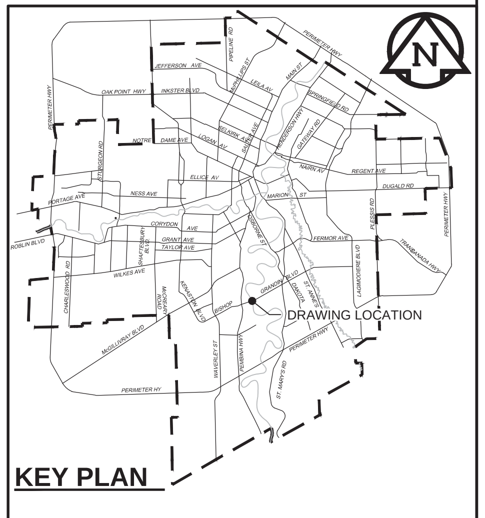
CONSTRUCTION PERIOD: SEPT. 14, 2018 to NOV. 8, 2019



PLAN

Scale 1:1000

0 25 50 m



KEY PLAN

METRIC

WHOLE NUMBERS INDICATE MILLIMETRES
DECIMALIZED NUMBERS INDICATE METRES

ENGINEERS GEOSCIENTISTS MANITOBA
Certificate of Authorization
AECOM Canada Ltd.
No. 4671 Date:____________

CONSTRUCTION COMPLETION DATE:
NOVEMBER 8, 2019

RECORD DRAWING

BID OPPORTUNITY NO. 492-2018

EXISTING	LEGEND - PLAN	NEW	EXISTING	LEGEND - PLAN	NEW	EXISTING	LEGEND - PROFILE	NEW
150 WM	WATERMAIN	150 WM	♂	CURB STOP	●	150 WM	WATERMAIN	150 WM
⌖	HYDRANT	●	▷	REDUCER	▶	†	HYDRANT	✝
⊗	VALVE	●	✕	COUPLING	—	X	VALVE	X
300 LDS	LAND DRAINAGE SEWER	300 LDS	⊃	ANODE	⊃	300 LDS	LAND DRAINAGE SEWER	300 LDS
250 WWS	WASTE WATER SEWER	250 WWS	—	HYDRO		250 WWS	WASTE WATER SEWER	250 WWS
○	MANHOLE	●	—	MTS		—	PAVEMENT CROWN	—
□	CATCH BASIN	■	—	GAS		◇	N/W PROPERTY LINE	
△	CURB INLET	▲	⊕	TESTHOLE		○	S/E PROPERTY LINE	
⊏ — — ⊐	CULVERT	⊏ — — ⊐	●—●	LAMP STANDARD		□	N/W GUTTER	
—∃	PIPE ABANDONMENTS	—∃	○	TREE		⊖	S/E GUTTER	
⊕	SURVEY BAR							

LOCATION APPROVED
UNDERGROUND STRUCTURES

SUPR. U/G STRUCTURES COMMITTEE ____ DATE ____

NOTE:
LOCATION OF UNDERGROUND STRUCTURES AS SHOWN ARE BASED ON THE BEST INFORMATION AVAILABLE. BUT NO GUARANTEE IS GIVEN THAT ALL EXISTING UTILITIES ARE SHOWN OR THAT THE GIVEN LOCATIONS ARE EXACT. CONFIRMATION OF EXISTENCE AND EXACT LOCATION OF ALL SERVICES MUST BE OBTAINED FROM THE INDIVIDUAL UTILITIES BEFORE PROCEEDING WITH CONSTRUCTION.

NO.	REVISIONS	YY/MM/DD	BY
1	ISSUED FOR RECORD	19/11/28	KLC
0	ISSUED FOR CONSTRUCTION	18/06/01	KLC

AECOM

DESIGNED BY	ADB	CHECKED BY	
DRAWN BY	KLC	APPROVED BY	
HOR. SCALE	AS NOTED	RELEASED FOR CONSTRUCTION	
VERT. SCALE	N/A		
DATE		DATE	

PROFESSIONAL'S SEAL
A. D. BRAUN Member 24453

CONSULTANT DRAWING NO.
60549028-SHT-30-0000-C-1006

THE CITY OF WINNIPEG
WATER AND WASTE DEPARTMENT

PROVISION OF PIPELINE ACCESS MODIFICATIONS, CLEANING AND SUPPORT SERVICES FOR RIVER CROSSING INSPECTIONS - PHASE TWO

SITE 6

FORT GARRY - ST. VITAL
FEEDER MAIN

SHEET 7 OF 7

CITY DRAWING NUMBER
D-15161 REV 1

Appendix G

Site 1 - Tunnel Inspection Report

AECOM Imagine it. Delivered.

AECOM
99 Commerce Drive 204 477 5381 tel
Winnipeg, MB, Canada R3P 0Y7 204 284 2040 fax
www.aecom.com

Project:	**Kildonan-Redwood Feeder Main Pipe Inspection**	Date:	March 29, 2019
Contractor:	J-Con Civil Ltd.	Project #:	60549028
Owner:	City of Winnipeg	Weather:	Snow, Blustery, -5° C

Inspection Report

On March 29, 2019, Marshall Gibbons of AECOM performed a visual inspection of the 600mm-diameter Kildonan-Redwood Feeder Main within the tunnel on the west side of the Red River crossing (Figure A 1 and Figure A 2). The purpose of AECOM's inspection was to:

- Evaluate the exterior condition of the feeder main and its joints within the tunnel;
- Detect and measure sites of corrosion on the exterior of the pipe and measure remaining pipe wall thickness at said locations using an ultrasonic thickness gauge. The data collected would be compared with the results of the inline electromagnetic (EM) survey of the feeder main completed by Pipeline Inspection and Condition Analysis Corporation (PICA);
- Evaluate the general condition of the tunnel structure based on cursory visual inspection; and
- Evaluate the accuracy of information depicted on historic WWD drawings (WA-12530 and WA-12531).

Concurrently, two workers from PICA examined the exterior condition of the feeder main, with attention being given to locations where electromagnetic anomalies were detected by the inline EM survey. Selected photos from the inspection are attached as Appendix A.

Background

Notes on historic drawing WA-12530 and details included in a water industry publication [1] indicate the river crossing pipe was constructed primarily of 610 mm (24-inch) diameter spiral-welded steel with a wall thickness of 7.94 mm (0.3125 inch) and conforming to American Water Works Association (AWWA) standard specification 7A.4-1949 [2]. The steel riser pipe in the shaft on the west side of the river crossing was to have a wall thickness no less than 12.7 mm (0.5 inch). For protection against corroding, the interiors and exteriors of the pipes were coated with coal tar enamel (CTE), with an added protective felt wrap installed on the exterior of the pipe, in conformance with AWWA standard specification 7A.6 [3]. Flanged pipe joints with stainless steel fasteners were used in preference to welded pipe joints, and a few Victaulic-coupled joints were used within the tunnel.

Inspection

At the start of the inspection, AECOM attached a measuring tape to the flange of the 90-degree elbow at the base of the tunnel shaft, and PICA layed-out the tape along the top of the feeder main to the east end of the tunnel. As this was being done, AECOM measured the locations of pipe features along the tunnel. PICA then

---

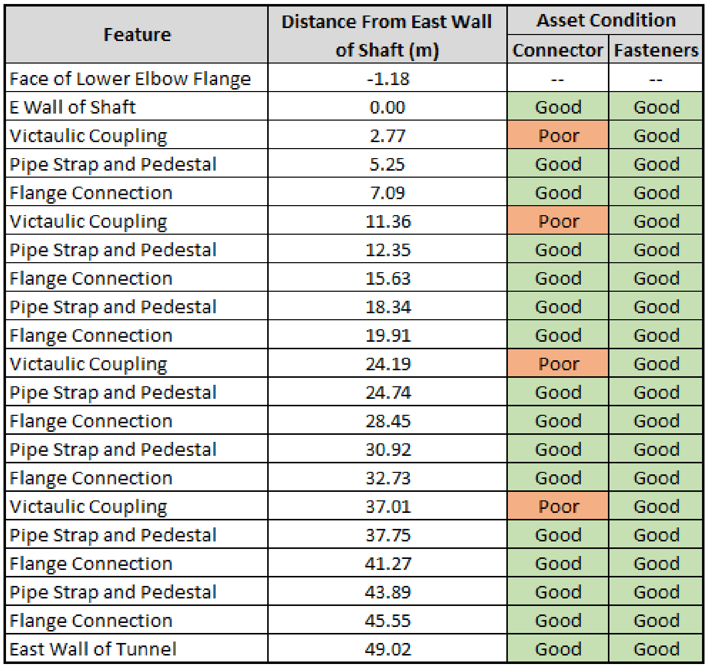
[1] *W.L. Wardrop, A New 24" Watermain Crossing Under The Red River At Redwood Avenue, Western Canada Water and Sewage Conference, Sep. 20, 1955, 13 pp.*

[2] *AWWA Standard Specification 7A.4 – 1949, Steel Water Pipe Of Sizes Of 4 Inches Up To But Not Including 30 Inches.*

[3] *AWWA Standard Specification 7A.6, Coal Tar Enamel Protective Coatings For Steel Water Pipe, Sizes Up To 30 Inches.*

returned to the tunnel shaft and commenced their inspection of the feeder main in an eastward direction. After reaching the east end of the tunnel, AECOM inspected the feeder main and tunnel in a westward direction, with the locations of features and defects being recorded based on distance from the base elbow and clock position around the pipe circumference (clock reference facing westward). A summary of feeder main features and their condition is provided in Table 1.

Table 1 - Summary of Feeder Main Features and Their Condition.



Feature	Distance From East Wall of Shaft (m)	Asset Condition	
		Connector	Fasteners
Face of Lower Elbow Flange	-1.18	--	--
E Wall of Shaft	0.00	Good	Good
Victaulic Coupling	2.77	Poor	Good
Pipe Strap and Pedestal	5.25	Good	Good
Flange Connection	7.09	Good	Good
Victaulic Coupling	11.36	Poor	Good
Pipe Strap and Pedestal	12.35	Good	Good
Flange Connection	15.63	Good	Good
Pipe Strap and Pedestal	18.34	Good	Good
Flange Connection	19.91	Good	Good
Victaulic Coupling	24.19	Poor	Good
Pipe Strap and Pedestal	24.74	Good	Good
Flange Connection	28.45	Good	Good
Pipe Strap and Pedestal	30.92	Good	Good
Flange Connection	32.73	Good	Good
Victaulic Coupling	37.01	Poor	Good
Pipe Strap and Pedestal	37.75	Good	Good
Flange Connection	41.27	Good	Good
Pipe Strap and Pedestal	43.89	Good	Good
Flange Connection	45.55	Good	Good
East Wall of Tunnel	49.02	Good	Good

Key observations from the inspection are as follows:

- The exterior of the feeder main pipe appears in very good condition; no corrosion of the pipe was observed.
- Locations of pipe features are accurately depicted on historic drawing 12531.
- Defects/conditions observed on the pipe segments include:
    - Only one break in the pipe coating was observed, 50 mm from east end of tunnel at 2:30 o'clock. The break was 25 mm-diameter and occurred only in the exterior CTE-saturated felt outer wrap. The outer wrap was slightly delaminated from the inner primary coating around the break, and groundwater had seeped into the space between the coating layers.
    - Several blemishes in the coating as may be caused by lifting the pipe with straps or forks while the coating was soft.
    - Dents or gouges in the steel pipe and coating caused by mechanical forces, which required the coatings to be repaired. PICA advised that such damage may be detected by their EM tool because it causes stress-related changes in the steel.
- Flanged connections were in good condition, exhibiting only slight general corrosion. Fasteners, which appeared to be galvanized or stainless steel, also were in good condition.
- Galvanized pipe straps at pedestals were in very good condition.
- Victaulic couplings were severely corroded and should be replaced. The bodies of the couplings were unevenly corroded with the appearance of grey cast iron, a brittle material. The underlying restraining rings

on the feeder main pipes, where visible, appeared in good condition and were smooth-surfaced with no pitting observed. The fasteners on the couplings also appeared in good condition.

- The finished tunnel length is 49.02 m (160.83 feet), not 57.3 m (188 feet) as shown on historic drawing WA-12531. The water industry publication discussed numerous difficulties that were encountered while constructing the east end of the tunnel.
- Tunnel was in good condition, but water was observed seeping (dripping to slow running) from some of the concrete cold-pour joints that were spaced at 4 or 6 feet along the tunnel.
- The original Universal cast iron air piping had been replaced with aluminum pipe, which has suffered galvanic corrosion due to direct contact with the steel or iron pipe hanger assemblies. Condition of the aluminum piping ranged from being relatively uncorroded to being completely perforated.
- The built-in air piping system was effective in ventilating the tunnel.

Since the length of the tunnel was shorter than depicted on drawing 12531, AECOM requested that PICA review the EM data to determine if any of the features outside the tunnel (bends, connections and pedestals) also had been relocated. PICA advised the following (see Figure 1):

- The data shows what potentially are two welds associated with each of the 30° flanged bends. At each bend there is a single 30° deflection at the weld furthest away from the flange. Drawing 12531 depicts this construction accurately.
- The apparent increase in wall thickness for the pipe at 170 to 174 m (shown with a dashed black line) may be due to rebar within the concrete encasement.
- Regarding the water stop shown within the east tunnel wall on drawing 12531, two wall-gain signals associated with possible water stop locations were identified. However, the signals are quite different from each other so if they are both puddle flanges, the actual flange make-up is different for each.
- In the EM data, PICA can distinguish between flanged and Victaulic connections. The connection just beyond the east tunnel wall appears to be a Victaulic, as is shown on drawing 12531. However, the pipe strap and pedestal shown further east does not appear to be present, though it is possible that the wall gain signal marked as a possible puddle flange is in fact the strap support at a relocated position.

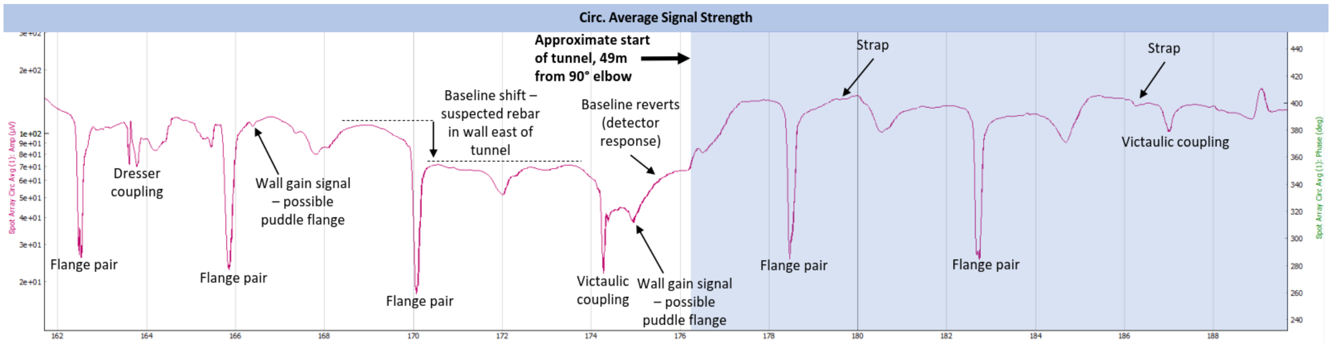


Figure 1 - Electromagnetic Inspection Data for the Feeder Main at the East Tunnel Wall. (East is left; vertical bends are on left side; tunnel area is shown grey)

Recommendations

If the feeder main will remain in service, the following recommendations should be implemented:

- Replace the Victaulic couplings.
- Install a petrolatum tape coating repair (mastic and tape wrap) around the feeder main pipe at the coating break near the east tunnel wall. The repair should extend from the wall to 0.2 m from the wall.
- Install petrolatum tape coating system over all flanged connections. Clean dirt and corrosion from flanges prior to coating and use profiling mastic to fill irregular spaces between flange fasteners, creating a smooth surface for tape wrapping.
- Repair or replace the deteriorated tunnel ventilation pipe. Options include:
  - Replace the existing aluminum piping with plastic piping (PVC or CPVC) that will not undergo galvanic corrosion on contact with the ferrous metal pipe anchors and will not corrode in the humid tunnel environment. Piping could be socketed for joining by solvent welding.
  - Wrap a neoprene sheet gasket around the air pipe at the hanger points to isolate the aluminum from the steel / iron hangers (eliminate galvanic corrosion) and seal the perforations through the pipe. Gaskets may be held in place by plastic zip ties.
  - Repair or replace the corroded aluminum piping at the anchor locations and secure the piping to the hangers with non-metallic anchors (for example, FRP anchor system).
- Monitor the condition of the ladders in the tunnel shaft.
- Seal cracks and / or joints in the tunnel structure from water infiltration by injecting them with hydrophilic polyurethane grout. Similar repairs have been made throughout the Shoal Lake Aqueduct.

Marshall Gibbons, C.E.T.
Senior Municipal Technologist
Water
MAG/pab

Appendix A

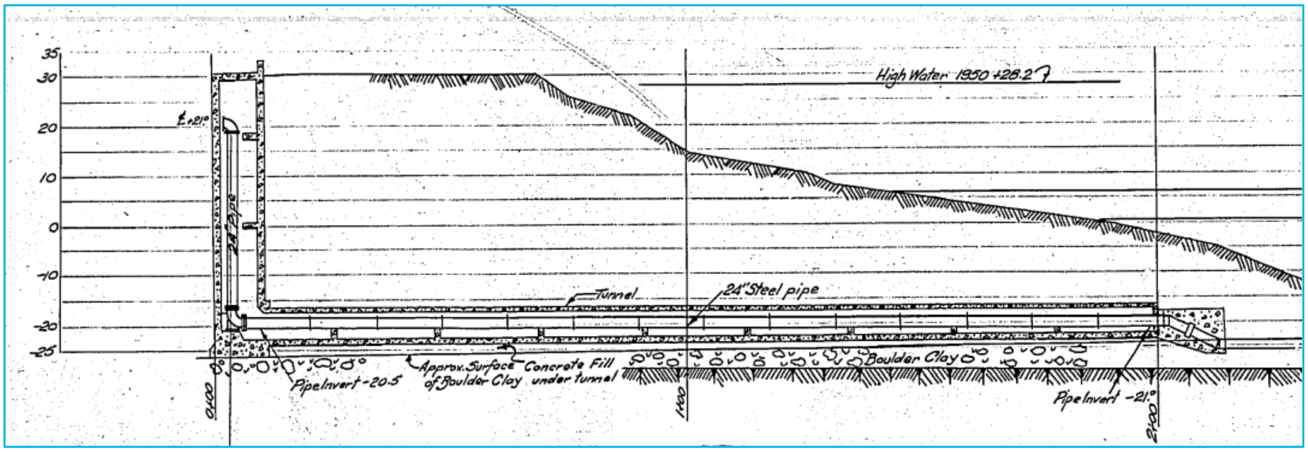


Figure A 1 –Feeder Main and Tunnel on West Side of Red River, View Looking Northward. (From WWD Historic Drawing WA-12535).

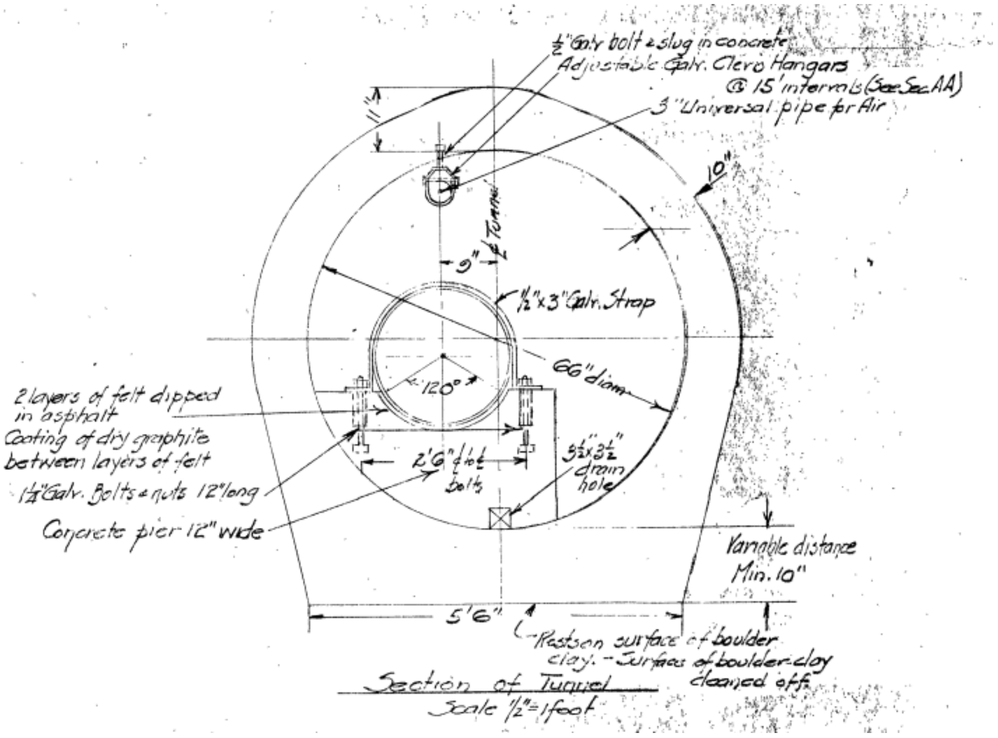


Figure A 2 – Cross-Sectional View of Feeder Main and Tunnel, View Looking Westward. (From WWD Historic Drawing WA-12531).

Figure A 3 – Contractor's Equipment at Top of Shaft on Redwood Avenue.

Figure A 4 – View Into Shaft on Redwood Avenue.

Figure A 5 – View Into Base of Shaft Showing Condition of Galvanized Ladder.

Figure A 6 – View Into 350 mm Water Main Through West Wall of Shaft.

Figure A 7 – East End of Tunnel, 49.0 m From Shaft.

Figure A 8 – End of Aluminum Air Pipe at East End of Tunnel.

Figure A 9 – View of Tunnel, Looking Westward From East End.

Figure A 10 – View of Tunnel, Looking Westward From East End.

Figure A 11 – Water Infiltrating at Concrete Cold Pour Joint 4.5 m from Tunnel End.

Figure A 12 – Water Infiltrating at Concrete Cold Pour Joint 4.5 m from Tunnel End.

Figure A 13 – Flange Connection at 10:00 o'clock, 46.78 m from Shaft. Note Corrosion Tubercle Between Bolt Heads at 9:30 o'clock.

Figure A 14 –Flange Connection at 6:00 o'clock, 46.78 m from Shaft.

Figure A 15 – Underside of Flange Connection at 46.78 m from Shaft.

Figure A 16 – Typical Galvanized Pipe Strap and Pedestal, 45.12 m From Shaft.

Figure A 17 – Typical Galvanized Pipe Strap and Pedestal, 45.12 m From Shaft.

Figure A 18 –Feeder Main on Pedestal at 6:00 o'clock, 45.12 m From Shaft.

Figure A 19 – Flange Connection at 2:00 o'clock, 42.50 m from Shaft.

Figure A 20 –Flange Connection at 6:00 o'clock, 42.50 m from Shaft.

Figure A 21 – Underside of Flange Connection at 42.50 m from Shaft.

Figure A 22 – Typical Corrosion of Aluminum Air Pipe at Anchor Point, ~40.0m From Shaft.

Figure A 23 – Victaulic Connection at 12:00 o'clock, 38.24 m from Shaft.

Figure A 24 – Victaulic Connection at 2:00 o'clock, 38.24 m from Shaft.

Figure A 25 – Underside of Victaulic Connection at 8:00 o'clock, 38.24 m from Shaft.

Figure A 26 – Underside of Victaulic Connection at 4:00 o'clock, 38.24 m from Shaft.

Figure A 27 – Corroded Body of Victaulic Connection at 1:00 o'clock, 38.24 m from Shaft.

Figure A 28 – Corroded Body of Victaulic Connection at 1:00 o'clock, 38.24 m from Shaft.

Figure A 29 – Corroded Body of Victaulic Connection at 1:30 o'clock, 38.24 m from Shaft.

Figure A 30 – 125 mm-square Dimple in Coating at 2:30 o'clock, 37.20 m From Shaft.

Figure A 31 – Flange Connection at 9:00 o’clock, 33.96 m from Shaft.

Figure A 32 – Flange Connection at 12:00 o’clock, 33.96 m from Shaft.

Figure A 33 – Flange Connection at 5:00 o’clock, 33.96 m from Shaft.

Figure A 34 – View of Tunnel, Looking Westward at 33.96 m From Shaft.

Figure A 35 – View of Tunnel, Looking Westward at 33.96 m From Shaft.

Figure A 36 – Flange Connection at 12:00 o'clock, 29.68 m from Shaft.

Figure A 37 – Flange Connection at 9:00 o’clock, 29.68 m from Shaft.

Figure A 38 –Flange Connection at 6:00 o’clock, 29.68 m from Shaft.

Figure A 39 – Coating Dimple at 11:00 o'clock, 26.30 m From Shaft. Possible Dent in Pipe.

Figure A 40 – Victaulic Connection at 12:30 o'clock, 25.42 m from Shaft.

Figure A 41 – Victaulic Connection at 2:00 o'clock, 25.42 m from Shaft.

Figure A 42 –Victaulic Connection at 6:00 o'clock, 25.42 m from Shaft.

Figure A 43 –Victaulic Connection at 10:00 o'clock, 25.42 m from Shaft.

Figure A 44 – Underside of Victaulic Connection at 8:00 o'clock, 25.42 m from Shaft.

Figure A 45 – Corroded Body of Victaulic Connection at 1:00 o'clock, 25.42 m from Shaft.

Figure A 46 – Flange Connection at 2:00 o'clock, 21.14 m from Shaft.

Figure A 47 – Flange Connection at 5:00 o'clock, 21.14 m from Shaft.

Figure A 48 – Flange Connection at 5:00 o'clock, 21.14 m from Shaft.

Figure A 49 – Coating Dimple at 12:30 to 2:30 o'clock, 20.30 m From Shaft. Possible Dent in Pipe.

Figure A 50 – Corrosion / Perforation of Aluminum Air Pipe, 19.50 m From Shaft.

Figure A 51 – Water Infiltrating (Slow Run) at Concrete Pour Joint, 19.10 m From Shaft.

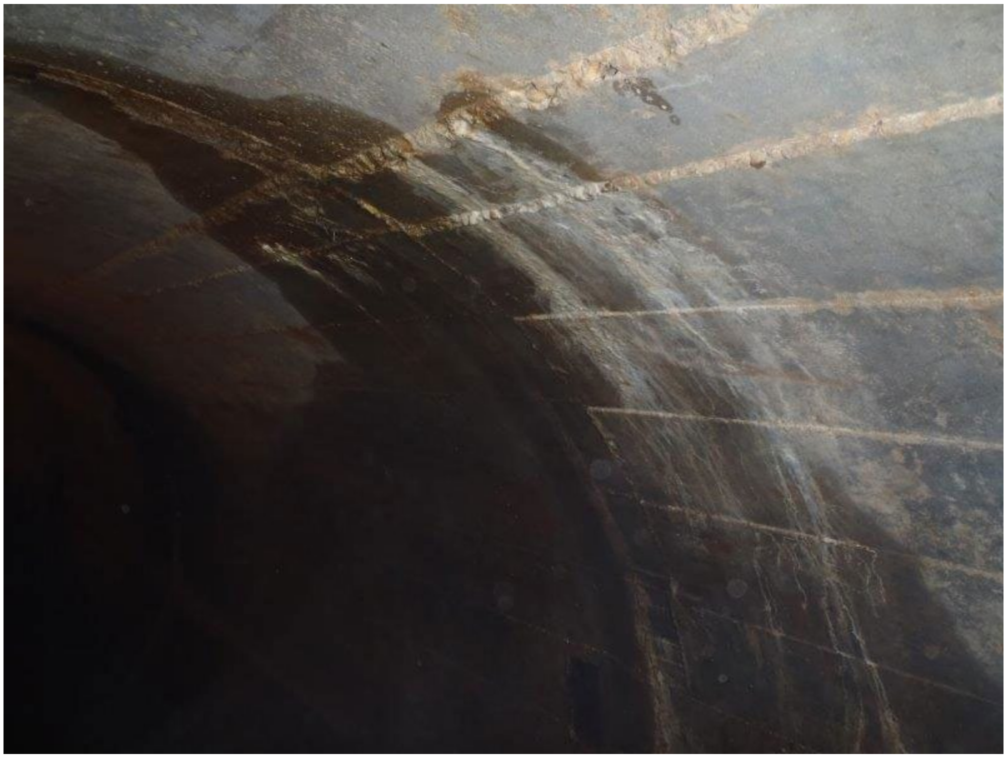


Figure A 52 – Water Infiltrating (Slow Run) at Concrete Pour Joint, and at 100 mm-diameter honeycomb pocket at 1:00 o'clock, 19.10 m From Shaft.

Figure A 53 – Flange Connection at 1:30 o'clock, 16.86 m from Shaft.

Figure A 54 – Flange Connection at 9:00 o'clock, 16.86 m from Shaft.

Figure A 55 –Flange Connection at 9:00 o'clock, 16.86 m from Shaft.

Figure A 56 – View of Tunnel, Looking Westward, 16.86 m From Shaft.

Figure A 57 – View of Tunnel, Looking Westward, 16.86 m From Shaft.

Figure A 58 – Galvanized Pipe Strap and Pedestal, 13.58 m From Shaft.

Figure A 59 – Feeder Main Pipe on Pedestal, 13.58 m From Shaft. Suspect rust staining is from pedestal reinforcing.

Figure A 60 – Feeder Main Pipe on Pedestal, 13.58 m From Shaft. Note graphite paste pipe cushion along surface of coating.

Figure A 61 – Victaulic Connection at 12:30 o'clock, 12.59 m from Shaft.

Figure A 62 – Victaulic Connection at 2:00 o'clock, 12.59 m from Shaft.

Figure A 63 –Victaulic Connection at 6:00 o'clock, 12.59 m from Shaft.

Figure A 64 –Victaulic Connection at 10:00 o'clock, 12.59 m from Shaft.

Figure A 65 –Victaulic Connection at 8:00 o'clock, 12.59 m from Shaft.

Figure A 66 – Coating Dimple at 1:00 o'clock, 10.50 m From Shaft.
Possible dent in pipe.

Figure A 67 – Flange Connection at 2:00 o'clock, 8.32 m from Shaft.

Figure A 68 – Flange Connection at 5:00 o'clock, 8.32 m from Shaft.

Figure A 69 –Flange Connection at 10:00 o'clock, 8.32 m from Shaft.

Figure A 70 – Galvanized Pipe Strap and Pedestal, 6.48 m From Shaft.

Figure A 71 – Galvanized Pipe Strap and Pedestal, 6.48 m From Shaft.

Figure A 72 – Corrosion / Perforation of Aluminum Air Pipe, 7.30 m From Shaft. Water infiltrating tunnel at concrete pour joint near 6.48m from shaft.

Figure A 73 – Victaulic Connection at 2:30 o’clock, 4.60 m from Shaft.

Figure A 74 – Victaulic Connection at 5:30 o’clock, 4.60 m from Shaft.

Figure A 75 –Victaulic Connection at 9:30 o'clock, 4.60 m from Shaft.

Figure A 76 – Base Elbow at Shaft, 2:00 o'clock.

Figure A 77 – Underside of Lower Flange Connection on Base Elbow at Shaft, 4:00 o'clock.

Figure A 78 – Upper Flange Connection on Base Elbow at Shaft, 4:00 o'clock.

Appendix H

PICA Inspection Reports

City of Winnipeg

600 mm Kildonan-Redwood Feeder Main

Condition Assessment Report, Standard Analysis

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

24in Potable Water Chimera RFT ILI Tool
600mm CML Spiral Welded Steel Pipe

Red River Crossing – RFP 495-2018
Redwood Ave from Main St to Glenwood Cr
Winnipeg, Manitoba

PICA Project: 8054

Inspection Date: March 26-27, 2019
Report Submission: August 26, 2019; June 11, 2019 (rev 1.0)
Operators: D. Burton, A. Russell, P. Ryhanen, B. Senka, A. Shatat
Analyst: A. Liwoch
Reviewers: M. Chia, A. Shatat
Report Revision: 1.0

CONFIDENTIAL

Table of Contents

City of Winnipeg:

600mm Kildonan-Redwood Feeder Main

Condition Assessment Report, Standard Analysis

Executive Summary

PICA, under contract with the City of Winnipeg (RFP 495-2018), inspected two 24in river crossing feeder mains for the City of Winnipeg using Remote Field Testing (RFT) Technology from March 26-28, 2019. The inspected lines are referred to as the Kildonan-Redwood Feeder Main and the Charleswood-Assiniboia Feeder Main. This report documents the inspection results for the 24in Kildonan-Redwood line, which crosses beneath the Red River. The inspected portion spanned between chambers on the east and west sides of Red River (refer to the line map in Figure 4). The inspection was performed on March 26 and 27, 2019, with a supplementary visual inspection of accessible piping performed on March 30th. This report documents PICA's findings.

In general, the feeder main was found to be in good condition. Nine (9) pipes were found to show evidence of pitting corrosion with a total of 18 localized pitting indications reported. Of these 18 indications, 11 indications measured to be "shallow" (≥65% RW), and 7 indications measured to be "medium" (40-64% RW).

A listing of all logged anomalies together with detailed analysis information can be found in the companion document, "*PICA Inspection Results - 24in Kildonan Feeder Main (rev1.1).xlsx*". Figure 1 and Figure 2 illustrate the axial and circumferential distribution of localized defect indications along the Kildonan-Redwood Feeder Main. Note that some data points partially overlap due to proximity. Clock position information cannot be provided for the low confidence indication within the vertical pipe 0340, so it is not shown in Figure 2.

A condition assessment summary detailing the top three defect indications, minimum and maximum circumferential remaining wall, and pipe average remaining wall (PARW) values for each pipe segment (greater than 2.11m in length) of the Feeder Main river crossing is provided in Figure 3. One pipe segment (0210) had a recorded average wall thickness more than 115% remaining wall, indicating a different pipe type with heavier wall thickness compared to those specified in construction records provided to PICA. Pipe segment 0340 is a single vertical pipe located in the chamber shaft on the west side of the Red River. This pipe segment has a greater nominal wall thickness value (12.7mm) than the other pipes (7.9mm) and was therefore scanned using a different frequency during the RFT inspection.

Various areas of interest were identified during a visual inspection of accessible piping within the Tunnel located on the west side of the river. These areas of interest included damage to the coating in the form of gauges and scratches, and imprints possibly left from transport or construction equipment. No visual damage to the external coating was observed at the anomaly locations flagged in the RFT data.

Table 1. Overview of the RFT findings for the 600mm Kildonan-Redwood Feeder Main

Table 1: Feature Indication Summary	
Inspected Length	237.76m
Number of Pipe Sections:	34
Number of Analyzed Pipe Sections:	34
Number of Elbows:	One 90° (Long Rad. CI, 25.4mm NWT)
Thinnest circumferential pipe wall (Tcircmin) (RW%):	96% (in pipe 0140)
Number of pipes without localized wall loss indications:	25
Number of pipes with localized wall loss indications:	9
• **Number of indications with >65% RW:**	11
• **Number of indications with 40-65% RW:**	7
• **Number of indications with <40% RW:**	0
Total number of wall loss indications reported:	18
Total Number of Connections:	34
• **Number of Flange-Pair Connections:**	25
• **Number of Victaulic Couplings:**	5
• **Number of Dresser Couplings:**	1
• **Number of Welds**	3
Number of Open Flange Faces:	1

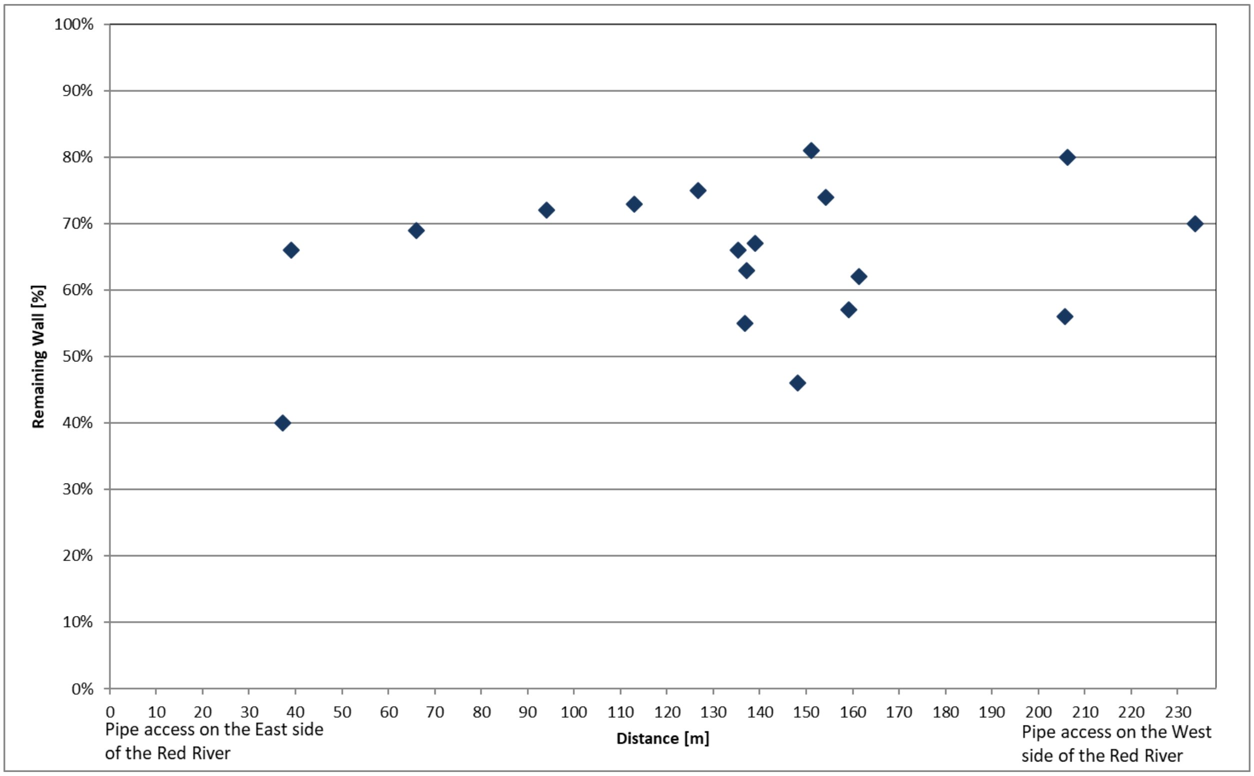


Figure 1. Axial distribution of defect indications and remaining wall (%) within the scanned length of the Kildonan-Redwood Feeder Main

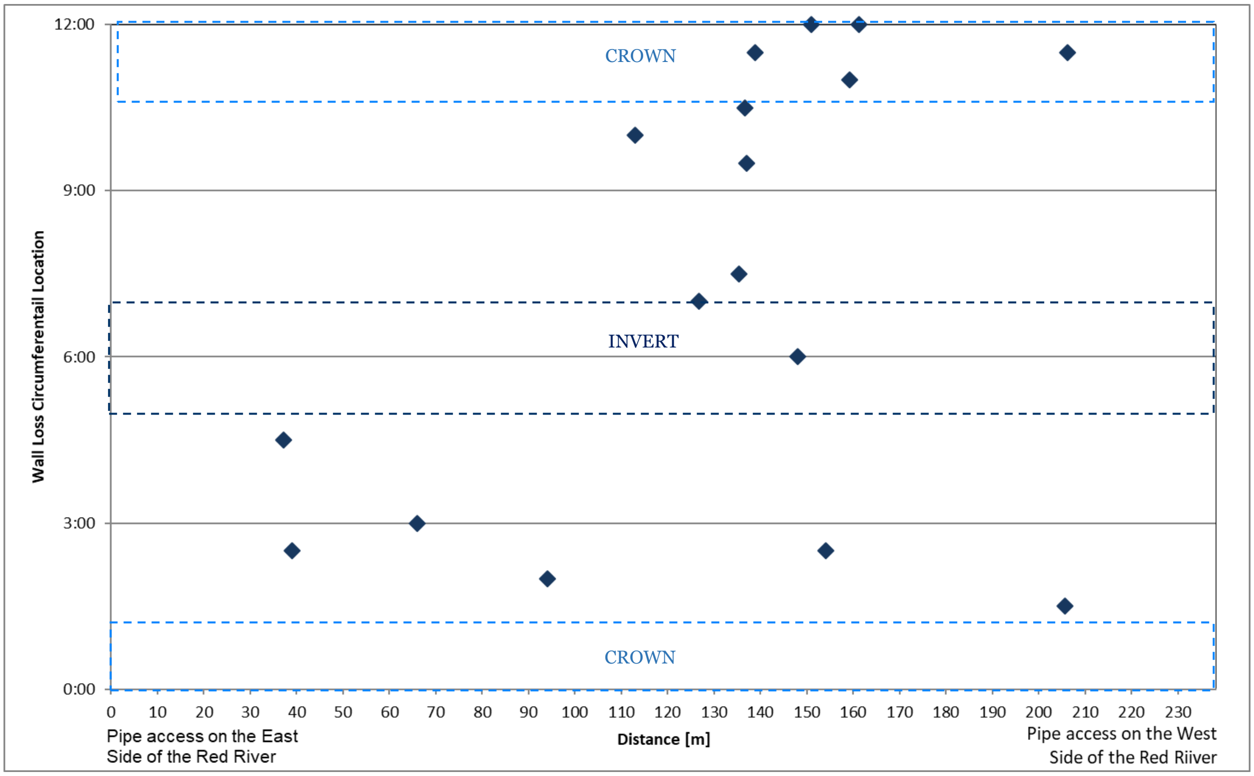


Figure 2. Circumferential distribution of pitting regions along the Kildonan-Redwood Feeder Main, described with clock positions referenced by looking from east to west. Note that the defect reported in pipe 0340 is not included in chart as clock position information is not available for this pipe.

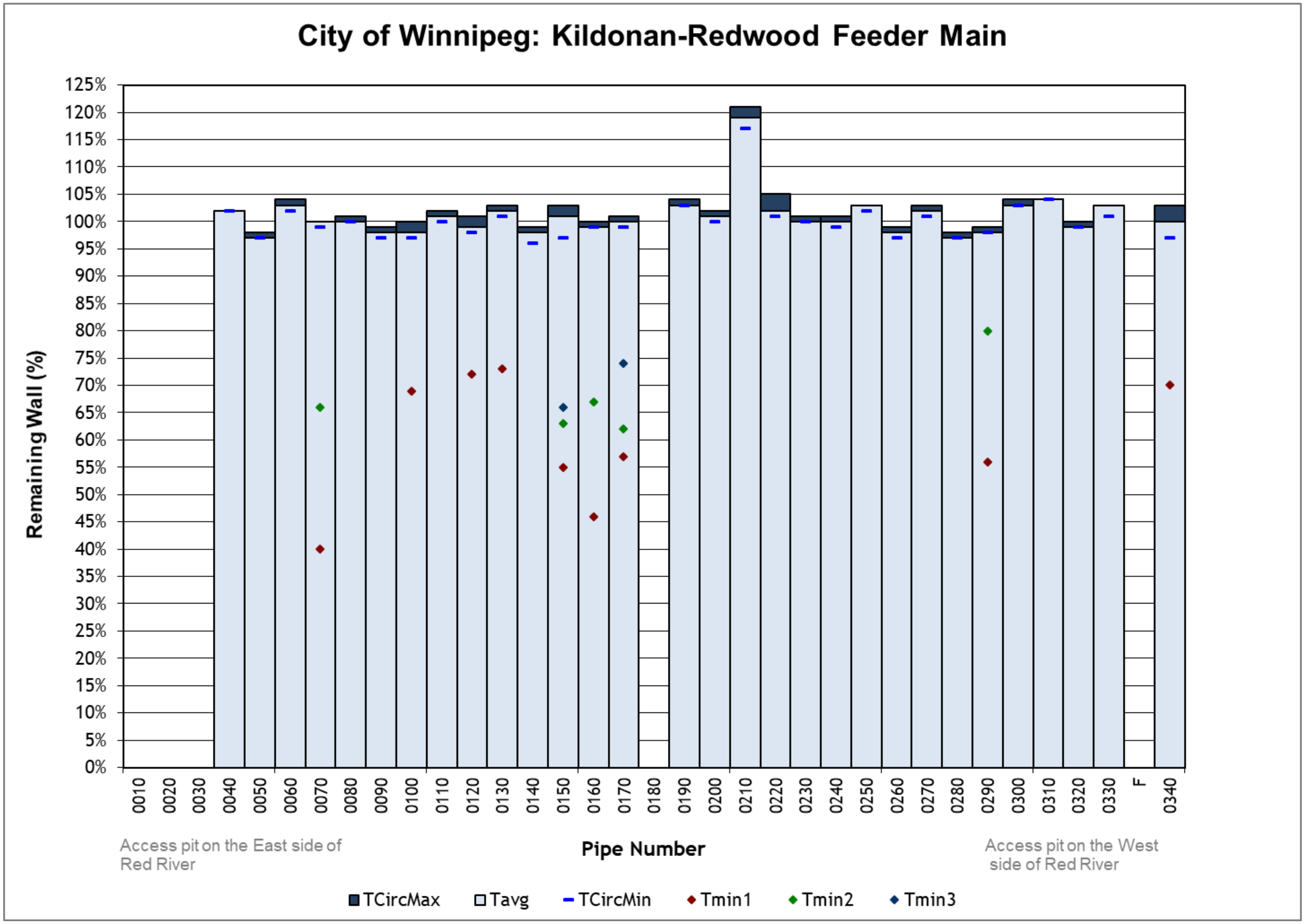


Figure 3. Condition assessment summary for the Kildonan-Redwood Feeder Main. Pipes less than 2.11 meters in length were not analyzed for pipe average remaining wall values. Pipe 0340 is the vertical pipe on the west side of the Red River and has a wall thickness of 12.7mm, differing from 7.9mm pipe wall thickness which was documented for the horizontal piping in drawing 12530. This pipe's Tavg is set to 100%.

Pipeline Inspection Background

The Kildonan-Redwood Feeder Main is a 600mm (24-inch) diameter steel main transporting potable water. A dual tethered Remote Field Technology (RFT) inspection of the feeder main was conducted by PICA on March 28th, 2019. The section inspected by PICA crosses beneath the Red River and extended between two access chambers east and west of the river (located along Redwood Avenue).

Table 2. Pipeline and RFT inspection information for the 600mm Kildonan-Redwood Feeder Main river crossing

Client:	City of Winnipeg	
Location:	Redwood Ave from Main St to Glenwood Cr Winnipeg, Manitoba	
Line Name/ Identifier:	Kildonan-Redwood Feeder Main	
Product:	Potable Water	
Pipe Diameter:	600 mm (24-inch)	
Material:	Steel, spiral welded; One Cast Iron elbow (90°)	
NWT:	(Steel) 7.9 mm (horizontal piping) and 12.7 mm (vertical pipe)	
Grade:		
Internal Liner:	Coal Tar Epoxy, ~4mm thick	
External Coating:	Asphalt dipped felt	
Bends:	Mitered: Two 20°; 13°, 7°, 12°; Radiused: one LR CI 90° elbow	
Joint Type:	Welding Flanges; Victaulic couplings; Welds	
Age:	65 yrs. (1954)	
RFT Inspection Access Locations:	Chambers east and west of river, along Redwood Ave	
	Elevation	**GPS Coordinates**
East Chamber:	231m	49°54’56.81”N, 97°07’30.55”W
West Chamber:	232m	49°54’59.55”N, 97°07’40.96”W
RFT Inspection Length:	**Vertical Portion:** 10.80m	**Horizontal Portion:** 225.60m
Reported Inspection Direction:	East to West	

The Kildonan-Redwood Feeder Main was divided into two inspection lengths separated by a 90-degree cast iron elbow. Due to the unknown ID of the cast iron elbow, it was not attempted to pull the Chimera tool through the elbow. The 90-degree cast iron elbow is located at the bottom of the West Chamber, within an access tunnel that extends approximately 49.02 meters southeast towards the river. The 600 mm Feeder Main runs through this tunnel, and past the end of the tunnel to the opposite bank of the Red River.

The two inspection lengths are:

1) Vertical pipe section at the West Chamber (10.80m length) extending down to the 90-degree elbow at the bottom of the vertical shaft.
2) Horizontal piping (225.60m length) from the East Chamber to the east facing flange face of the 90-degree cast iron elbow at the bottom of the vertical shaft at the West Chamber.

Figure 4 shows an overview map of the Kildonan-Redwood Feeder Main section inspected by PICA that crosses beneath the Red River, and the profile drawing of the Feeder Main section.

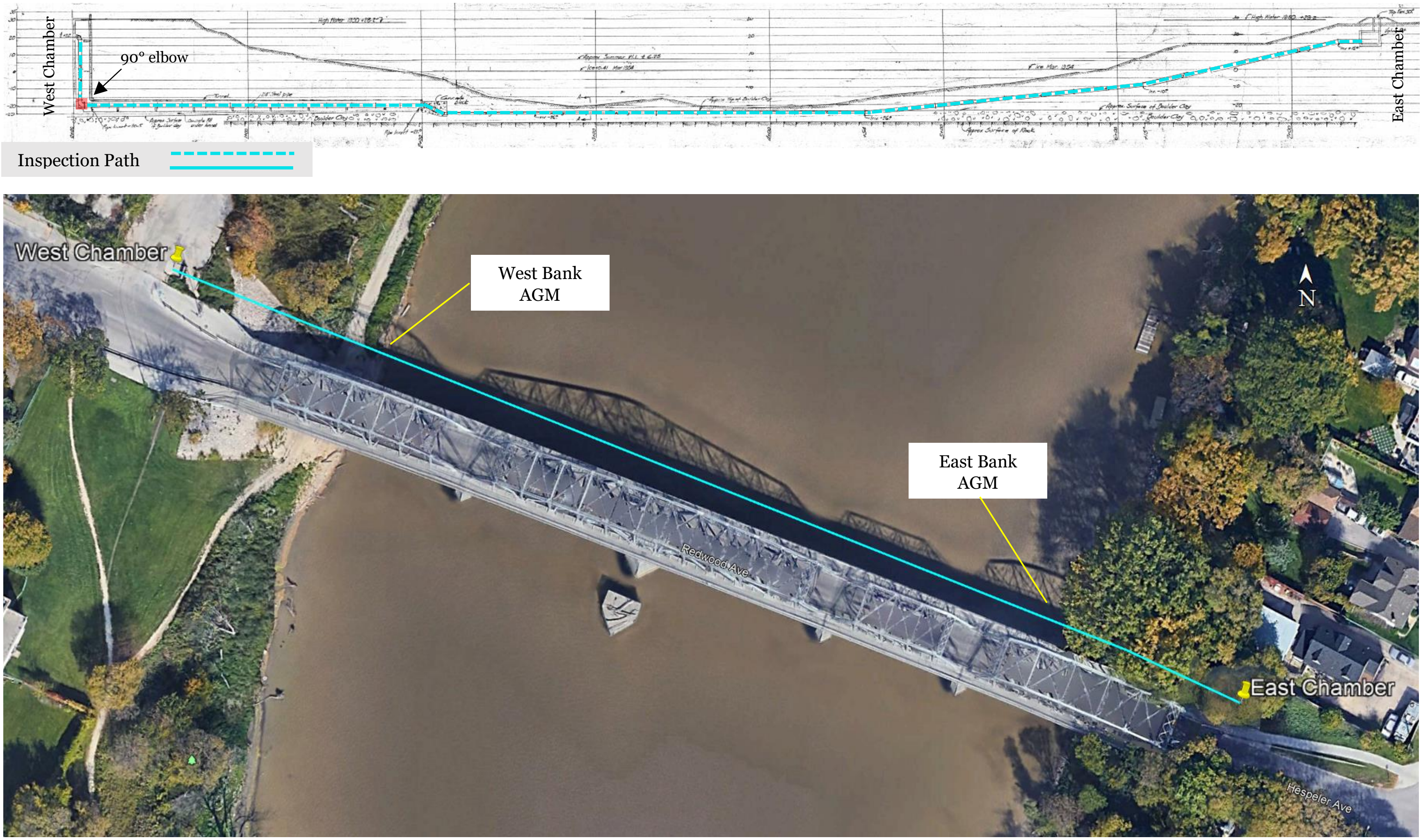


Figure 4. Profile drawing (Drawing No. 12529) and overview path map (Google Earth) of the Kildonan-Redwood Feeder Main that was inspected by PICA during the March 2019 mobilization, which crosses beneath the Red River (source: Google Earth)

Inspection Details

RFT Inspection Preparation

Prior to performing the RFT inspection, all available Critical Application Information (CAI) was reviewed including drawings and measurements of pipes and wall thicknesses provided by the client and subcontractor to ensure a successful inspection.

In advance of the RFT in-line inspection (ILI), the pipeline section was isolated, and a line cleaning was performed. A gauge pig was pulled through the horizontal portion of the Main by J-Con to confirm the minimum bore of the section and ensure Chimera RFT tool passage. A tagline was left through the entire length of the river crossing after preparatory activities for use during the RFT inspection of the Main.

To facilitate the inspection, winches were positioned at the East and West Chambers. The winchline at the West Chamber was connected to the tagline in the pipe. The tagline and connected winchline from the West side winch were pulled up on the East side of the river to be connected to the Chimera RFT ILI tool for the dual tethered inspection runs.

Before contacting the Main, the RFT Chimera tool assembly was sprayed with a 200mg/L free chlorine disinfecting solution (Figure 5). The tool was then lowered into the West Chamber to begin the inspection of the vertical piping section (Figure 6).

Figure 5. Disinfecting the Chimera assembly using a 200 mg/L free chlorine solution

Figure 6. Chimera tool being lowered into the chamber on the west side of the river crossing

RFT Inspection Procedure

Vertical Pipe at West Chamber

The vertical portion (on the west side of the river) of the Kildonan-Redwood Feeder Main was inspected first. The winch positioned at the West Chamber (Figure 8) was used to lower the tethered tool into the chamber and into the vertical piping (Figure 7). The winchline odometer as well as the on-board odometers were zeroed with the Chimera tool's odometer wheels flush with the pipe opening. For the first run, the RFT tool's detector module was leading (first to enter pipe) scanning at a frequency of 7 Hz.

Once the tool reached the 90-degree cast iron at the bottom of the vertical piping, the winch was stopped, and the pull was reversed to bring the Chimera tool back up to be extracted from the pipe.

Figure 7. Looking down into West Chamber from above. Chimera tool is in vertical steel pipe.

Figure 8. Winch set-up on west side of Red River.

Figure 9. East side of Red River winch set-up.

The total length of the vertical portion of the Main is documented as 10.91m, and was measured by the Chimera RFT tool's odometers as spanning 10.80 meters in length. Figure 10 shows an image taken from the tunnel showing the vertical piping extending from the tunnel up to the West Chamber.

A second run was performed with the Chimera's exciter module leading, and a third run with the detector module again leading, both runs completed at a frequency of 5 Hz. Once the tool was received for the final time at the top of the vertical shaft in the West Chamber, data was downloaded, and the Chimera was disconnected from the winchlines. The Chimera RFT tool was brought to the East Chamber to begin the inspection of the horizontal portion of the Feeder Main.

Figure 10. Looking up vertical shaft from tunnel. Image shows the vertical steel pipe portion of the Kildonan-Redwood Feeder Main.

Horizontal Pipe from East Chamber to Cast Iron Elbow

A second winch was positioned at the East Chamber (Figure 9) to enable a dual tethered inspection of the horizontal section of the Feeder main.

First, the winchline from the winch at the West Chamber was pulled through the pipeline to allow connection to the tool in the East Chamber.

The tool was lowered into the chamber on the East side of the river (Figure 11). Winchlines from the East and West access locations were connected to either end of the Chimera tool and the tool was then inserted into the pipe in the East Chamber with the detector module leading (facing west).

Figure 11. Chimera tool being lowered into East Chamber to inspect the horizontal portion of the Feeder Main.

Using the winches, the Chimera was pulled from the East Chamber (Figure 12, Figure 13) west towards the 90-degree elbow in the tunnel on the west side of the Red River. The tool scanned at a frequency of 14 Hz, traveling at a velocity of 2.1 m/min. Once the tool had reached the elbow, the pull was reversed, and the Chimera was brought back to the East Chamber. The distance the tool traveled (in one direction) for this portion of the pipeline was 225.60 meters, with data being collected successfully over the entire inspected length. The length of the inspected portion was measured from the open pipe end in the east chamber.

Figure 12. Chimera with trailing pig loaded into the pipe in the East Chamber.

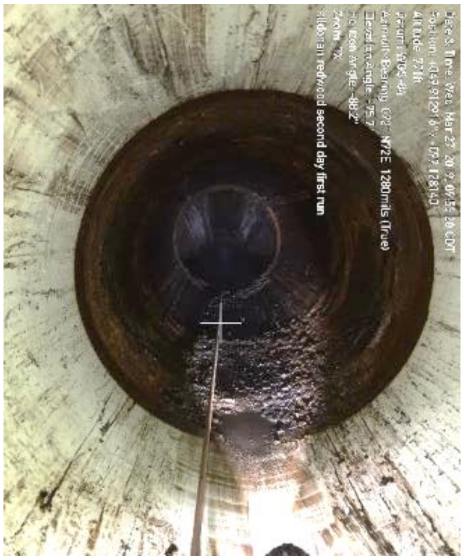


Figure 13. Looking into pipe from the East Chamber towards first downward deflection.

Due to the use of flange connections in the construction of this portion of the Feeder Main, multiple inspection runs were necessary to collect RFT data for the full length of the pipeline. As a result, two additional runs were completed on the following day; one inspection run was conducted at 14Hz, with the tool's exciter module leading, and one 10Hz inspection run was conducted with the detector module leading, targeting the tunnel section of the force main.

During the inspection of the horizontal section of the Feeder Main, PICA's custom Above Ground Monitors (AGMs) were used as an additional source of positional information. AGM units were positioned above the pipeline on both the east (Figure 14) and west (Figure 15) banks of the Red River so that the underwater portion of the Feeder Main could be identified in the data during analysis and reporting. These units are designed to pick up the signal from the RFT ILI tool and are used to track the location and time of passage of the tool during inspections.

Figure 14. AGM unit on the east side of the Red River, recording tool passage

Figure 15. AGM units on the west side of the Red River

Following the completion of the RFT inspection, a visual inspection was conducted on the horizontal portion of the Main that runs through the tunnel (Figure 16) on the west side of the Red River. Measurements were taken of construction features such as the straps, Victaulic couplings and flange pair connections (Figure 17). Details about the visual investigation can be found in the section ***Analysis Results – Visual Inspection***.

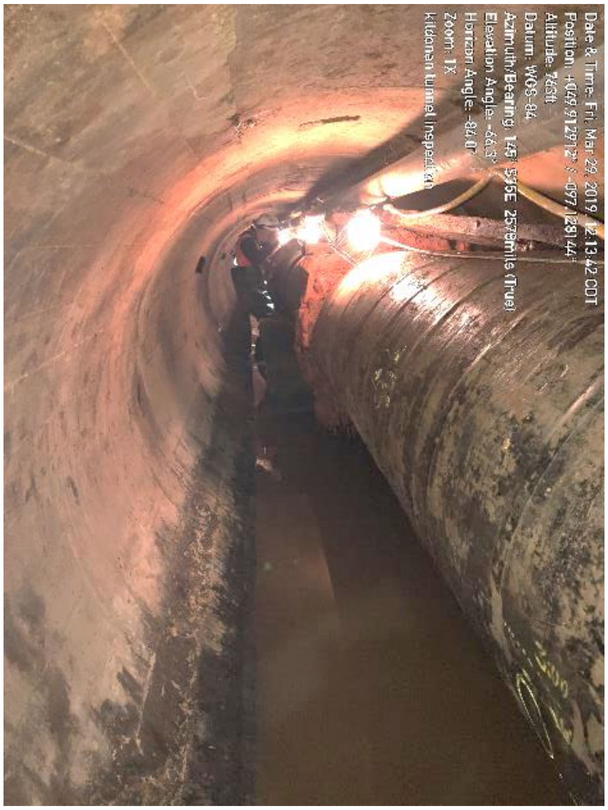


Figure 16. Visual investigation of the horizontal portion of the Kildonan-Redwood Feeder Main within the tunnel on the west side of the river.

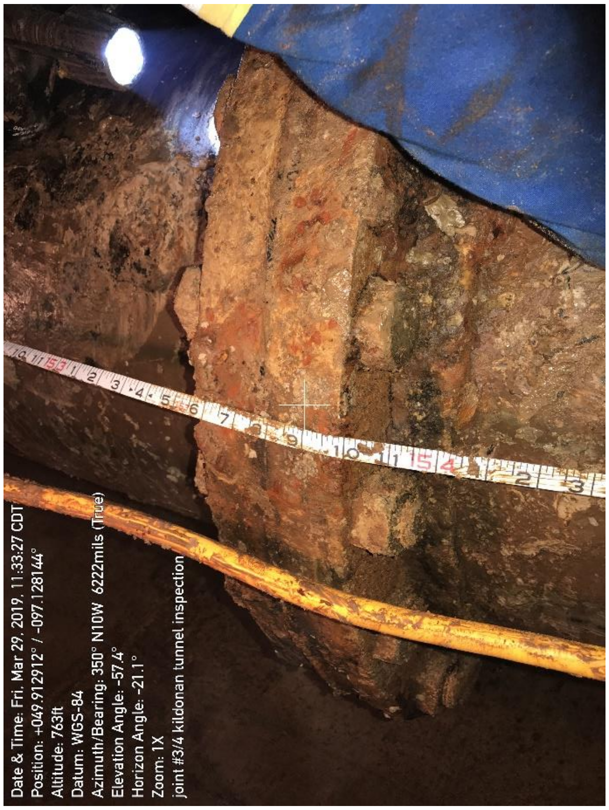


Figure 17. Measurements were taken of construction features within the horizontal portion of the pipe in the tunnel on the west side of the river.

Analysis Results

For reporting purposes, the Zero Reference Datum (ZRD) point was set to the open face of the pipe in the East Chamber. The End Reference Datum (ERD) was established at the open flange face at the top of the vertical pipe in the West Chamber. Pipe segments are therefore reported from east to west, and pitting indications described with clock positions referenced by facing west. Distances stated in this report refer to the distance from the ZRD at the East Chamber unless otherwise noted.

Multiple data sets were collected during the inspection of the Kildonan-Redwood Feeder Main. Three runs were completed within the vertical pipe section at the West Chamber, resulting in six data sets being collected. These runs were performed at 5Hz and 7Hz frequencies, and with the tool's orientation flipped for one of the runs.

Three runs were completed within the horizontal piping section of the Feeder Main, which also resulted in six data sets being collected. Two runs were performed at 14Hz with the detector and exciter orientation alternating, and one run was performed at 10Hz, with the tool's detector leading. For the 14Hz inspections, only the launch portions (east to west runs), were acquired at the optimal inspection velocity. The 10Hz inspection run targeted the tunnel section of the force main, and an optimal inspection velocity was achieved for the launch portion of the inspection run from approximately 127.28m to the 90-degree elbow.

The three data sets gathered at the proper inspection velocities were compared and analyzed for defect indications, with recorded indications correlated between the data sets. This provides a high level of confidence in the location and sizing of the reported defect indications. Odometer and Inertial Measurement Unit (IMU) data were compared between all six data sets.

Location Reporting, Pipe Lengths & Features

Resource information collected and reviewed during the inspection project was used to supply the most accurate summary of positional information for each of the reported potential wall loss indications. This information included data collected by an Inertial Measurement Unitwhich recorded the pitch, yaw and roll of the Chimera tool as it was pulled through the pipeline; AGM passage locations; and physical measurements of accessible piping. This information was compared with the client supplied drawings of the Feeder Main and was used for corroboration of reported construction feature and wall loss indications.

A visual inspection was performed on the accessible portion of the Main that runs through the tunnel on the west side of the Red River. During the visual inspection, measurements of pipe lengths, connection types, and other key areas of interest were taken. Pipe segment lengths recorded with the Chimera tool's on-board odometers correlate well with the physically measured pipe segments within the tunnel.

The total length inspected of the Kildonan-Redwood Feeder Main was 237.76m. The Main was constructed with pipe segments of various lengths, most measuring around either 12.15m (~40.9 feet) or 4.25m (~13.9 feet). Some discrepancies were found between the client records and RFT findings. The length of the horizontal portion of the Feeder Main was expected to be 231.34 meters; however, during review of the data collected during the inspection, the length was measured to be 225.60m. This 5.74m difference is thought to be mostly due to errors within the construction records, partially related to the piping modifications made for access in the East Chamber (pipe 0010).

Pipe number 0330 measured 2.0m shorter in length than described in client records (detail “11” in drawing No. 12530). Physical measurements were taken during the visual inspection, which confirmed the reported length of pipe 0330 to be 3.95m (instead of 6.09m).

The length of the 90-degree elbow at the bottom of the west shaft was estimated at 1.36m, based on the dimensions of an AWWA 90° long sweep bend found in the *CIPRA 2952 Cast Iron Pipe Handbook*.. The actual length of the elbow may differ from this estimated centerline length.

Pipe segments were connected primarily with flanges, though five Victaulic Couplings and one Dresser Coupling were also identified in client provided drawings. Record drawing number 12530 details two pipe segments joined with a Dresser Coupling (detail “5”), spanning 12’0” (3.66m) together. These two pipe segments measured 1.54 and 2.37 meters in length respectively, using odometer data that was recorded during the RFT inspection.

Two connections at the east side of the river crossing are thought to be welds, concurrent with suspected Dresser Style 63 connection rings. These two connections join pipe segments 0010 with 0020, and 0020 with 0030. Pipe 0010 is the white-painted pup piece that was welded to the main in the East Chamber. This pipe segment measures approximately 0.7m in length (please note that odometer data collected at the very beginning of the pipeline is poor in quality due to inconsistencies in the velocity of the RFT tool as it is brought up to the target inspection velocity). Figure 18 shows an image of the pup piece photographed in the East Chamber during one of the inspection runs.

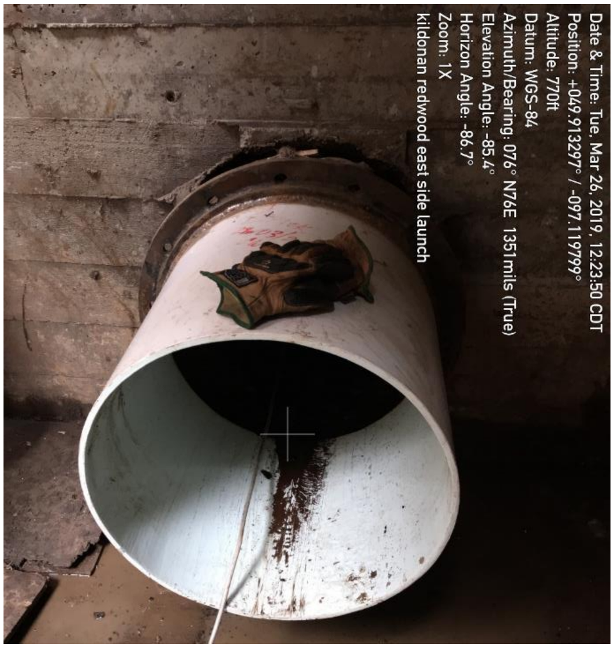


Figure 18. Short segment of pipe in the East Chamber.

A third girth weld was identified between pipes 0030 and 0040. At the connection between these two pipe segments, a pitch change of -6.0 degrees was recorded by the on-board IMU module, while a -5.6-degree pitch change was recorded at the flange connection between pipes 0040 and 0050. Client records provided to PICA report a 12-degree vertical bend between pipes detailed “9” (PICA pipe number 0050) and “10” (pipe 0040) (drawing 12530). This documented 12-degree bend is thought to be inaccurate, as the RFT weld indications as well as the corresponding pitch changes were recorded in all data sets collected during the condition assessment. Table 3 provides a summary of construction features noted in collected data from the inspection of the Kildonan-Redwood Feeder Main.

Table 3. Summary of construction features recorded during the RFT inspection of the 600mm Kildonan-Redwood Feeder Main. Connections not mentioned in table are flange pairs which make up most pipe connections.

Distance (m)	Pipe Number	Description
0.68	0020	Weld
1.43	0030	Weld
2.65	0040	Weld
163.75	0190	Dresser Coupling
166.58	0200	Possible Water Stop
172.62	0210	Strap
174.68	0220	Victaulic Coupling
175.31	0220	Possible Water Stop
180.60	0230	Strap
186.74	0240	Strap
187.49	0250	Victaulic Coupling
193.56	0260	Strap
199.73	0270	Strap
200.29	0280	Victaulic Coupling
206.41	0290	Strap
212.52	0300	Strap
213.10	0310	Victaulic Coupling
219.20	0320	Strap
221.64	0330	Victaulic Coupling
226.28	Between 0330, 0340	90-Degree Elbow**
229.96	0340	Stiffener
232.38	0340	Stiffener
235.11	0340	Stiffener

*Connection types refer to start of pipe number given

**90-degree elbow length is estimated based on the dimensions of an AWWA 90˚ long sweep elbow as documented on pg266 of the CIPRA 1952 Cast Iron Pipe Handbook.

General Wall Thickness

Pipe sections longer than 2.11m were analyzed to obtain the Pipe Average Remaining Wall (PARW) thickness calculated over the length of the inspected section. This value is reported as the "Tavg" RW in Table 4. The Chimera's sensor-exciter spacing (SES) is 2.11m, therefore pipe segments shorter than 2.11m were not analyzed for Tavg to ensure that the Chimera was not spanning between two or more separate pipes.

Due to manufacturing tolerances, fluctuations of ±15% in the individual PARW values are common. Variations outside the ±15% spread can be an indicator of a different nominal wall thickness or pipe type or point towards a problem like aggregate pitting or general wall loss. All pipes that were analyzed in the 600mm (24in) Kildonan-Redwood Feeder Main fall within this tolerance allowance, except pipe 0210. This pipe presented a clear baseline shift to a higher wall thickness which was observable in all data sets. This suggests a different pipe type with a larger nominal wall thickness (NWT) was used for stick 0210 in place of a pipe with a NWT of 7.9mm.

The vertical pipe segment 0340 was scanned with a lower frequency as the pipe has a thicker NWT value (12.7mm) than the majority of the Main (7.9mm). The Tavg for this pipe segment was 12.7mm (100%).

Local Wall Thickness

Nine (9) pipes show evidence of pitting corrosion with a total of 18 localized pitting indications reported. Of these 18 indications, 11 indications measured to be "shallow" (≥65% RW), and 7 indications measured to be "medium" (40-64% RW). Five of these indications have been reported with low confidence. Confidence levels are assigned based on signal strength, noise levels, signal reproducibility between RFT recordings, and sizing consistency between the recordings.

Table 4 details the three worst pitting indications per pipe (Tmin1, Tmin2 and Tmin3), as well as the average (Tavg), minimum circumferential (Tcircmin) and maximum circumferential (Tcircmax) remaining wall values for the inspected portion of the Kildonan-Redwood Feeder Main.

Colour maps can highlight wall loss, wall gain, and material stresses within the pipe. Figure 24 to Figure 34 in Appendix 1 show colour maps of the data for the medium-high confidence wall loss indications. Defect indications are highlighted in the figures with bounding boxes.

If AECOM and the City decide to perform verification and repair work on the onshore portions of the feeder main crossing, please let your PICA representative know. PICA can assist by providing dig sheets for the selected areas.

Data Quality

Prior to the inspection of the Kildonan-Redwood Feeder Main, the 24-inch Chimera RFT ILI tool was calibrated using a calibration pipe at PICA's shop. Details about the calibration can be found in Appendix 2.

Flange connections are difficult to assess. Flange pairs produce very large signals in the RFT data due to the amount of material present; these large flange signals can mask small wall loss indications at or adjacent to the connections. To mitigate the effects of the large RFT signal indications from flange pairs, the Chimera tool was pulled through the Main alternating between the detectors leading and exciter leading for each run. This maximizes the area of analyzable data collected on either side of these connections.

The nominal pipe wall thickness of the vertical pipe is considerably thicker than the calibration pipe. As a result, signal-to-noise levels are lower, and the one defect identified in the vertical portion is reported with low confidence.

Table 4. Summary of pipe tally and wall thickness readings for the Kildonan-Redwood Feeder Main

Pipe List and Wall Thickness Readings – 600 mm (24in) Kildonan-Redwood Feeder Main

PICA Pipe #	Pipe Location***			Tavg RW	Circumferential Wall Thickness		Local Wall Thickness**									Comments
							Tmin1			Tmin2			Tmin3			
	Start (m)	End (m)	Length (m)		Tcircmax RW	Tcircmin RW	RW	Location (m)	Clock Position**	RW (%)	Location (m)	Clock Position	RW (%)	Location (m)	Clock Position	
0010	0.00	0.68	0.68													Prelim zero datum is the pipe opening at the east end of the line
0020	0.68	1.43	0.75													Start weld concurrent with suspected Dresser Style 63 Connection Ring
0030	1.43	2.65	1.22													Start weld concurrent with suspected Dresser Style 63 Connection Ring
0040	2.65	5.75	3.10	102%	102%	102%										-6.0° mitered vertical bend at start of segment
0050	5.75	16.39	10.64	97%	98%	97%										-5.6° mitered vertical bend at start of segment
0060	16.39	28.57	12.18	103%	104%	102%										
0070	28.57	40.62	12.05	100%	100%	99%	40%	37.20	4:30	66%	39.00	2:30				Two defect indications reported
0080	40.62	52.69	12.07	100%	101%	100%										mitered 6.2° vertical bend and 8° horizontal bend left (south) at start of segment

Pipe List and Wall Thickness Readings – 600 mm (24in) Kildonan-Redwood Feeder Main

PICA Pipe #	Pipe Location***			Tavg RW	Circumferential Wall Thickness		Local Wall Thickness**									Comments
	Start (m)	End (m)	Length (m)		Tcircmax RW	Tcircmin RW	Tmin1			Tmin2			Tmin3			
							RW	Location (m)	Clock Position**	RW (%)	Location (m)	Clock Position	RW (%)	Location (m)	Clock Position	
																East side AGM is located at 46.51m
0090	52.69	64.85	12.16	98%	99%	97%										
0100	64.85	77.06	12.21	98%	100%	97%	69%	66.03	3:00							One defect indication reported
0110	77.06	89.22	12.16	101%	102%	100%										
0120	89.22	101.40	12.18	99%	101%	98%	72%	94.15	2:00							6.0° mitered vertical bend One defect indication reported
0130	101.40	113.59	12.18	102%	103%	101%	73%	113.01	10:00							One defect indication reported
0140	113.59	125.75	12.16	98%	99%	96%										
0150	125.75	137.94	12.19	101%	103%	97%	55%	136.74	10:30	63%	137.15	9:30	66%	135.39	7:30	Four defect indications reported
0160	137.94	150.14	12.20	99%	100%	99%	46%	148.13	6:00	67%	138.92	11:30				Two defect indications reported
0170	150.14	162.22	12.08	100%	101%	99%	57%	159.19	11:00	62%	161.37	12:00	74%	154.17	2:30	Four defect indications reported
0180	162.22	163.76	1.54													30.0° mitered vertical bend at start of segment
0190	163.76	166.14	2.37	103%	104%	103%										-30.0° mitered vertical bend at end of segment. Possible water stop at 166.58m.

Pipe List and Wall Thickness Readings – 600 mm (24in) Kildonan-Redwood Feeder Main

PICA Pipe #	Pipe Location***			Tavg RW	Circumferential Wall Thickness		Local Wall Thickness**									Comments
	Start (m)	End (m)	Length (m)		Tcircmax RW	Tcircmin RW	Tmin1			Tmin2			Tmin3			
							RW	Location (m)	Clock Position**	RW (%)	Location (m)	Clock Position	RW (%)	Location (m)	Clock Position	
0200	166.14	170.43	4.29	101%	102%	100%										West side AGM is located at 173.54m
0210	170.43	174.68	4.25	119%	121%	117%										Strap located at 172.6m. Possible water stop at 175.31m
0220	174.68	178.97	4.29	102%	105%	101%										
0230	178.97	183.24	4.27	100%	101%	100%										Strap located at 180.6m
0240	183.24	187.50	4.26	100%	101%	99%										Strap located at 186.7m
0250	187.50	191.76	4.26	103%	103%	102%										
0260	191.76	196.03	4.26	98%	99%	97%										Straps located at 193.6m
0270	196.03	200.30	4.27	102%	103%	101%										Strap located at 199.7m
0280	200.30	204.57	4.27	97%	98%	97%										
0290	204.57	208.84	4.27	98%	99%	98%	56%	205.66	1:30	80%	206.24	11:30				Strap located at 206.4m Two defect indications reported
0300	208.84	213.11	4.26	103%	104%	103%										Strap located at 212.5m
0310	213.11	217.37	4.26	104%	104%	104%										
0320	217.37	221.65	4.28	99%	100%	99%										Strap located at 219.2m
0330	221.65	225.60	3.95	103%	102%	101%										
F	225.60	226.96	1.36*													90° elbow

Pipe List and Wall Thickness Readings – 600 mm (24in) Kildonan-Redwood Feeder Main

PICA Pipe #	Pipe Location***			Tavg RW	Circumferential Wall Thickness		Local Wall Thickness**									Comments
							Tmin1			Tmin2			Tmin3			
	Start (m)	End (m)	Length (m)		Tcircmax RW	Tcircmin RW	RW	Location (m)	Clock Position**	RW (%)	Location (m)	Clock Position	RW (%)	Location (m)	Clock Position	
0340	226.96	237.76	10.80	100%	103%	97%	70%	232.84	Not available							Vertical west shaft. One defect indication reported. End datum at open flange face in west chamber.

*90-degree elbow length is estimated based on the dimensions of an AWWA 90˚ long sweep elbow as documented on pg266 of the CIPRA 1952 Cast Iron Pipe Handbook.

**Clock positions are reported clockwise facing west. Clock positions are not available for defects in the vertical west pipe segment.

***Reported pipe segment lengths may not all be consistent with the segment start and end locations due to rounding.

Visual Inspection

A visual inspection was performed by PICA on the Kildonan-Redwood Feeder Main that was accessible within the tunnel on the west side of the Red River. The portion of the Main that runs through the tunnel is comprised of eleven (11) full pipe segments, eight (8) flange pairs, four (4) Victaulic Couplings, seven (7) straps, and a total measured distance of 49.02m from the end of the tunnel to the center of the flange connection between pipe 0330 and the 90-degree vertical elbow.

Asphalt-dipped felt was used to coat the exterior of the pipeline during construction of the Main. Though this coating was found to be in good condition over most of the accessible length, multiple areas of interest were identified and recorded. These areas of interest included damage to the coating in the form of gouges and scratches; imprints possibly left from transport or construction equipment; and potential repair points. Figure 19 to Figure 23 show some of the anomalies that were observed during the visual inspection. Note that the distances stated are measured from the 90-degree elbow flange connection at the bottom of the west shaft, east towards the end of the tunnel. Clock positions described are referenced by facing west.

Figure 19. Coating anomaly observed at the 3 o'clock position on pipe 0300, at 13.8m from the center of the flange pair connecting the 90-degree elbow to the first horizontal pipe.

Figure 20. Coating anomaly observed on pipe 0280, at 21.85m from the 90-degree elbow flange. Ridges can be seen pushed up in the coating, as well as what appears to be an imprint resembling fabric, possibly caused by a transport strap.

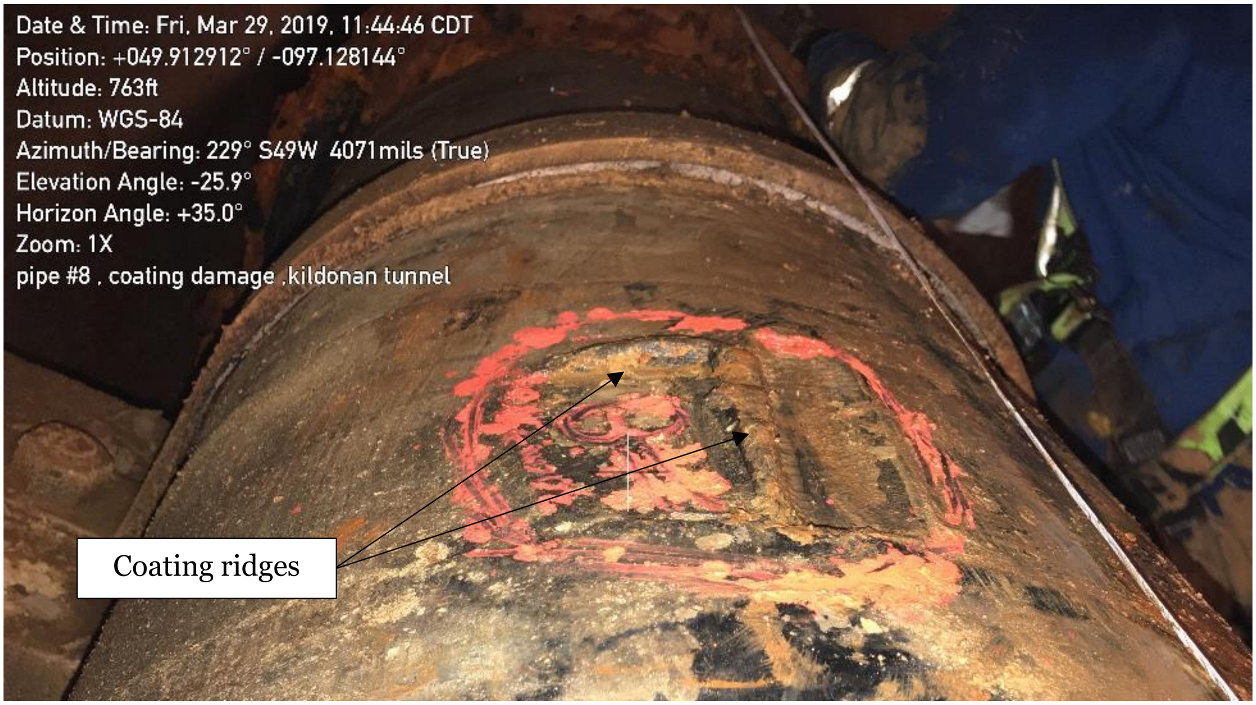


Figure 21. Coating anomaly observed on pipe 0270, at 26.4m from the 90-degree elbow flange.

Figure 22. Divot observed in coating material at the 1:30 clock position on pipe 0310, 10.5m west from the 90-degree elbow.

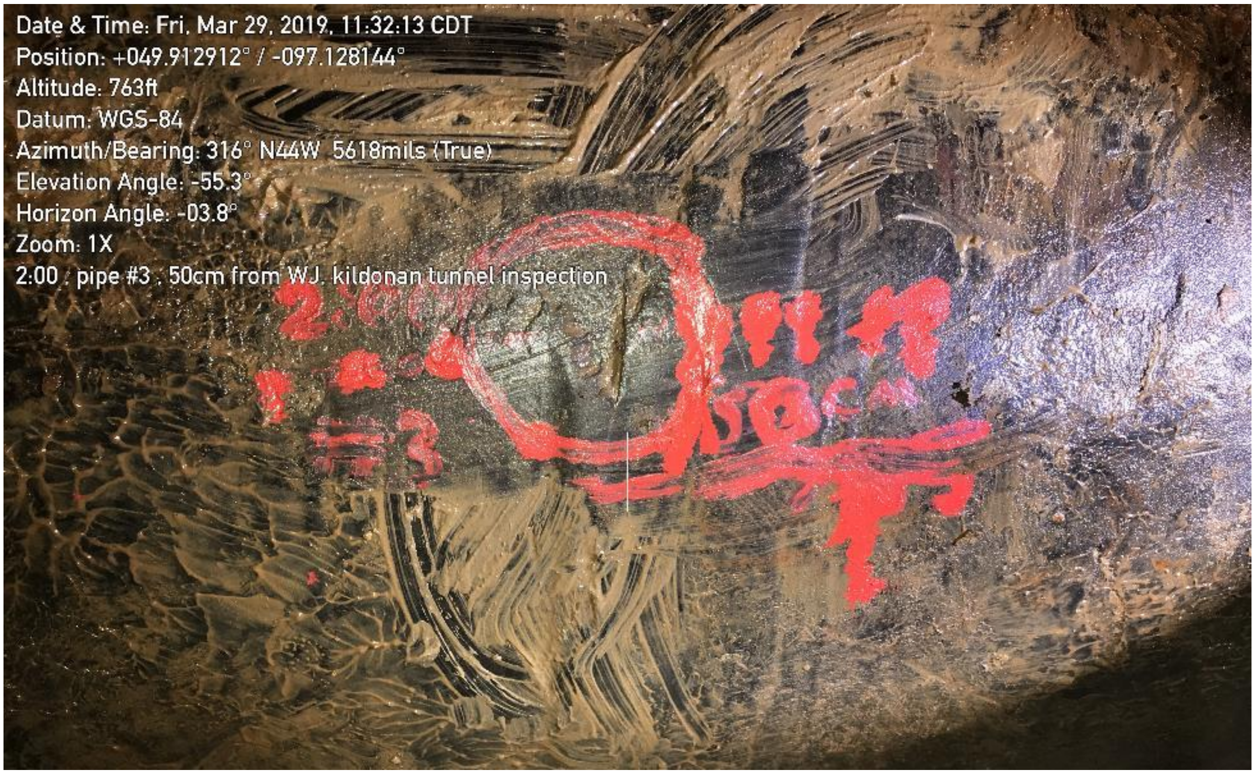


Figure 23. Coating loss at 2 o'clock position on pipe 0220, 47.33m from the 90-degree elbow flange.

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.

Appendix 1: Colour Maps of Signal Indications

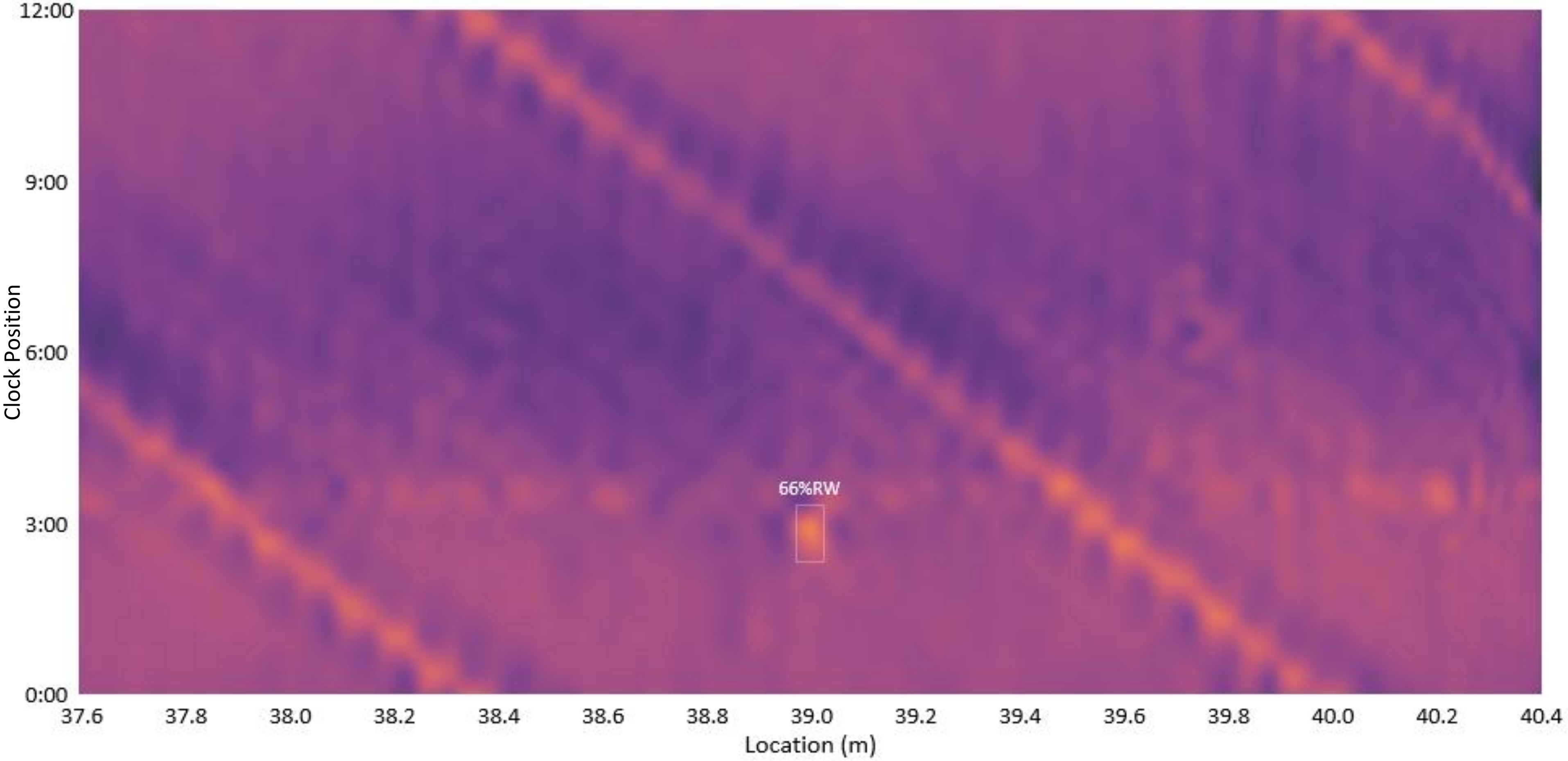


Figure 24. Pipe 0070 colour map showing 66% remaining wall indication, 39.00 meters from the ZRD.

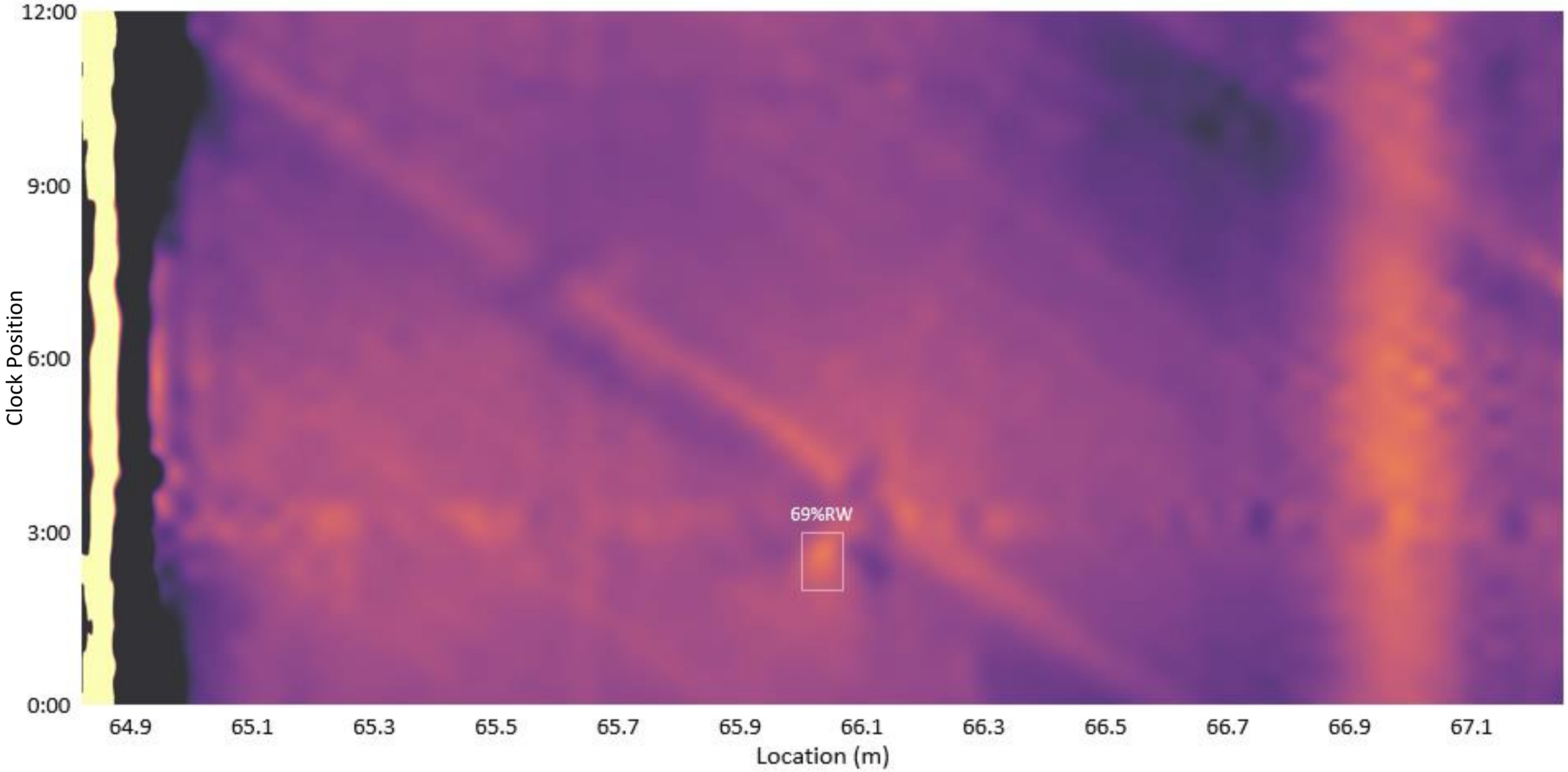


Figure 25. Pipe number 0100 colour map showing 69% RW indication, 66.03 meters from the ZRD.

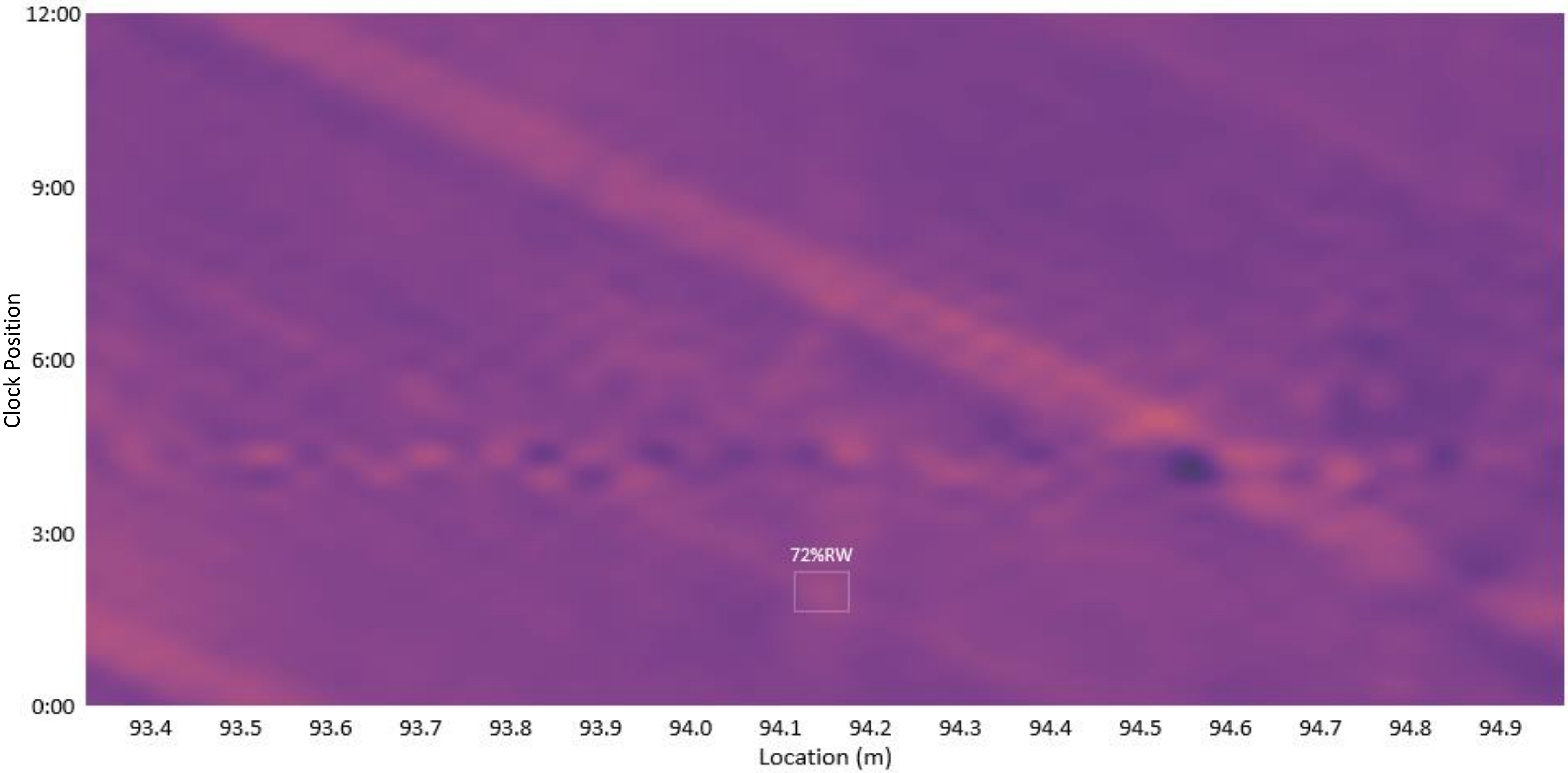


Figure 26. Pipe 0120 colour map of 72% RW indication, 94.15 meters from the ZRD.

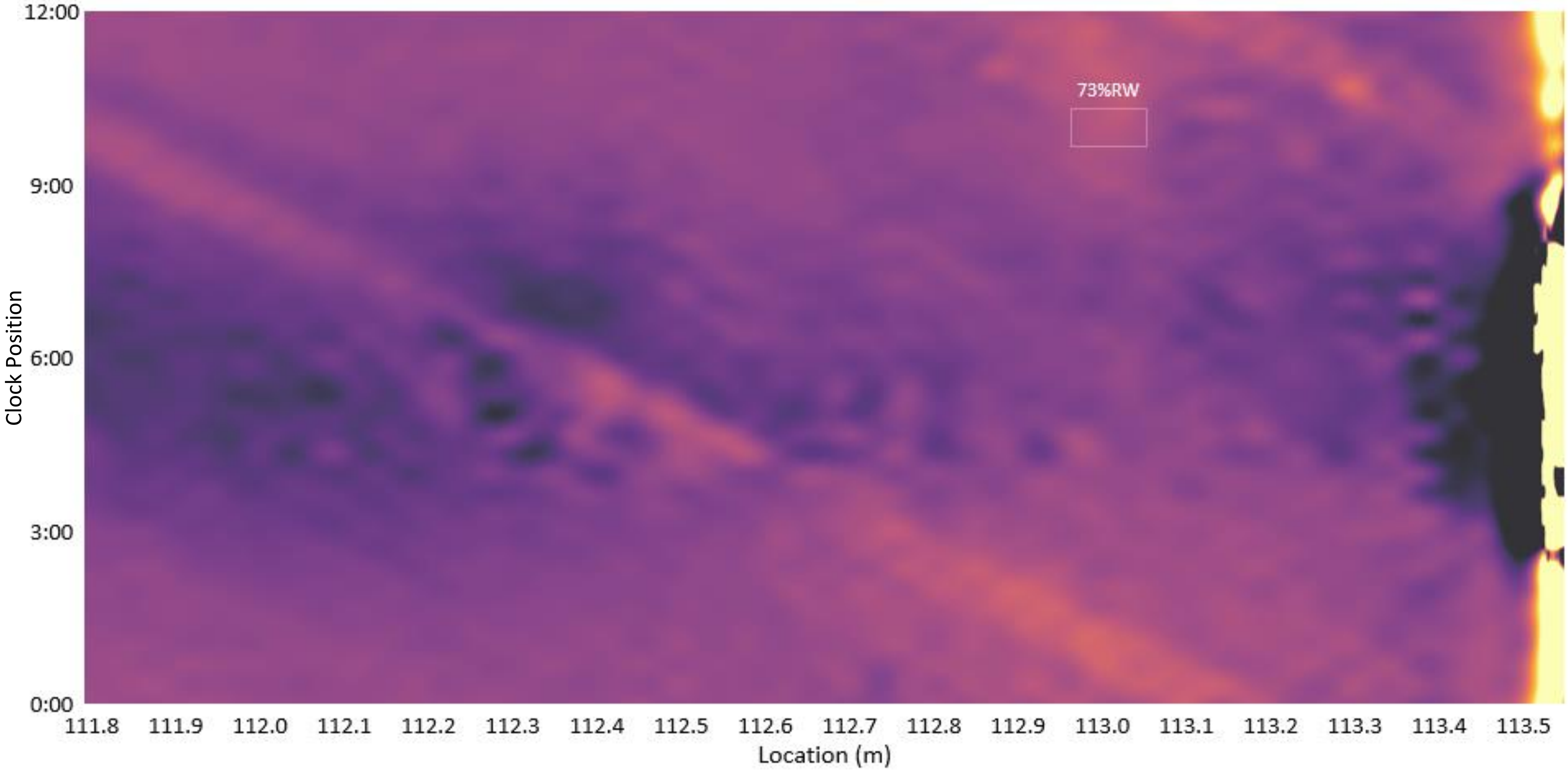


Figure 27. Pipe 0130 colour map showing 73% RW indication at 113.01 meters from the ZRD.

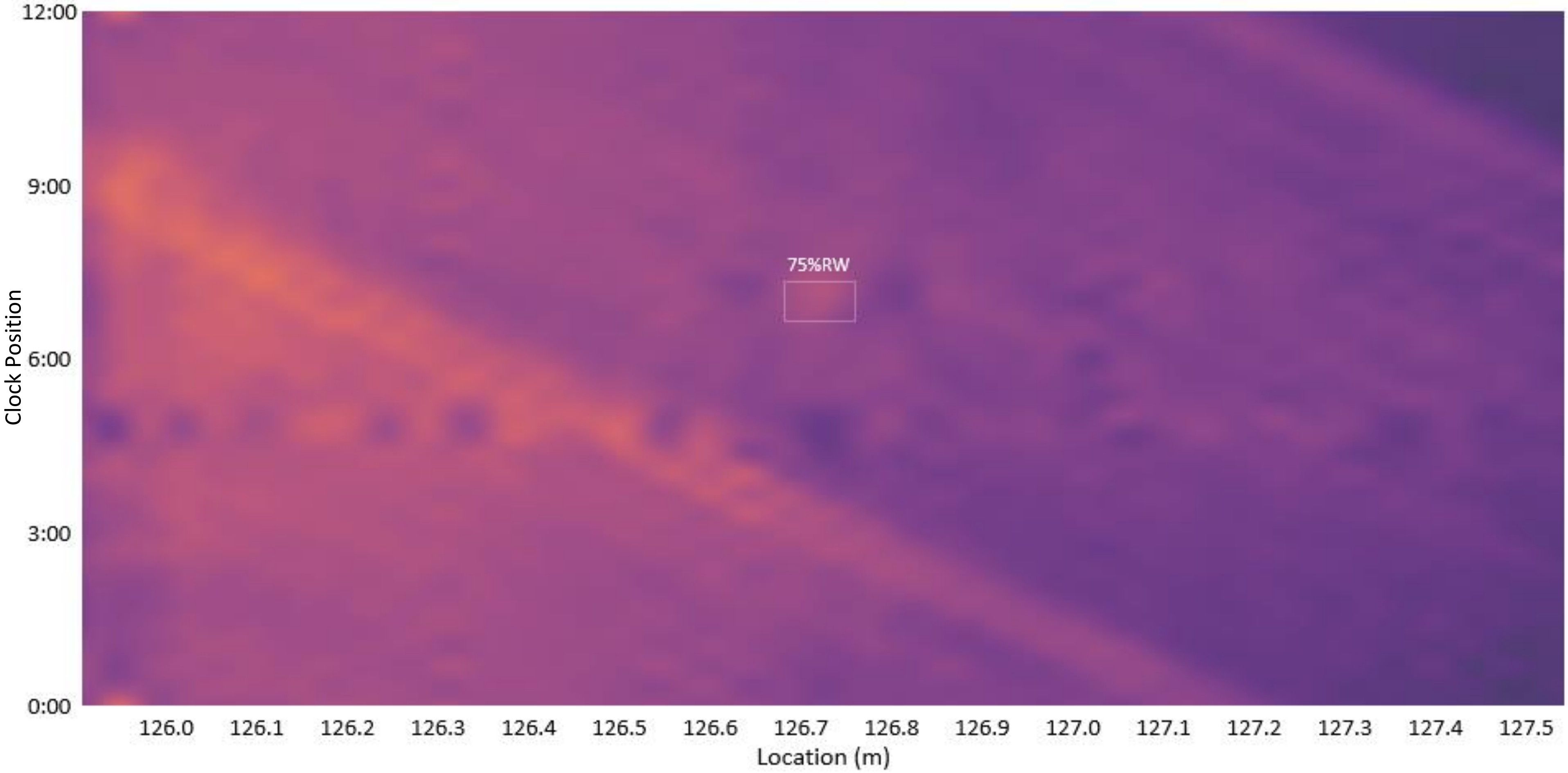


Figure 28. Pipe 0150 colour map showing 75% RW indication, 126.73 meters from the ZRD.

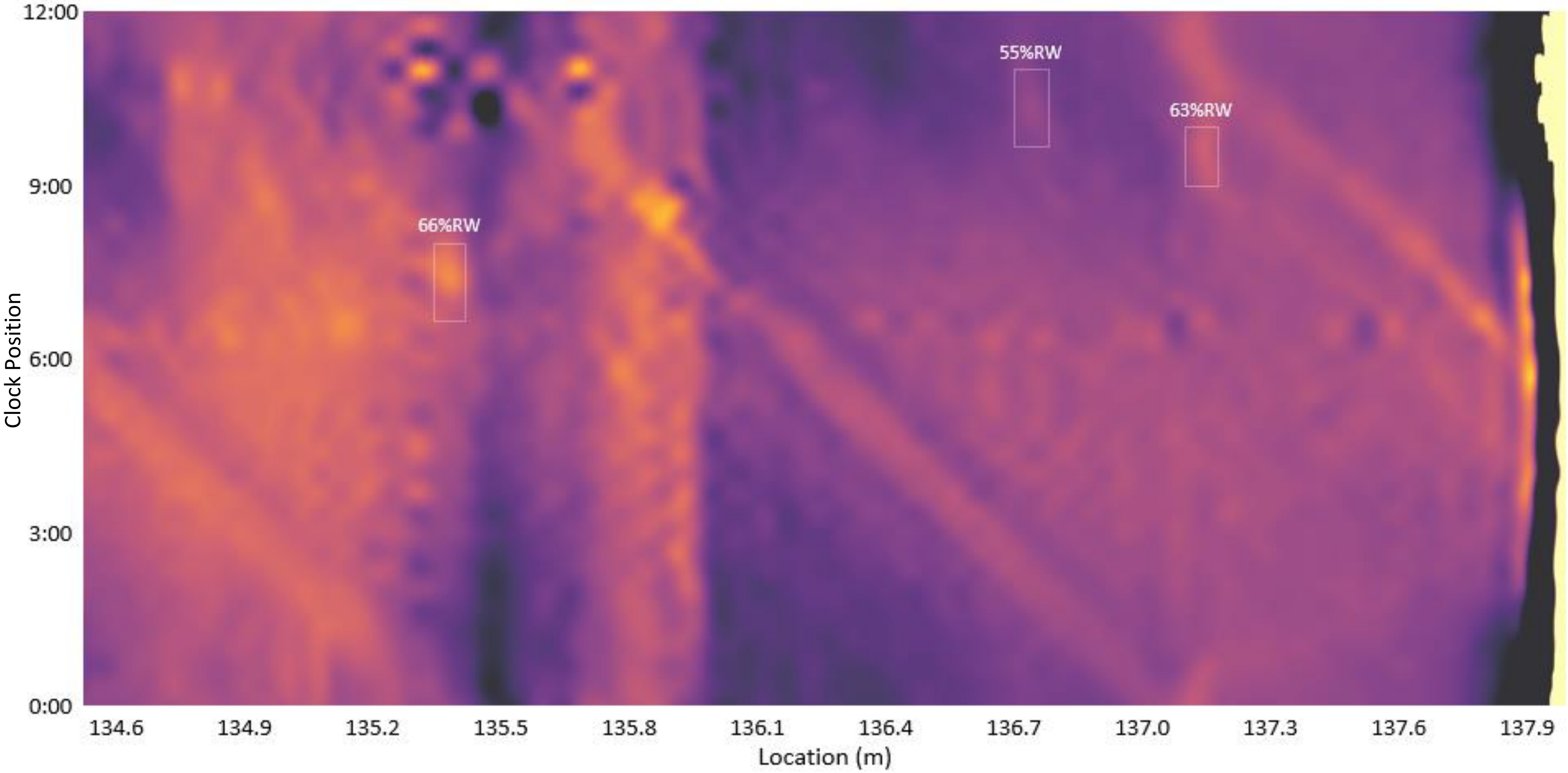


Figure 29. Pipe 0150 colour map showing the 66% RW, 55% RW, and 63% RW indications.

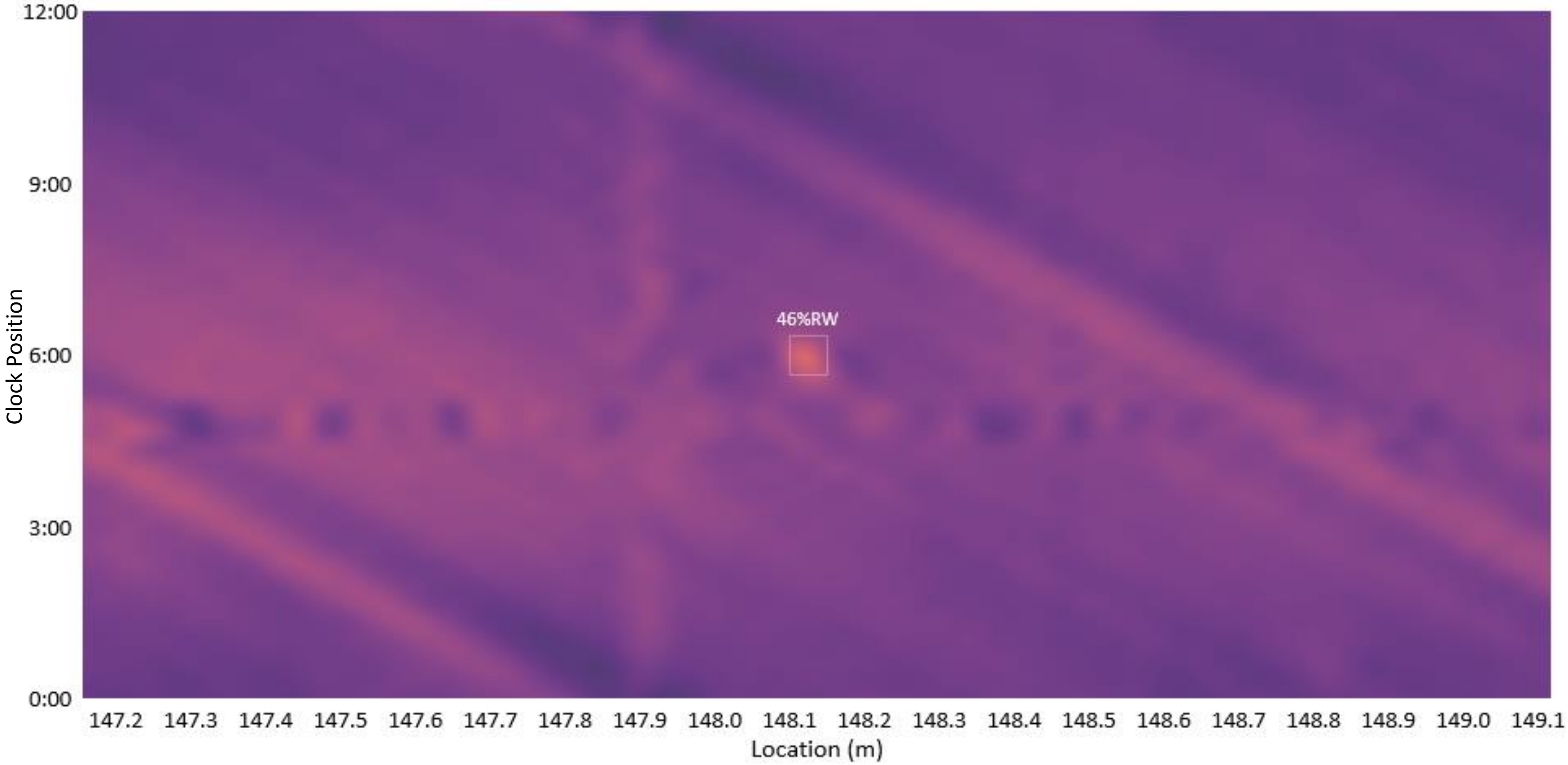


Figure 30. Pipe number 0160 showing the 46% RW indication, 148.13m from the ZRD.

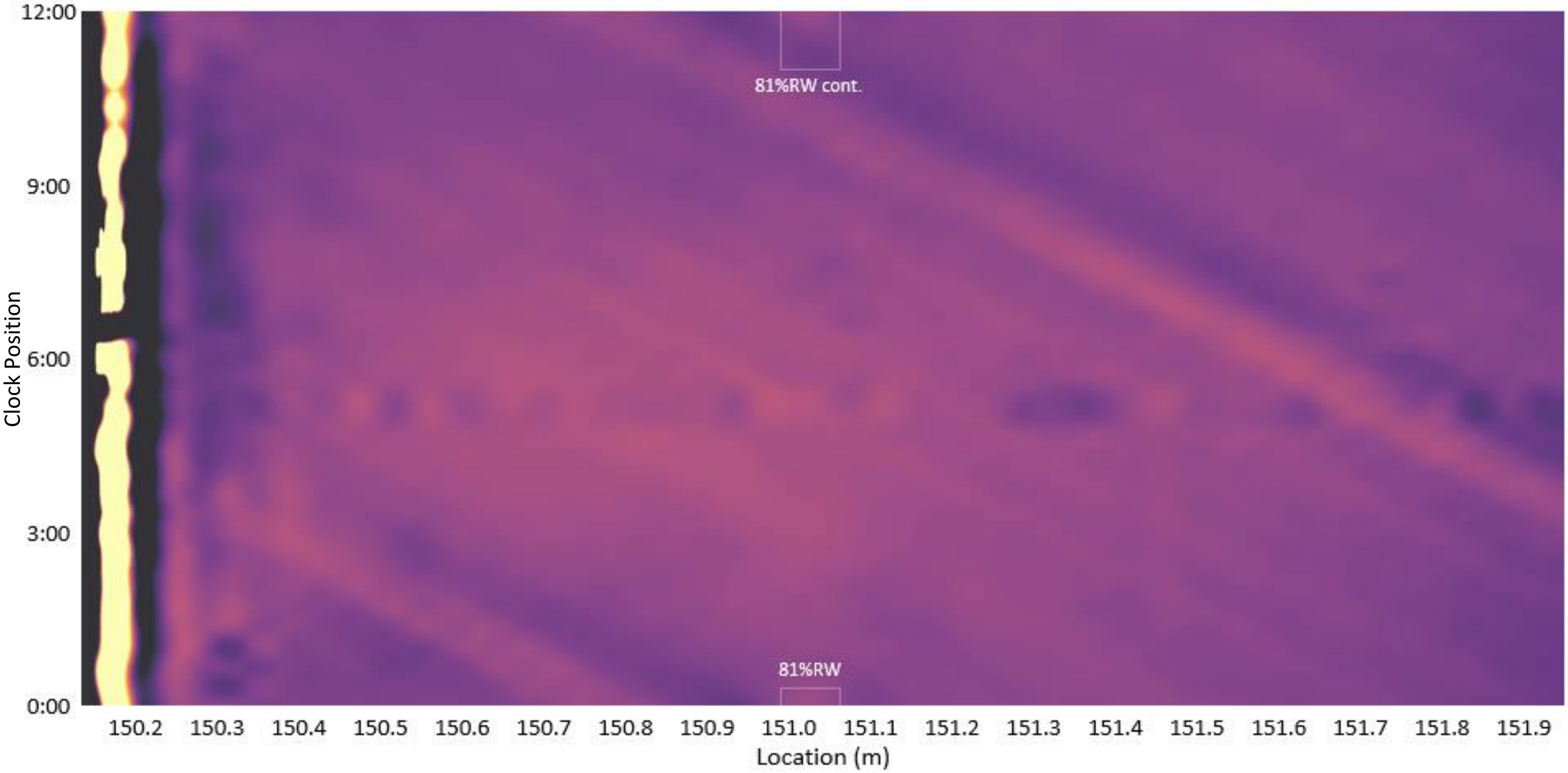


Figure 31. Pipe 0170 colour map showing 81% RW indication at 151.03m from the ZRD.

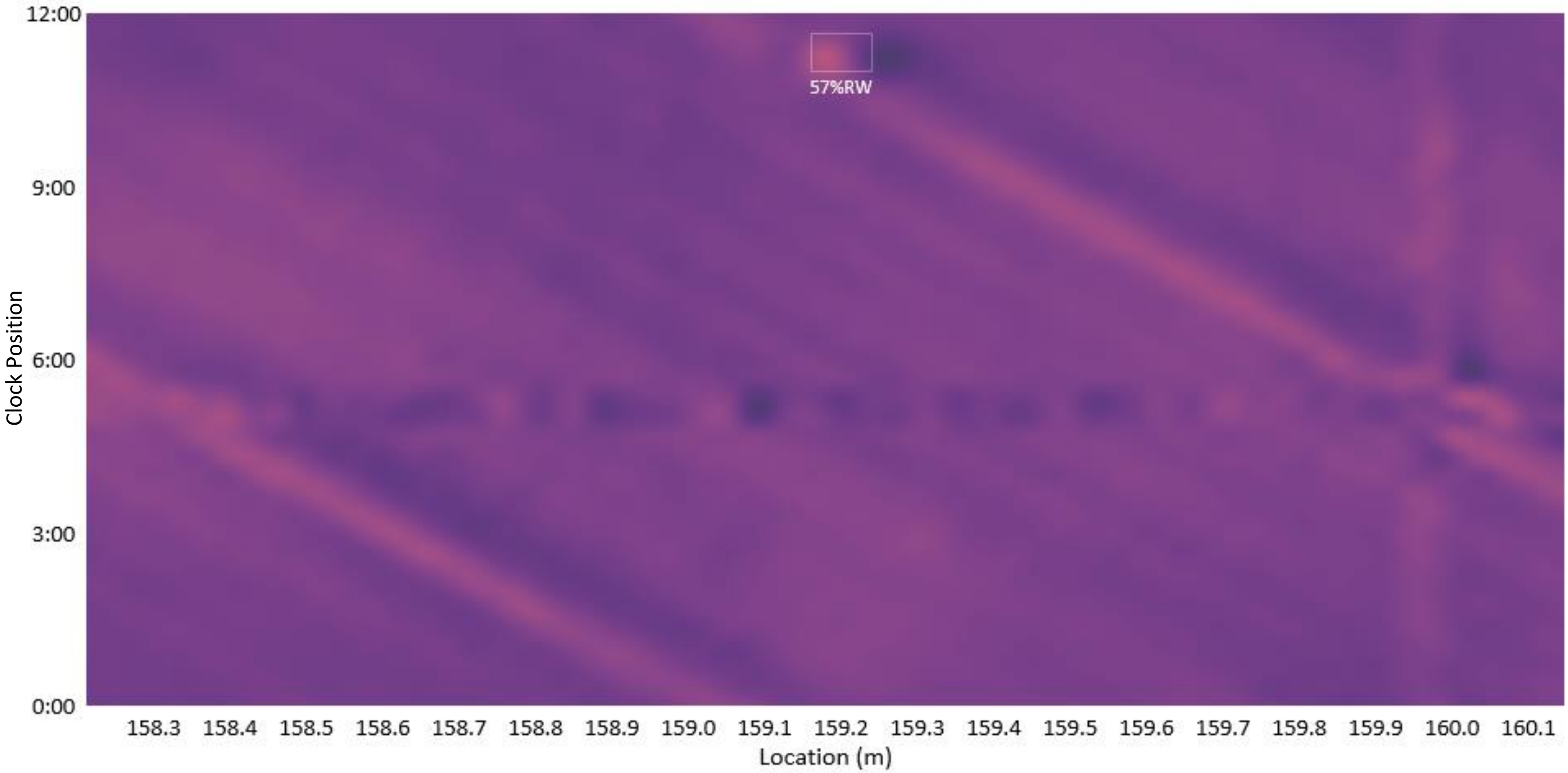


Figure 32. Pipe 0170 colour map showing the 57% RW indication, 159.19m from the ZRD.

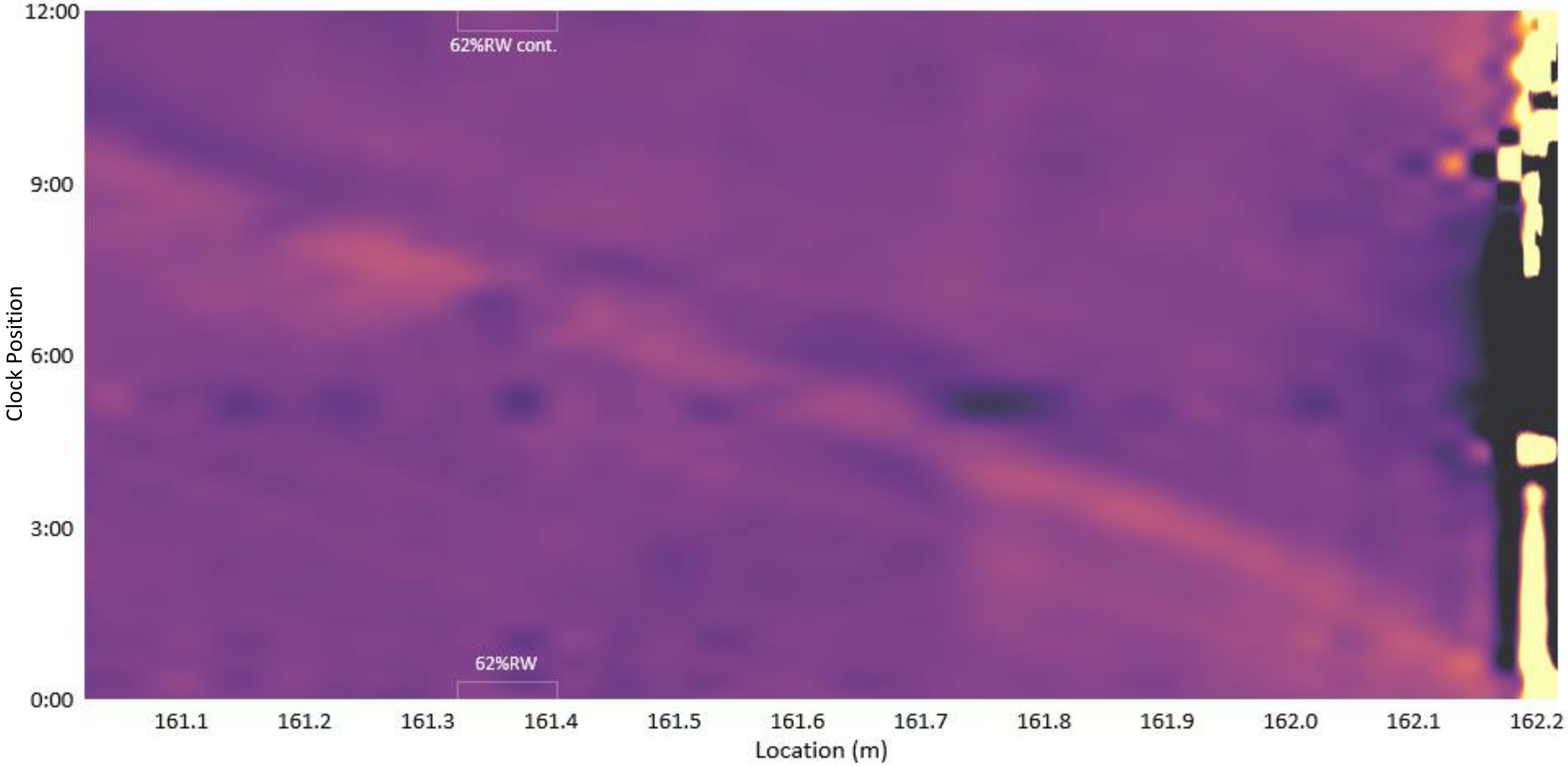


Figure 33. Pipe 0170 colour map showing the 62% RW indication, 161.37m from the ZRD.

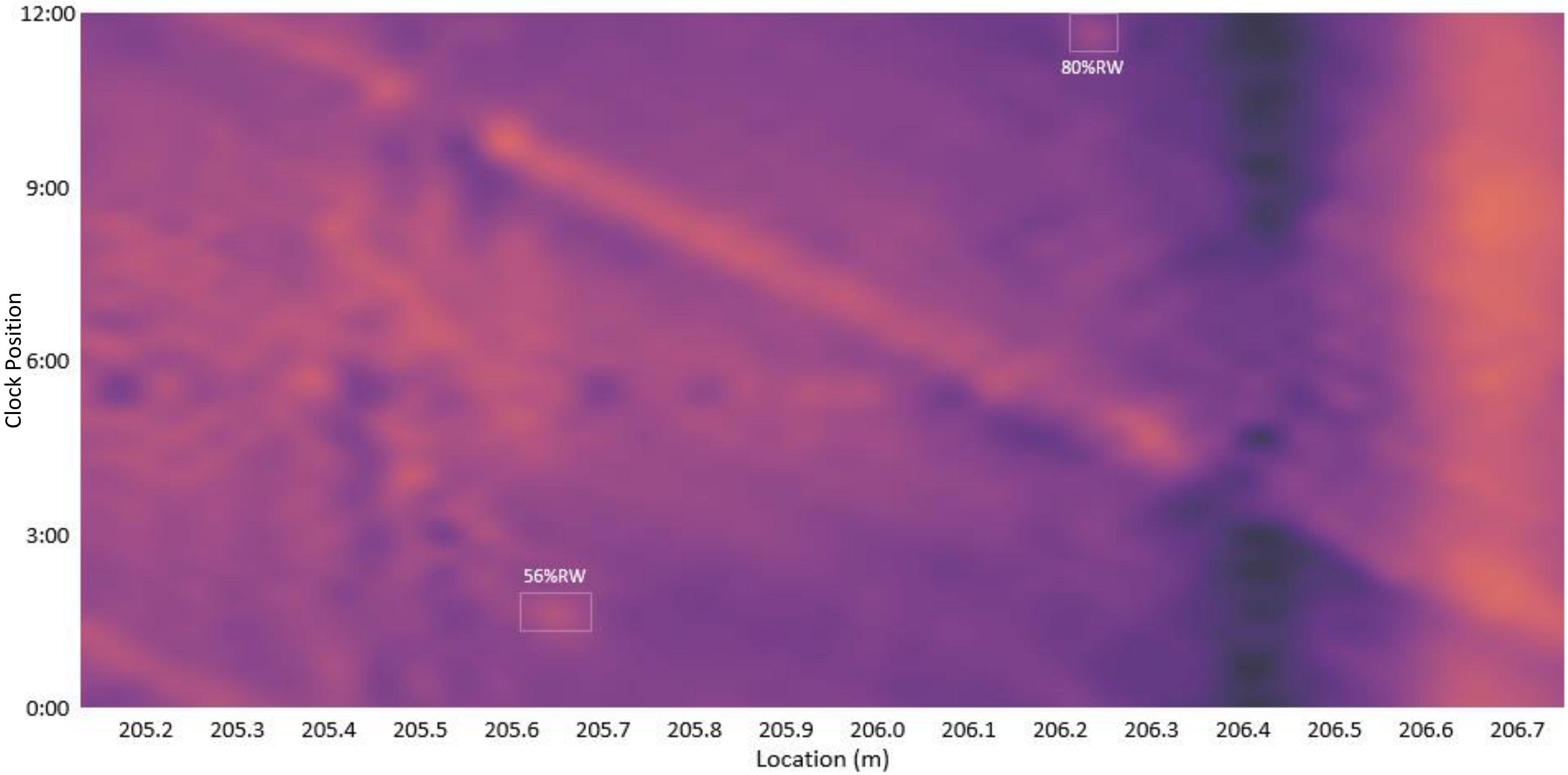


Figure 34. Pipe 0290 colour map with 56% RW and 80% RW indications shown.

Appendix 2: RFT Tool Calibration

Prior to arriving on site, PICA performed a test run of the 24in Chimera tool in a 24in diameter, 9.5mm (0.375in) NWT, unlined spiral welded steel pipe to ensure that the tool was in proper working condition. The calibration pipe contained 5.1cm diameter circular flat bottom defects of varying wall loss percentages, and circular through holes (TH) of varying diameters. Machined defects were measured with an Ultrasonic Testing (UT) device to confirm final wall thicknesses. Table 5 provides an overview of the defects present in the calibration pipe.

Table 5. Defects machined into the 600mm diameter, 9.5mm (0.375in) NWT spiral welded steel calibration pipe

Defect Type	Remaining Wall:	Volume of Defect:
Circular Flat Bottom Defects 5.1 cm (2.0 in) diameter:	73%	5.2 cm^3 (0.3 in^3)
	53%	9.1 cm^3 (0.6 in^3)
	20%	15.4 cm^3 (0.9 in^3)
	Diameter of Defect:	**Volume of Defect:**
Circular Through Holes (0% RW):	1.3 cm (0.5in)	1.3 cm^3 (0.1 in^3)
	2.5 cm (1.0in)	4.7 cm^3 (0.3 in^3)
	5.1 cm (2.0in)	19.5 cm^3 (1.2 in^3)
	7.6 cm (3.0in)	43.2 cm^3 (2.7 in^3)
	10.2 cm (4.0in)	77.8 cm^3 (4.7 in^3)

Figure 35 show the circular flat bottom defects, and Figure 37 shows the circular through hole defects machined into the calibration pipe. All defects were visible in the RFT scan of the calibration pipe, including the 1.3cm (0.5in) diameter through hole and the 5.1cm x 73%RW flat bottom defect. Figure 36 shows the RFT scan of the calibration pipe. It is important to note that the results of the calibration may not be directly comparable to the 6.4 mm (0.250in) CML pipe used to construct the Kildonan-Redwood Feeder Main due to the differences in nominal pipe wall thickness and steel grade.

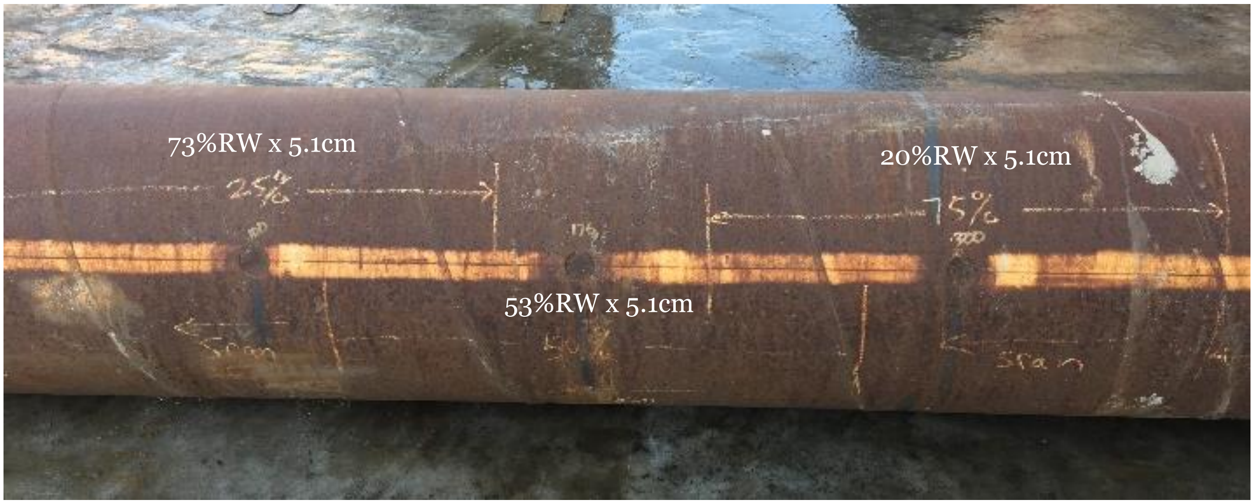


Figure 35. Calibration pipe with 73% RW, 53% RW, and 20% RW 5.1cm diameter flat bottom defects.

Figure 37. Through holes machined into the 24in calibration pipe. Defects measured 1.3 cm, 2.5 cm, 5.1 cm, 7.6 cm, and 10.2 cm (0.5in, 1.0in, 2.0in, 3.0in, and 4.0in) in diameter.

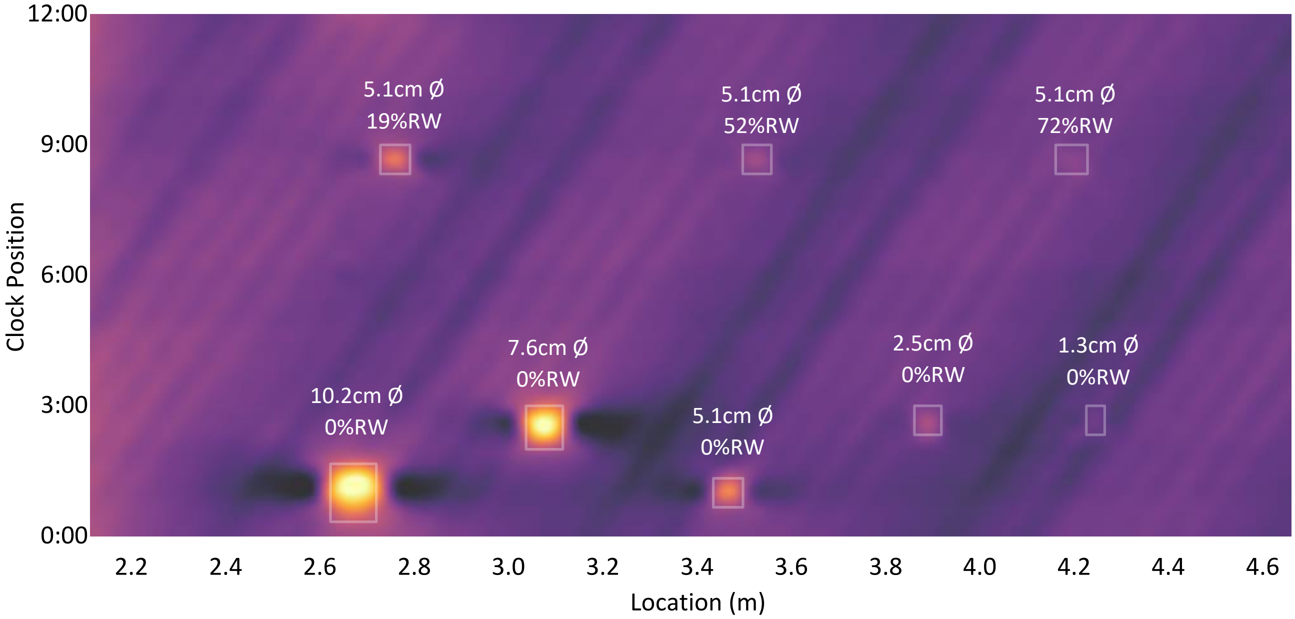


Figure 36. Colour map produced of scanned calibration pipe showing flat-bottom and through hole defects.

Appendix 3: Job Notes and Tool Log

Time (CST)	Operational Comments
Mar 26, 2019	
7:35	Inspection crew arrive on site at Kildonan-Redwood Feeder Main, West Chamber
7:40	First (mechanical Winch) skid steer off trailer
7:48	Laptop synced
8:20	Tool assembled
9:00	Odometer calibrated
9:14	Chimera tool powered up
9:32	Zeroed the wireline odometer with odometer wheel flush with pipe opening.
9:33	Tool scans at 7 Hz with detector leading as the tool begins its descent through the vertical piping
9:45:15	Tool reaches bottom of vertical piping, begin to pull tool back up
10:40	Doing rerun at 5hz with Exciter leading
11:12	Stopped recording
11:14	Restart tool at 5Hz for detector leading run
12:06	East side crew leaves with tool in preparation for inspection of horizontal section.
13:30	Wireline odometer zeroed when Exciter end plate is flush with white painted pipe opening at 13:30. Begin inspection with detector leading at 14 Hz. Start logging at 13:34.
15:13	AGM passage time (west side, AGM Q08874)
15:38	Tool arrived at 90 degrees elbow – start retrieve.
17:05	East side all packed up. Finished pulling tagline back into main from East to West.
17:20	Leave Site
Mar 27, 2019	
7:43	Arrival at Kildonan West side. Tail gate safety meeting.
8:02	Tool is brought to East Chamber
9:00	Tagline is pulled out on the East side, bringing up west winchline.
9:19	Chimera tool powered up. Begin inspection of horizontal portion of Feeder Main at 14Hz.
10:10	Tool passing AGM P40171 on East side of river.
11:21	Tool passing AGM Q08874 on West side of river.
11:49	Tool arrived at 90-degree elbow.
11:59	Tool passing AGM Q08874 on West side of river.
13:02	Data download complete.
13:14	Startup Chimera tool for last 10 Hz run.
13:31	Tool launch east to west at 10Hz.
14:12	Tool passes AGMs on West side of river.
14:41	Tool arrived at 90-degree elbow, start retrieve.
15:16	Tool arrives at East chamber.
16:45	Leave site.
Mar 28, 2019	
7:43	Arrive on site at Kildonan-Redwood, north excavation access.
10:25	Run Start north to south
11:02	Passage of AGM Q08874 on North side of river.
12:08	Tool arrives at south excavation
14:00	Winch slipped at South Chamber. Tool dropped and requiring repairs.
16:00	Zeroing wireline odometers with trailing conical pig 3 inch into the pipe.
16:08	Tool launch south to north.
16:38	Passage of AGM P40171 on South side of river.
17:21	Passage of AGM Q08874 on North side of river.
17:55	Tool arrives

City of Winnipeg - AECOM

600 mm Charleswood-Assiniboia Feeder Main

Condition Assessment Report, Standard Analysis

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

24in Potable Water Chimera RFT ILI Tool
600mm CML Spiral Welded Steel Pipe

Assiniboine River Crossing – RFP 495-2018
Rouge Rd to Berkley St
Winnipeg, Manitoba

PICA Project: 8054

Inspection Date:	March 28, 2019
Report Submission:	August 26, 2019 (rev 1.1); June 1, 2019 (rev 1.0)
Operators:	D. Burton, A. Russell, P. Ryhanen, B. Senka, A. Shatat
Analyst:	A. Liwoch
Reviewers:	M. Chia, A. Shatat
Report Revision:	1.0

CONFIDENTIAL

Table of Contents

City of Winnipeg:

600mm Charleswood-Assiniboia Feeder Main

Condition Assessment Report, Standard Analysis

Executive Summary

PICA, under contract with the City of Winnipeg (RFP 495-2018), inspected two 24in river crossing feeder mains for the City of Winnipeg using Remote Field Testing (RFT) Technology from March 26-28, 2019. The inspected lines are referred to as the Kildonan-Redwood Feeder Main and the Charleswood-Assiniboia Feeder Main. This report documents the inspection results for the 24in Charleswood-Assiniboia line, which crosses beneath the Assiniboine River and was inspected on March 28. The inspected portion spanned between excavations on the north and south sides of Assiniboine River (refer to the line map in Figure 4). This report documents PICA's findings.

A total of 111 localized wall loss indications were identified in the inspection data. Among these defects, two (2) defects measured <20% remaining wall (RW): a 19% RW indication in pipe 0100, and a 16% RW indication in pipe 0130. An additional 36 indications were measured as *deep* (20-39% RW), 62 were classified as *medium* (40-64% RW), and 11 were *shallow* (≥65% RW). Most (89) of the defect indications are located within a single stretch of the Feeder Main spanning from about 61m to 78m, corresponding to pipes 0100 and 0110 (PICA designation).

A listing of all logged anomalies together with detailed analysis information can be found in the companion document, "*PICA Inspection Results - 24in Charleswood-Assiniboia Feeder Main (rev1.1).xlsx*". Figure 1 and Figure 2 illustrate the axial and circumferential distribution of localized defect indications along the Charleswood-Assiniboia Feeder Main. Note that some data points partially overlap due to proximity. A condition assessment summary detailing the top three defect indications, as well as pipe average remaining wall values for each segment (greater than 2.11m in length) of the Feeder Main river crossing is provided in Figure 3.

Table 1. Overview of the RFT findings for the 600mm Charleswood-Assiniboia Feeder Main

Table 1: Feature Indication Summary	
Inspected Length	220.08m
Number of Pipe Segments: *(includes mitered elbows listed as single segments)*	28
Number of Analyzed Pipe Sections:	28
Thinnest circumferential pipe wall (Tcircmin) (RW %):	89% (in Pipe 0160)
Number of pipes without localized wall loss indications:	18
Number of pipes with localized wall loss indications:	10
• **Number of indications with >65% RW:**	11
• **Number of indications with 40-65% RW:**	62
• **Number of indications with <40% RW:**	36
• **Number of indications with <20% RW:**	2
Total number of wall loss indications reported:	111
Number of Fully Circumferential Indications Reported:	1
Number of Flange-Pair Connections:	27
Number of Open-Faced Flanges:	2
Number of Mitered Bends:	7

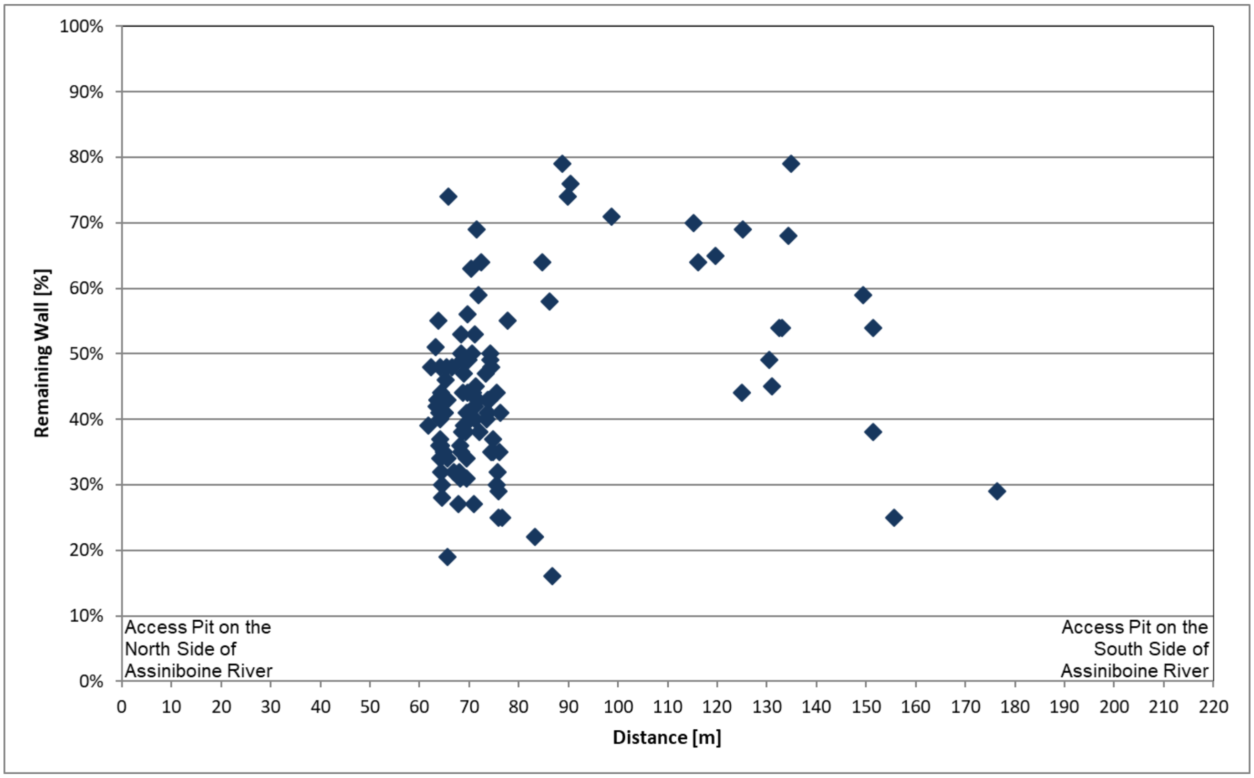


Figure 1. Axial distribution of defect indications and remaining wall (%) within the scanned length of the Charleswood-Assiniboia Feeder Main

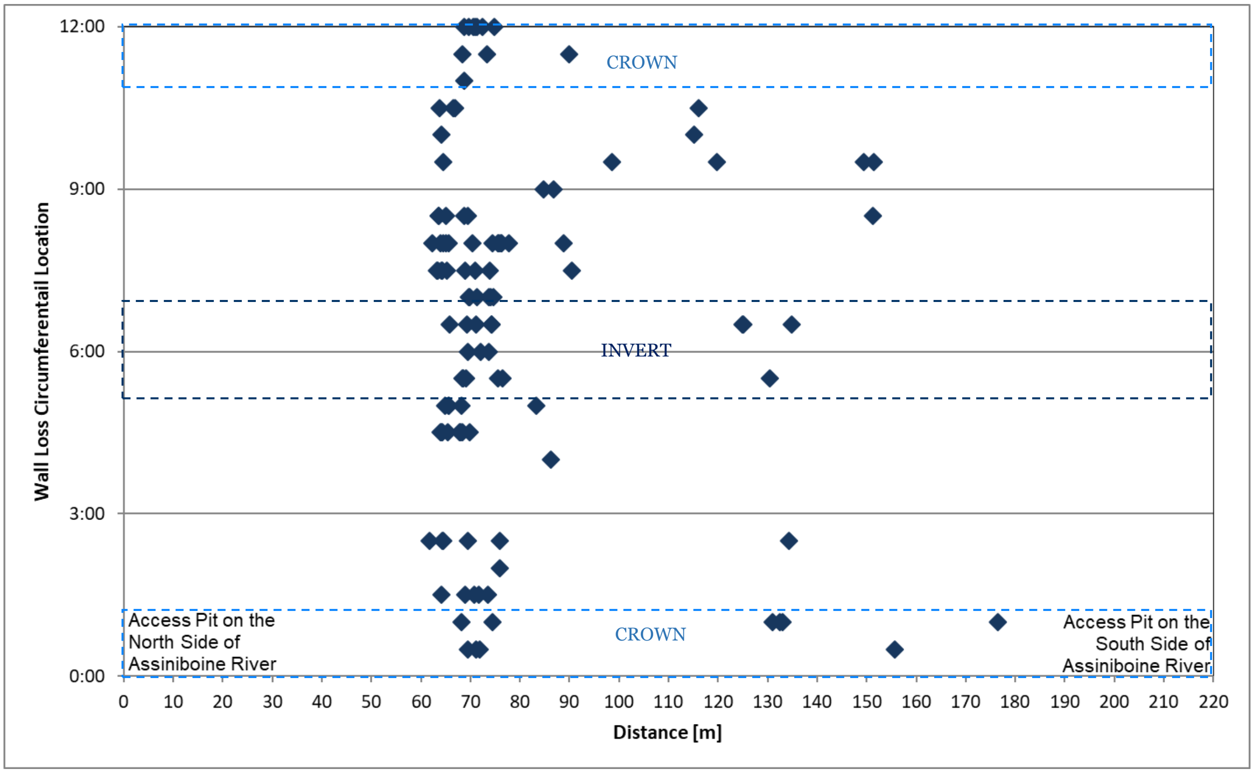


Figure 2. Circumferential distribution of pitting regions along the Charleswood-Assiniboia Feeder Main, described with clock positions referenced by looking from north to south

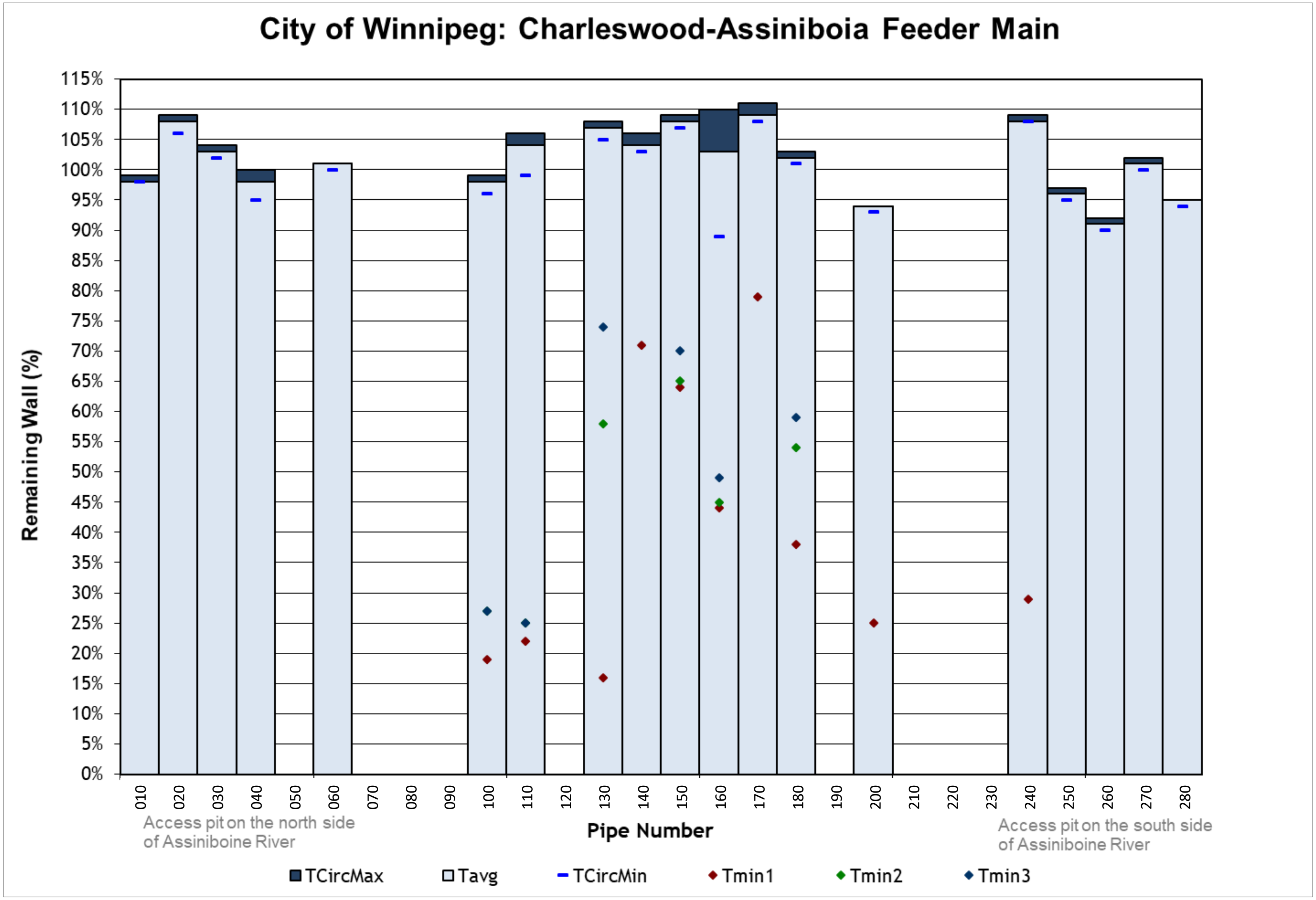


Figure 3. Condition assessment summary for the Charleswood-Assiniboia Feeder Main. Pipes less than 2.4 meters in length were not analyzed for pipe average remaining wall values.

Pipeline Inspection Background

The Charleswood-Assiniboia Feeder Main is a 600mm (24-inch) diameter steel main transporting potable water. A dual tethered Remote Field Technology (RFT) inspection of the feeder mains was conducted by PICA on March 28th, 2019. The section inspected by PICA crosses beneath the Assiniboine River, and extended between two excavations dug north and south of the river (in line with Rough Road and Berkley Street).

The excavations were made adjacent to existing chambers to gain access to the pipeline. Sections of pipe were cut and removed at each of the excavation locations. Figure 4 shows an overview of the inspected length of the Charleswood-Assiniboia Feeder Main crossing beneath the Assiniboine River, and approximate locations of the north and south excavations.

Table 2. Pipeline and RFT inspection information for the 600mm Charleswood-Assiniboia Feeder Main river crossing

Client:	City of Winnipeg	
Location:	Rouge Rd to Berkley St., Assiniboine River Crossing, Winnipeg, Manitoba	
Line Name/ Identifier:	Charleswood-Assiniboia Feeder Main	
Product:	Potable Water	
Pipe Diameter:	600 mm (24-inch)	
Material:	Steel, spiral welded	
NWT:	6.4 mm (0.250 in)	
Grade:		
Internal Liner:	CML (AWWA C205-62T); 0.25in WT (6.4mm) measured in field at excavations	
External Coating:	Coal-Tar Enamel (AWWA C203-57)	
Bends:	(Mitered) 43°, 30°, 6°, 7°	
Joint Type:	Flanges AWWA Class D	
Age:	55 yrs. (1964)	
RFT Inspection Access Locations:	Excavations north and south of the Assiniboine River	
	Elevation	**GPS Coordinates**
North Excavation:	233.0m	49°51'56.38"N, 97°18'2.00"W
South Excavation:	232.5m	49°51'51.74"N, 97°18'8.96"W
RFT Inspection Length:	220.08 m	
Reported Inspection Direction:	North to South	

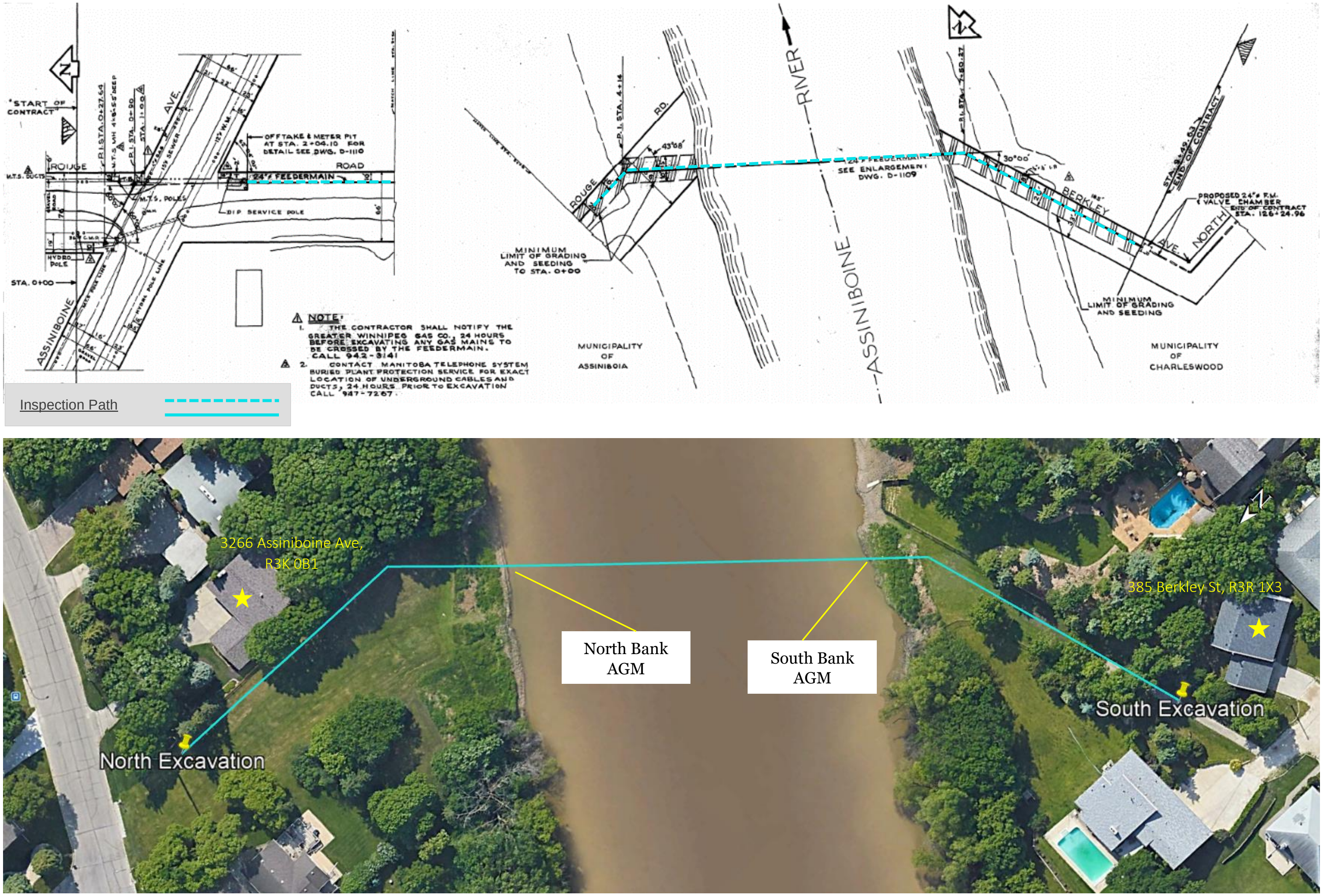


Figure 4. Path map overview of the 600 mm Charleswood-Assiniboia Feeder Main that crosses beneath the Assiniboine River.

Inspection Details

RFT Inspection Preparation

Prior to performing the RFT inspection, all available Critical Application Information (CAI) was reviewed including drawings and measurements of pipes and wall thicknesses provided by the client and subcontractor to ensure a successful inspection.

The pipeline section was isolated, and excavations were made at locations north and south of the river. Pipe sections were cut and removed to gain access to the pipeline. Cleaning pigs were pulled through the river crossing by J-Con to remove any loose scale in preparation for the In-Line Inspection (ILI).

Before PICA's mobilization, the pipeline was gauged by J-Con using PICA's custom gauge assembly to confirm the minimum bore and ensure the 24-inch Chimera RFT ILI tool could safely navigate the Charleswood-Assiniboia Feeder Main. Figure 5 shows the gauge tool after being pulled through the Feeder Main. Minor deflections were observed on the gauge fins, but no major bore restrictions were indicated that would prevent tool passage. A tagline was left in the pipe after the gauging activities were completed, for use during the RFT inspection mobilization.

The RFT Chimera tool assembly was sprayed with a 200mg/L free chlorine disinfection solution (Figure 6) before the tool was introduced into the Main.

Figure 5. PICA's custom gauge tool assembly shown after being pulled through the Charleswood-Assiniboia Feeder Main

Figure 6. Disinfecting the Chimera assembly using a 200 mg/L free chlorine solution

Winches were positioned at the South and North Excavations. The winchline at the South Excavation was connected to the tagline in the pipe. The tagline was pulled up on the north side of the river, pulling the south side winchline through the pipeline (Figure 7). Once the winchline from the South Excavation was pulled through the river crossing to the north access location, it was connected to the Chimera tool assembly.

Figure 7. Tagline being pulled up at north excavation to pull south side winchline through the Feeder Main.

RFT Inspection Procedure

The Chimera tool was lowered into the excavation at the north access location. The winchline odometer as well as the on-board odometer were both zeroed with the Chimera tool's odometer wheel flush with the pipe opening. For the first run, the Chimera tool's detectors were leading (facing south). Figure 8 shows the north access site with the Chimera RFT tool being lowered into the excavation and inserted into the Feeder Main.

Figure 8. North Excavation site, looking northeast

For the inspection of the Charleswood-Assiniboia Feeder Main, the Zero Reference Datum (ZRD) was set to the open flange face of the steel pipe at the North Excavation. The End Reference Datum (ERD) was established at the open flange face of the steel pipe at the South Excavation.

The Chimera tool was pulled from the North Excavation to the South Excavation at a velocity of 2.1 m/min, scanning at a frequency of 14 Hz. Figure 9 shows the excavation and winchline set-up at the south access location. Once the tool reached the excavation on the south side of the river, the tool was removed from the pipe to be turned around for a second inspection run.

During the process of extracting the tool and turning it around, the transmission on the southside winch slipped and the tool was dropped, causing minor tool damage and a short (2hr) delay in the return inspection.

Figure 9. South Excavation winchline set-up.

Following repairs, the Chimera tool was reinserted in the pipe with the detector module again leading (facing north-east). The return run was conducted at the same frequency and velocity as the first inspection run, with the tool traveling from the south excavation to the north excavation for retrieval. After the Chimera was received at the north access location, the collected RFT data was downloaded and reviewed for quality. The total length inspected was 220.08 meters.

PICA's custom Above Ground Monitors (AGMs) were used during the inline inspection as an additional source of positional information. These AGM units are programed to pick up the signal from the RFT ILI tool and were located on the edges of the riverbank so that the underwater portion of the river crossing could be identified during analysis and reporting. Figure 10 and Figure 11 show AGM units positioned on the north and south riverbanks of the Assiniboine River.

Figure 10. AGM unit positioned on the north bank of the Assiniboine river

Figure 11. AGM unit positioned on the south bank of the Assiniboine River

Analysis Results

Two data sets were collected during the inspection of the Charleswood-Assiniboia Feeder Main. One data set was collected as the Chimera tool traveled from the north excavation to the south. After arriving at the south excavation, the tool's orientation was flipped and a second data set was collected as the tool travelled from the south excavation back to the north for retrieval. Both data sets were analyzed, with defect indications correlated between the two inspections. This provides a high level of confidence in the location and sizing of the reported defect indications. All indications are reported with clock positions, referenced by facing south.

Location Reporting, Pipe Lengths & Features

Resource information collected and reviewed during the inspection project was used to supply the most accurate summary of positional information for each of the reported potential wall loss indications. This information included data collected by an Inertial Measurement Unit (IMU) which recorded the pitch, yaw and roll of the Chimera tool as it was pulled through the pipeline; AGM passage locations; and physical measurements of accessible piping. This information was compared with the client supplied drawings of the Feeder Main and was used for corroboration of reported construction feature and wall loss indications.

The 24-inch Chimera RFT ILI tool's on-board odometers, as well as winchline odometers were calibrated and zeroed prior to pulling the tool through the Charleswood-Assiniboia Feeder Main. The total inspected length recorded by the tool was 220.1 meters, with the ZRD set to the open flange face of the north end of the river crossing (identified as P6 in client records; PICA designation: pipe 0010), and the ERD set to the south open flange face at the south excavation (identified as P2 in client records; PICA designation: 0280). Distances stated in this report refer to the distance from the ZRD that was set at the north excavation.

The Charleswood-Assiniboia Feeder Main was constructed with pipe segments of various lengths joined with AWWA Class D Flange pairs. Most of the pipe segments were measured to be approximately 12m in length, corresponding to 40-foot pipe lengths specified in the construction records. Good correlation was observed between pipe lengths documented in client records and lengths measured by on-board odometers during the RFT inspection. A total of 28 pipe segments were identified during analysis of the RFT data. Due to the location of the excavation on the north shore, pipe segments identified in the client records as E7 (two-piece mitered bend) and P9 were not captured in the RFT inspected section.

AGM units were placed on the north and south banks of the Assiniboine river (GPS Coordinates). Recordings of the RFT tool passage correspond to 73.4 meters from the ZRD (AGM on the north riverbank), and 155.0 meters from the ZRD (AGM on the south riverbank).

Seven bends were identified in the RFT data during analysis. These bends correlate with the two horizontal and five vertical mitered bends documented in records provided by the client (Drawing No 7183 /64B.). A total of 27 flange pair connections were observed during analysis correlating to client provided drawings, as well as the open flange face located at the start of pipe 0010 and end of pipe 0280.

The flange connection between pipes 0240 and 0250 was noted to be leaking in 1965 (drawing number 7183/64B). Extra attention was given to this area near the flange connection during analysis to determine if any anomalous signal indications were recorded.

Pipe stress can be detected with RFT technology and correlated with locations of construction features such as anchor blocks. A fully circumferential indication of pipe stress was observed from 129.82m to 134.60m, within pipe 0160, corresponding to the location of an anchor block (listed as anchor block No. 3 in drawing D-1111, at Sta. 6+45). A list of all four anchor block locations is presented in Table 3. The Station Numbers in this Table are derived from client records.

Table 3. List of anchor block locations. Location information sourced from drawing D-1111 and 7183/64B.

Anchor Block No.	Pipe Number	Corresponding Station No.
1	0120, 0130	Sta. 5+05
2	0140	Sta. 5+70
3	0160	Sta. 6+45
4	0180	Sta. 7+15

General Wall Thickness

Pipe sections longer than 2.4m were analyzed to obtain the Pipe Average Remaining Wall (PARW) thickness calculated over the length of the inspected section. This value is reported as the “Tavg” RW in Table 4. The Chimera’s sensor-exciter spacing (SES) is 2.11m, therefore pipe segments shorter than 2.4m were not analyzed for Tavg to ensure that the Chimera was not spanning between two or more separate pipes.

Due to manufacturing tolerances, fluctuations of ±15% in the individual PARW values are common. Variations outside the ±15% spread can be an indicator of a different nominal wall thickness or pipe type or point towards a problem like aggregate pitting or general wall loss. All pipes that were analyzed in the 600mm (24in) Charleswood-Assiniboia Feeder Main fall within the tolerance allowance.

Local Wall Thickness

Ten (10) pipes show evidence of pitting corrosion with a total of 111 localized pitting indications reported. Of these 111 indications, 11 indications measured to be “shallow” (≥65% RW), 62 indications measured to be “medium” (40-64% RW), and 36 indications measured “deep” (20-39% RW). In addition, two (2) defect measured below 20% RW: a 19% RW indication in pipe 0100, and a 16% RW indication in pipe 0130. Most (89) of the defect indications are located within a single stretch of the Feeder Main spanning from about 61m to 78m, corresponding to pipes 0100 and 0110 (PICA designation). Within these two pipe segments (0100 and 0110) significant corrosion patches were recorded along the invert. These corrosion patches range from4:30-8:30 o’clock and span between 68.0m-71.5m in pipe 0100, and between 73.5m-76.3m in pipe 0110. Only the larger localized indications that stand out from the surrounding corrosion were flagged during the analysis.

A colour map of the area with the smallest remaining wall indication recorded (16% RW in pipe 0130), located at 86.75m from the ZRD, is shown in Figure 12. Colour maps can highlight wall loss, wall gain, and material stresses within the pipe. Figure 13 to Figure 24 in Appendix 1 show additional colour maps of the data for all sections of pipe with reported wall loss indications, with defect indications annotated. Defect indications are highlighted in the figures with bounding boxes.

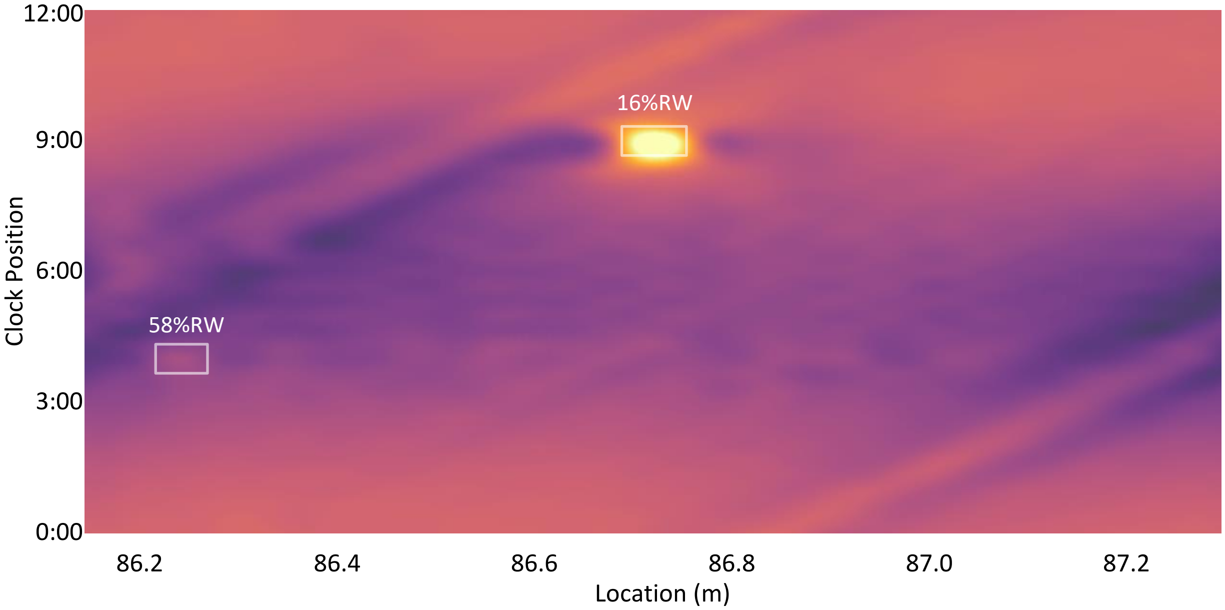


Figure 12. Colour map showing a close up of the 16% RW indication detected in pipe 0130, 86.75m from the ZRD on the north side of the river crossing

Table 4 details the three worst pitting indications per pipe (Tmin1, Tmin2 and Tmin3), as well as the average (Tavg), minimum circumferential (Tcircmin) and maximum circumferential (Tcircmax) remaining wall values for the inspected portion of the Charleswood-Assiniboia Feeder Main.

If AECOM and the City decide to perform verification and repair work on the onshore portions of the feeder main crossing (for example for pipe 0100), please let your PICA representative know. PICA can assist by providing dig sheets for the selected areas.

Data Quality

Prior to the inspection of the Charleswood-Assiniboia Feeder Main, the 24-inch Chimera RFT ILI tool was calibrated using a calibration pipe at PICA's shop. Details about the calibration can be found in Appendix 2.

Flange connections are difficult to assess. Flange pairs produce very large signals in the RFT data due to the amount of material present; these large flange signals can mask small wall loss indications at or adjacent to the connections. To mitigate the effects of the large RFT signal indications from flange pairs, the Chimera tool was pulled through the Main with the detectors leading for the first run and exciter leading for the second run. This maximizes the area of analyzable data collected on either side of these connections.

A significant number of pitting indications were observed near the invert of the pipeline within pipes 0100 and 0110 in both data sets. Several of these indications are reported with medium confidence due to proximity to spiral welds or travel noise in the RFT data. Defect indications detected within this area were compared between the two data sets to provide the highest accuracy possible during reporting. All defect indications reported are detailed in the Excel Workbook "*PICA Inspection Results - 24in Charleswood-Assiniboia Feeder Main (rev1.0)", Defects* spreadsheet. One of the detector pads (~4:30 clock position in the South-to-North run) was in contact with the pipe wall for most of the inspection – as a result the data from this pad is noisier than the data from other pads in that run. Rubbing of the detector pads against the pipe wall was also observed at the mitered bends, resulting in increased (travel) noise in the data at those locations. This rubbing noise as well as the RFT signal response to the bend (created by the changing alignment between exciter and detector at the bend locations) decreases the data quality at the bend locations.

Table 4. Summary of pipe tally and wall thickness readings for the Charleswood-Assiniboia Feeder Main

Pipe List and Wall Thickness Readings – 600 mm (24in) Charleswood-Assiniboia Feeder Main																
PICA Pipe #	**Pipe Location**			**Tavg RW**	**Circumferential Wall Thickness**		**Local Wall Thickness**									**Comments**
							Tmin1			**Tmin2**			**Tmin3**			
	Start (m)	**End (m)**	**Length (m)**		**Tcircmax RW**	**Tcircmin RW**	**RW**	**Location (m)**	**Clock Position**	**RW**	**Location (m)**	**Clock Position**	**RW**	**Location (m)**	**Clock Position**	
0010	0.0	11.5	11.5	98%	99%	98%										Located on north side of river
0020	11.5	23.6	12.1	108%	109%	106%										
0030	23.6	35.8	12.2	103%	104%	102%										
0040	35.8	48.0	12.2	98%	100%	95%										
0050	48.0	48.7	0.7													-11° pitch change recorded (2 PC 11° 39' elbow in drawing No.) 7183/64B)
0060	48.7	58.4	9.7	101%	101%	100%										
0070	58.4	59.6	1.2													50° horizontal bend RT (3 PC 43°08' horizontal elbow in drawing No. 7183/64B)
0080	59.6	60.6	1.0													
0090	60.6	61.3	0.7													8° pitch change recorded (2 PC 7°13' elbow in drawing No. 7183/64B)
0100	61.3	73.0	11.7	98%	99%	96%	19%	65.61	8:00	27%	67.81	4:30	27%	70.92	12:00	Numerous indications (see defect list for details)
0110	73.0	85.2	12.2	104%	106%	99%	22%	83.21	5:00	25%	75.86	2:30	25%	76.56	5:30	North side AGM is located on north river bank,

Pipe List and Wall Thickness Readings – 600 mm (24in) Charleswood-Assiniboia Feeder Main

PICA Pipe #	Pipe Location			Tavg RW	Circumferential Wall Thickness		Local Wall Thickness									Comments
	Start (m)	End (m)	Length (m)		Tcircmax RW	Tcircmin RW	Tmin1			Tmin2			Tmin3			
							RW	Location (m)	Clock Position	RW	Location (m)	Clock Position	RW	Location (m)	Clock Position	
																73.37m in data (see photo on separate sheet). See defect list for defect details
0120	85.2	85.9	0.7													6.6° Pitch change recorded (2 PC elbow in drawing No. 7183/64B)
0130	85.9	98.0	12.2	107%	108%	105%	16%	86.75	9:00	58%	86.27	4:00	74%	89.85	11:30	Five defect indications reported, one with 16% RW (see defect list)
0140	98.0	110.2	12.2	104%	106%	103%	71%	98.61	9:30							One defect indication reported
0150	110.2	122.5	12.2	108%	109%	107%	64%	116.08	10:30	65%	119.71	9:30	70%	115.14	10:00	Three defect indications reported
0160	122.5	134.6	12.1	103%	110%	89%	44%	124.97	6:30	45%	131.00	1:00	49%	130.39	5:30	Seven defect indications reported, including a circumferential stress indication from 129.82m to 134.60m that correlates with a documented

Pipe List and Wall Thickness Readings – 600 mm (24in) Charleswood-Assiniboia Feeder Main

PICA Pipe #	Pipe Location			Tavg RW	Circumferential Wall Thickness		Local Wall Thickness									Comments
							Tmin1			Tmin2			Tmin3			
	Start (m)	End (m)	Length (m)		Tcircmax RW	Tcircmin RW	RW	Location (m)	Clock Position	RW	Location (m)	Clock Position	RW	Location (m)	Clock Position	
																anchor block location
0170	134.6	146.8	12.2	109%	111%	108%	79%	134.79	6:30							One defect indication reported
0180	146.8	152.2	5.4	102%	103%	101%	38%	151.33	8:30	54%	151.49	9:30	59%	149.41	9:30	Three defect indications reported
0190	152.2	152.9	0.7													5.8° Pitch Change (2 PC 6°17' elbow in drawing No. 7183/64B)
0200	152.9	161.2	8.3	94%	94%	93%	25%	155.61	0:30							South side AGM is located @ 154.97m in data on south river bank (see photo on separate sheet). Contains 1 indication, estimated as 25% RW
0210	161.2	162.3	1.1													2.6° Pitch Change; horizontal bend RT (3 PC 30° horizontal elbow in drawing No. 7183/64B)
0220	162.3	164.3	2.0													

Pipe List and Wall Thickness Readings – 600 mm (24in) Charleswood-Assiniboia Feeder Main

PICA Pipe #	Pipe Location			Tavg RW	Circumferential Wall Thickness		Local Wall Thickness									Comments
	Start (m)	End (m)	Length (m)		Tcircmax RW	Tcircmin RW	Tmin1			Tmin2			Tmin3			
							RW	Location (m)	Clock Position	RW	Location (m)	Clock Position	RW	Location (m)	Clock Position	
0230	164.3	165.0	0.7													(2 PC 1°28' elbow in drawing No. 7183/64B)
0240	165.0	177.1	12.1	108%	109%	108%	29%	176.47	1:00							Contains 1 indication, estimated as 25% RW
0250	177.1	189.3	12.2	96%	97%	95%										Start Joint flagged as leaking during install (1965) in drawing No. 7183/64B
0260	189.3	201.5	12.2	91%	92%	90%										
0270	201.5	213.6	12.1	101%	102%	100%										
0280	213.6	220.1	6.5	95%	95%	94%										Located on south side of river

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.

Appendix 1: Colour Maps of Signal Indications

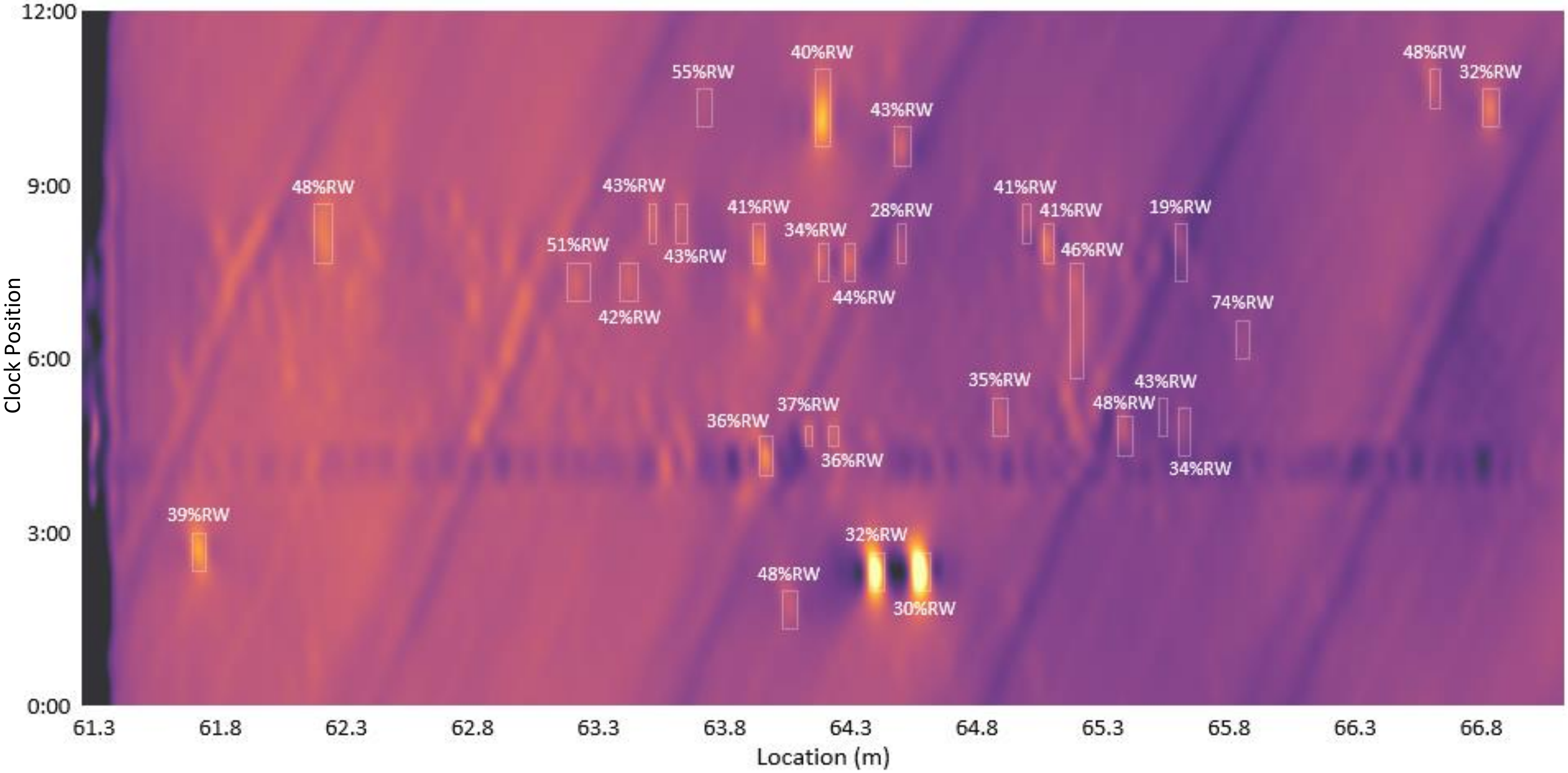


Figure 13. Colour map showing locations of defect indications reported within pipe number 0100 (Client pipe segment P10). A total of 66 indications have been reported in this pipe segment, continued in Figure 14. One deep defect (19% RW) was reported in this pipe segment, located 65.85m from the ZRD. Flange connection at beginning of pipe (61.29m) shown on left of colour map.

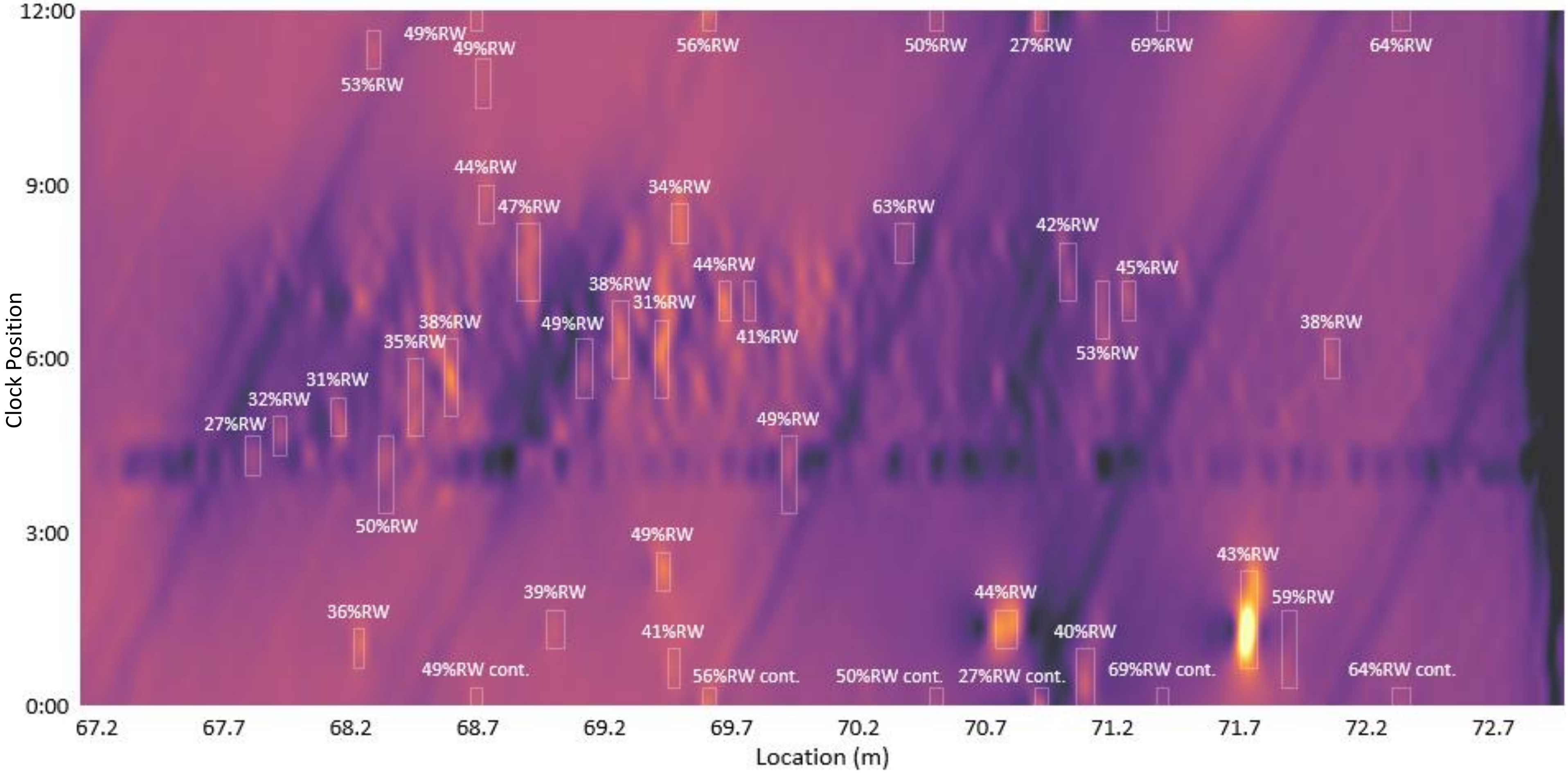


Figure 14. Continuation of pipe number 0100 colour map with defect indication locations shown. Flange connection at end of pipe (72.95m) shown on right of colour map.

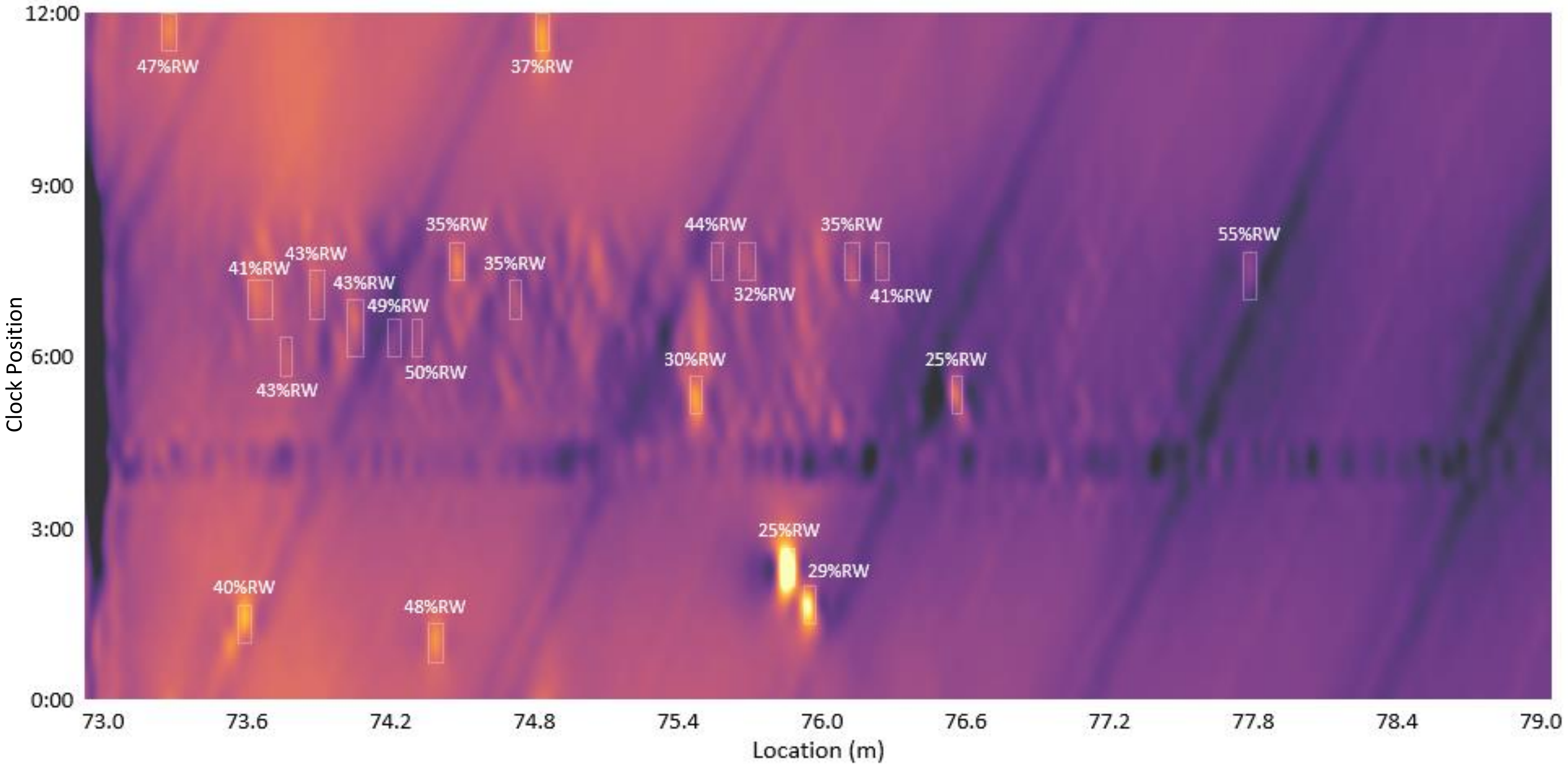


Figure 15. Pipe number 0110 colour map showing defect indication locations. A total of 23 indications have been reported in this pipe segment. Continued in Figure 16. Flange connection at start of pipe shown on left of colour map (72.95m).

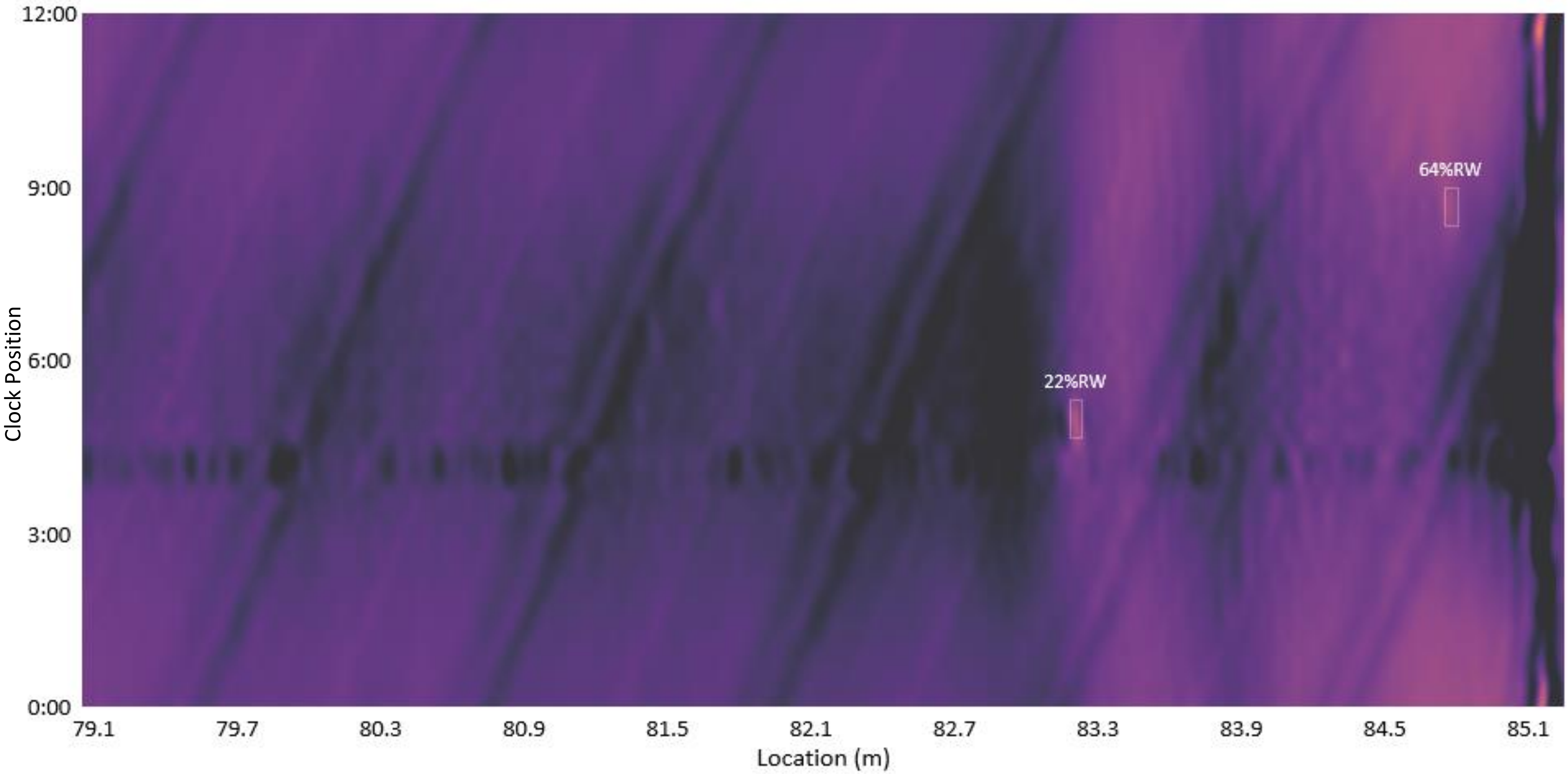


Figure 16. Continuation of pipe number 0110. Flange connection at end of pipe (85.15m) shown on right of colour map.

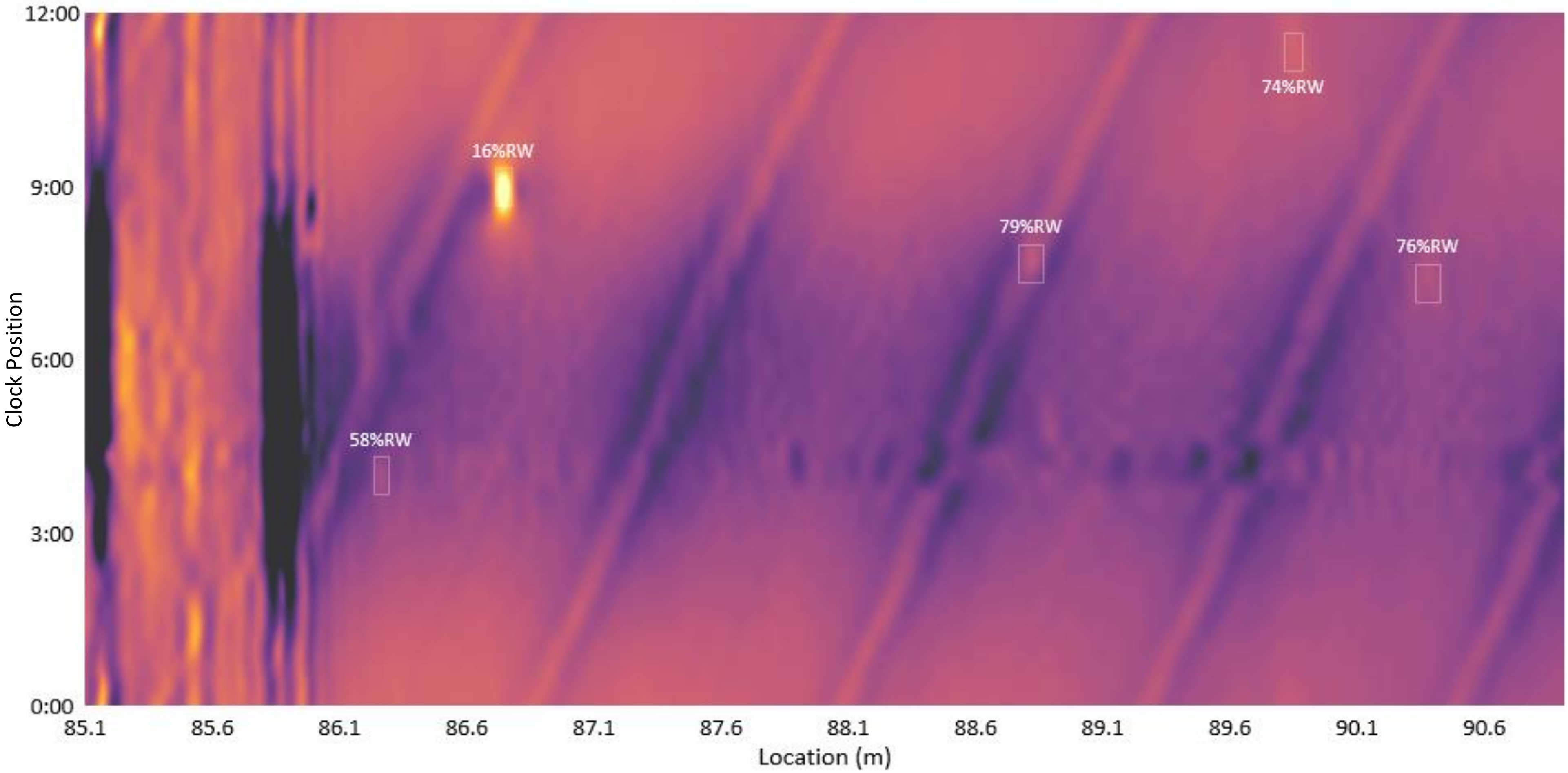


Figure 17. Pipe numbers 0120 (two piece mitered elbow extending from 85.15m to 85.86m) and 0130. A 6.6° pitch change was recorded at the elbow. Five defect indications were reported in pipe 0130, including a 16% RW indication at 86.75m from the ZRD.

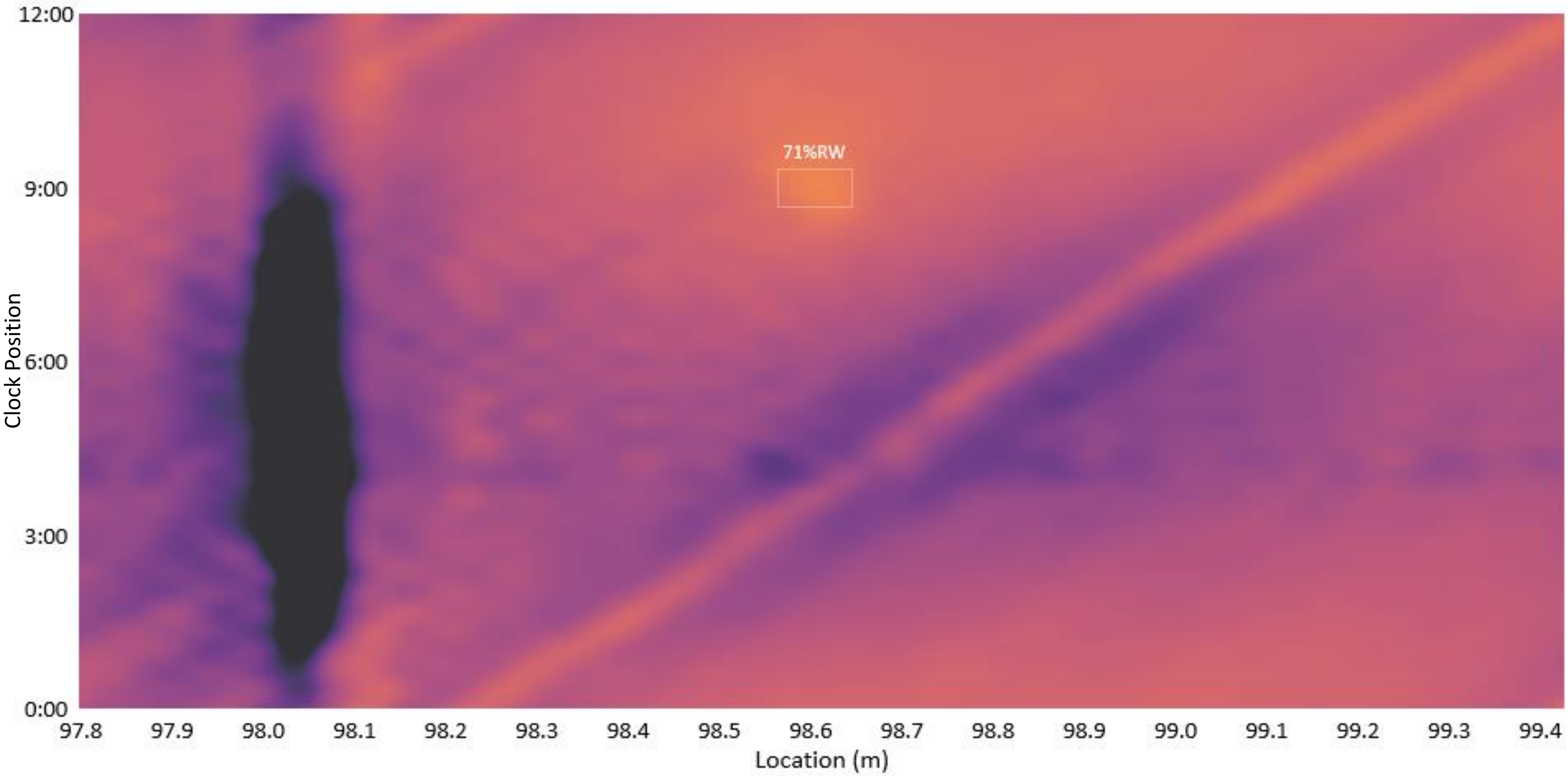


Figure 18. Pipe number 0140 colour map showing the single defect indication reported in this segment. A flange connection is visible beginning at 98.03m.

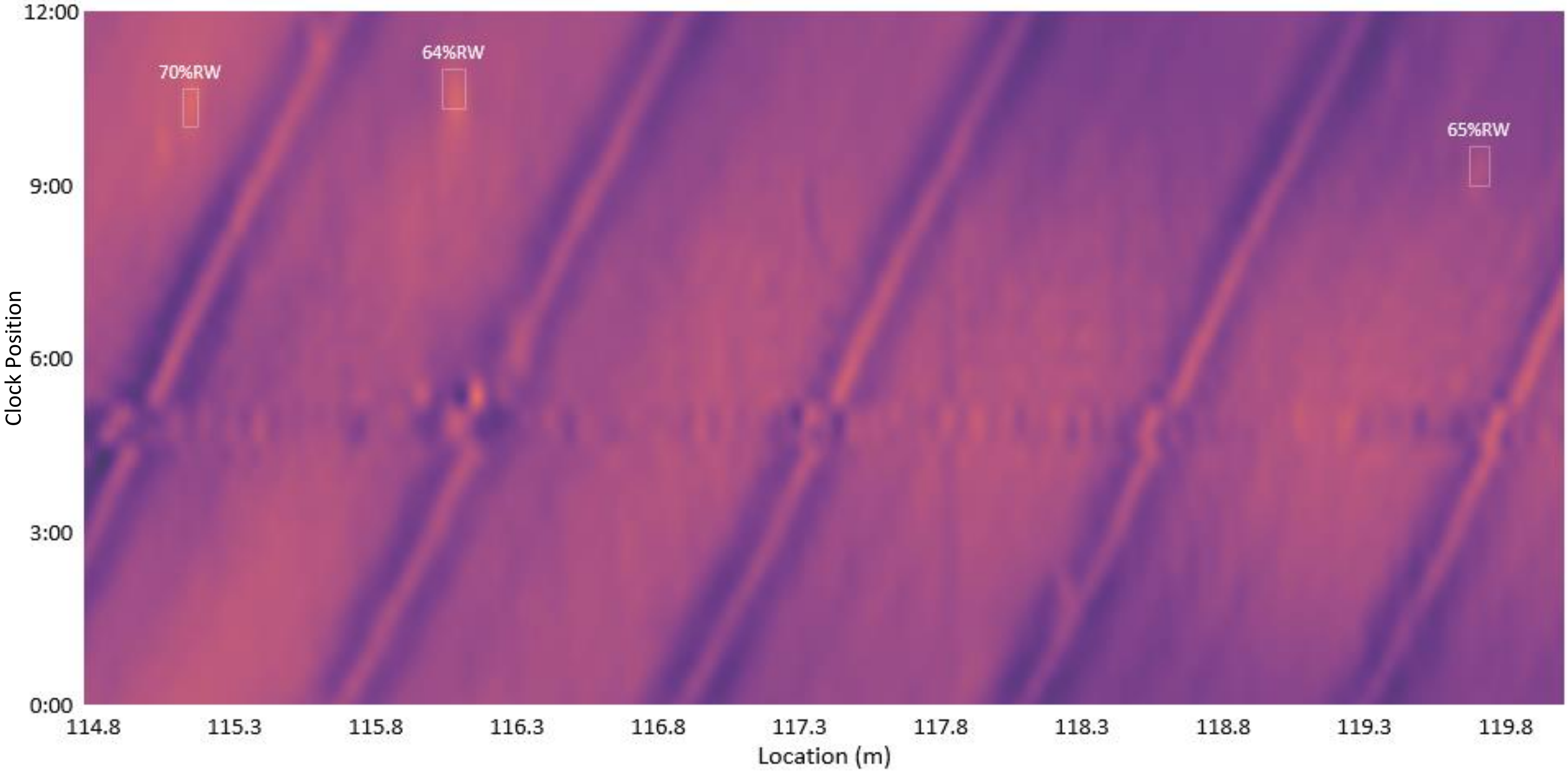


Figure 19. Pipe number 0150 colour map with three reported indications.

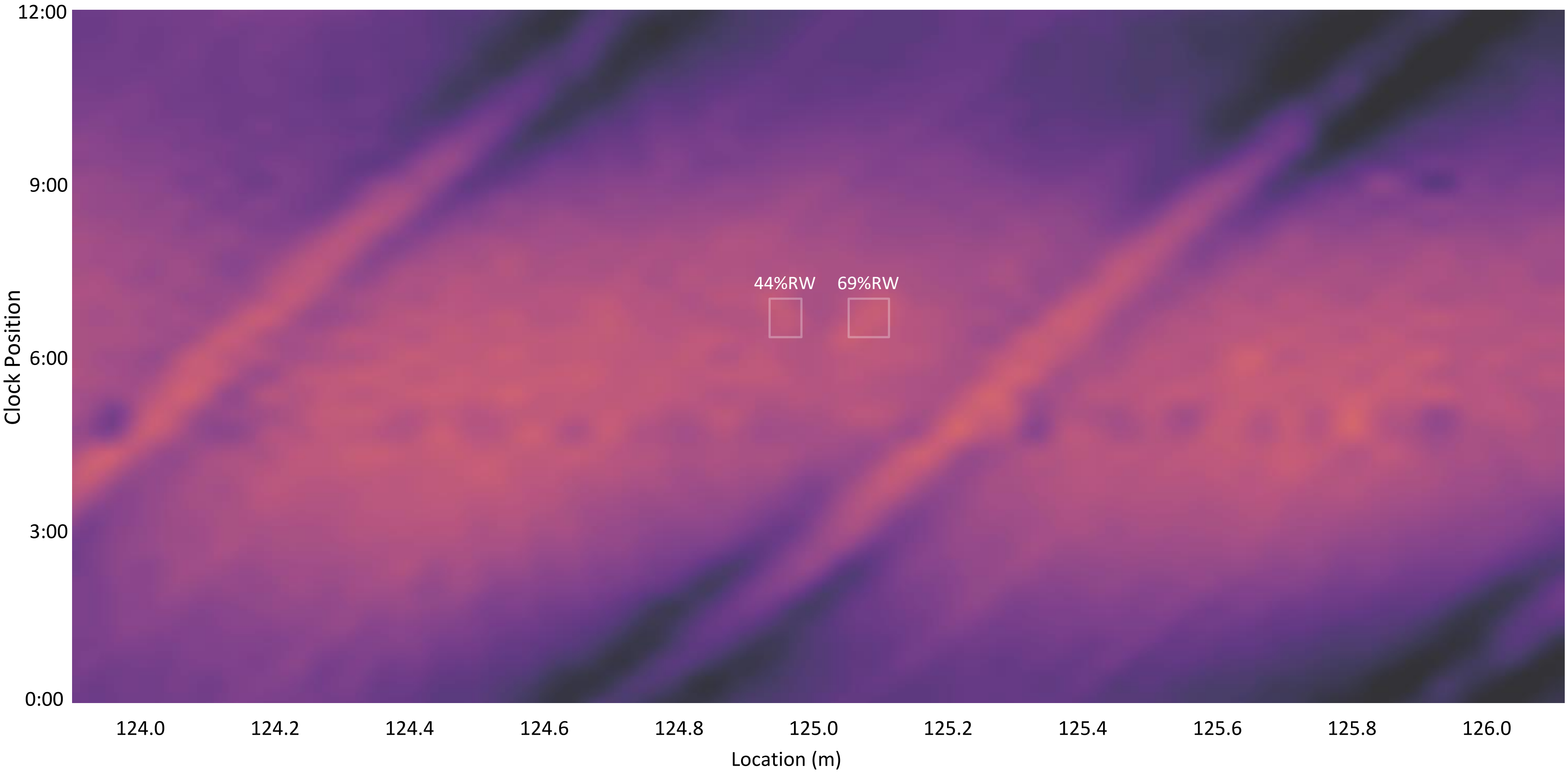


Figure 20. Pipe number 0160 showing locations of two of the seven defects reported within this pipe segment. Continued in Figure 21.

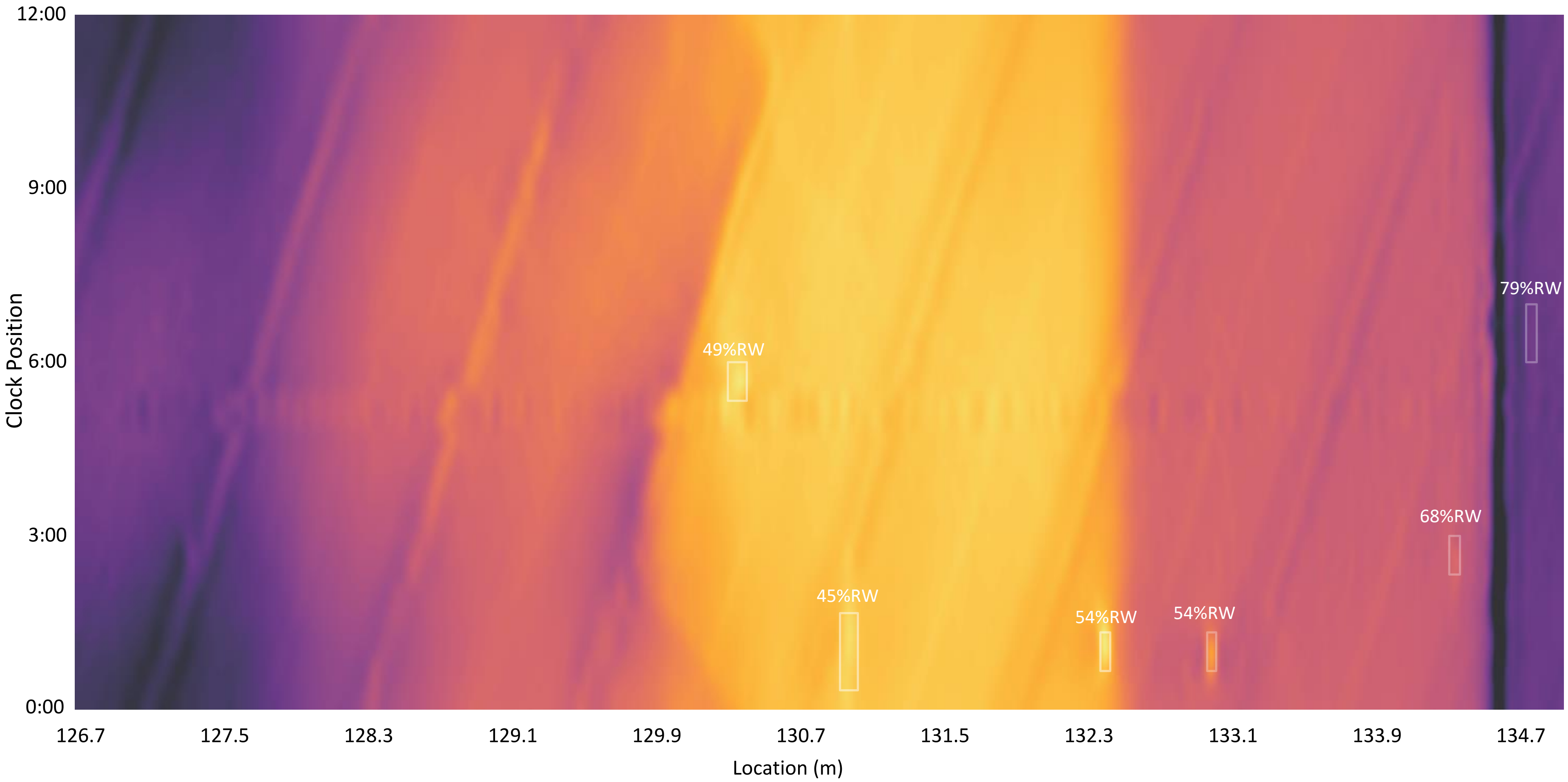


Figure 21. Colour map of pipe numbers 0160 and 0170. Seven defect indications were reported within pipe 0160, along with detected stress located from 129.82m to 134.60m corresponding to the location of an anchor block. The single reported indication within pipe number 0170 is shown past the flange connection at 134.6m.

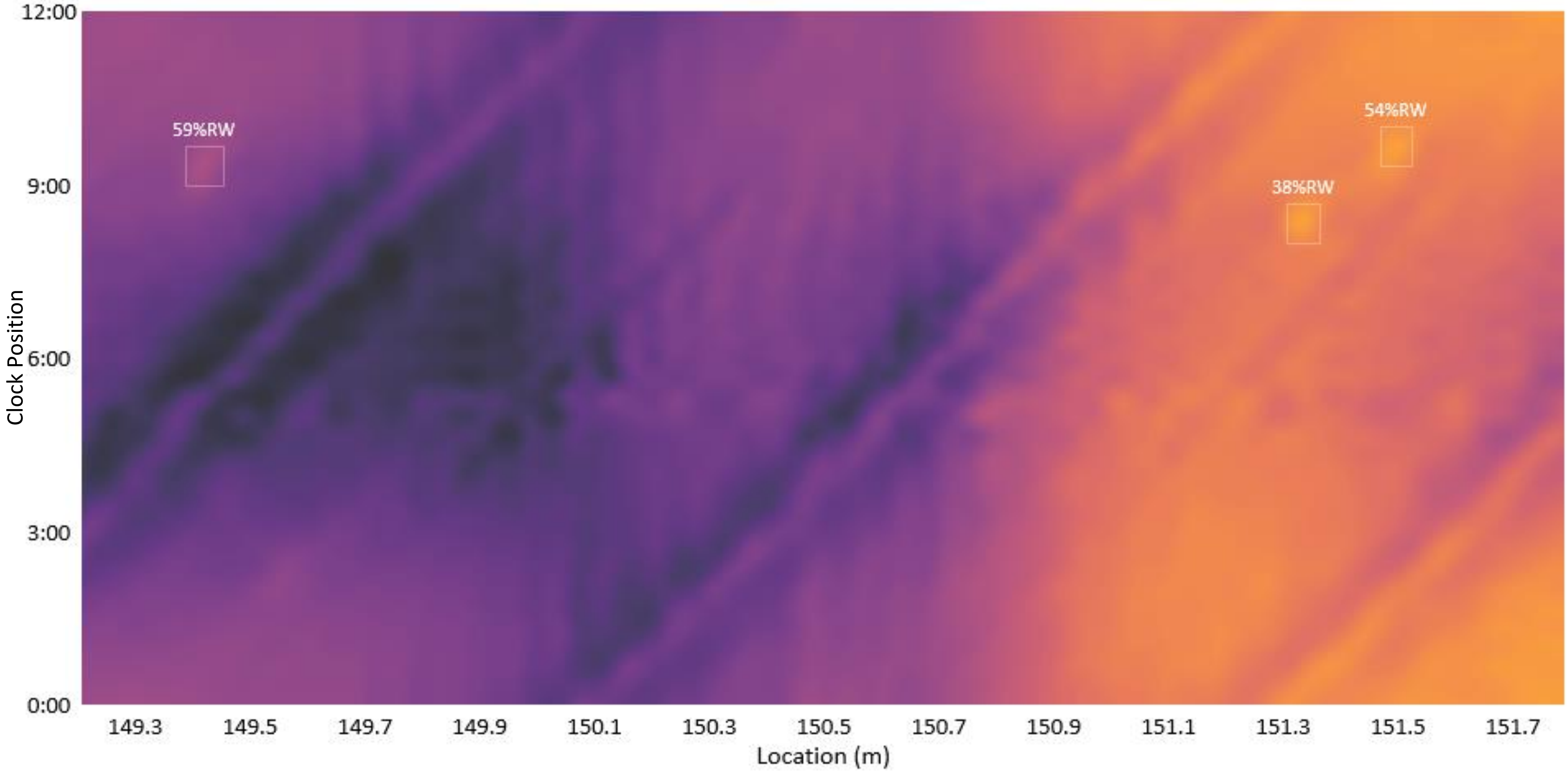


Figure 22. Pipe number 0180 showing the three indications reported within this pipe segment.

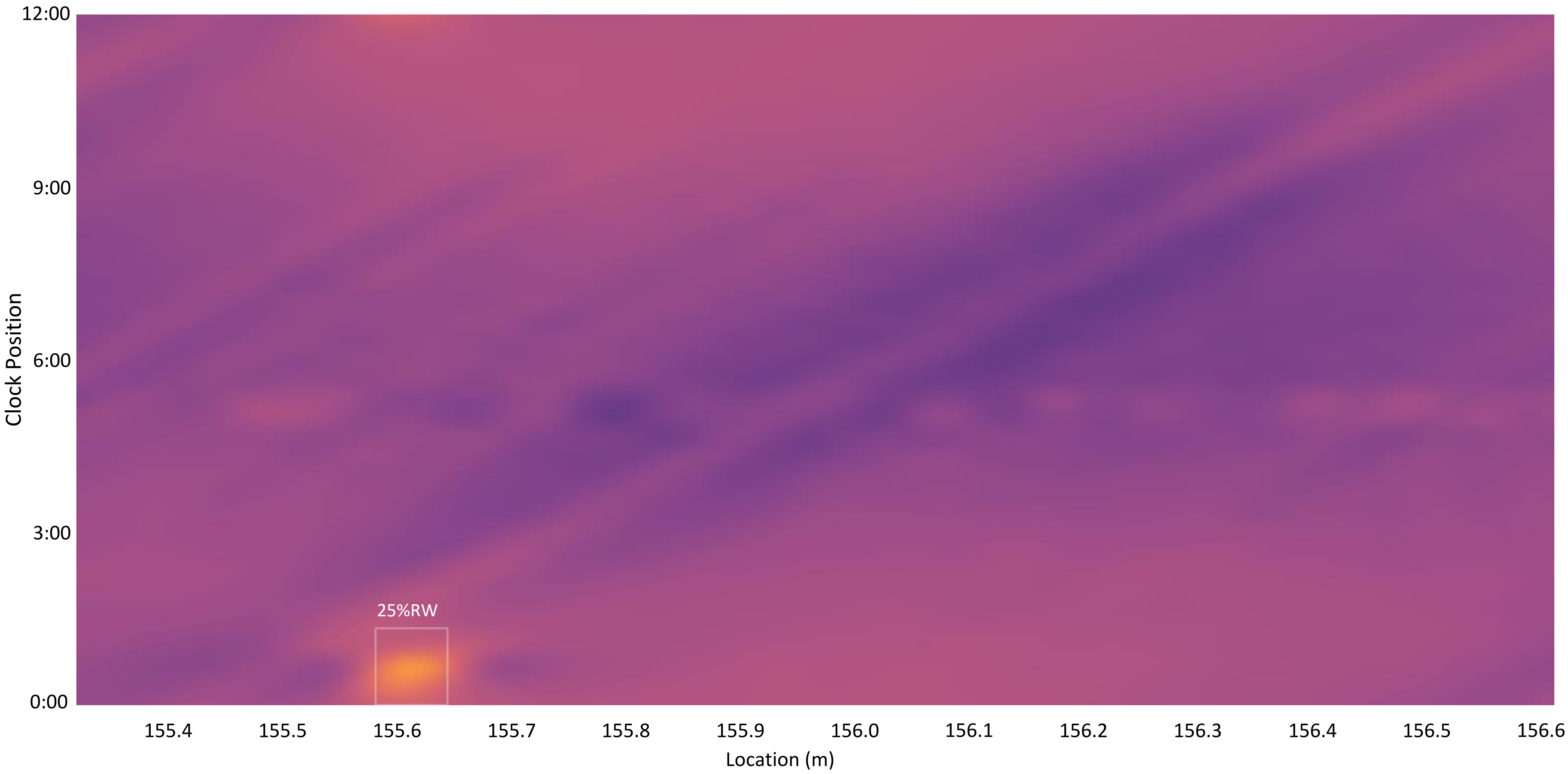


Figure 23. Pipe number 0200 colour map showing the single reported defect indication location within this pipe segment

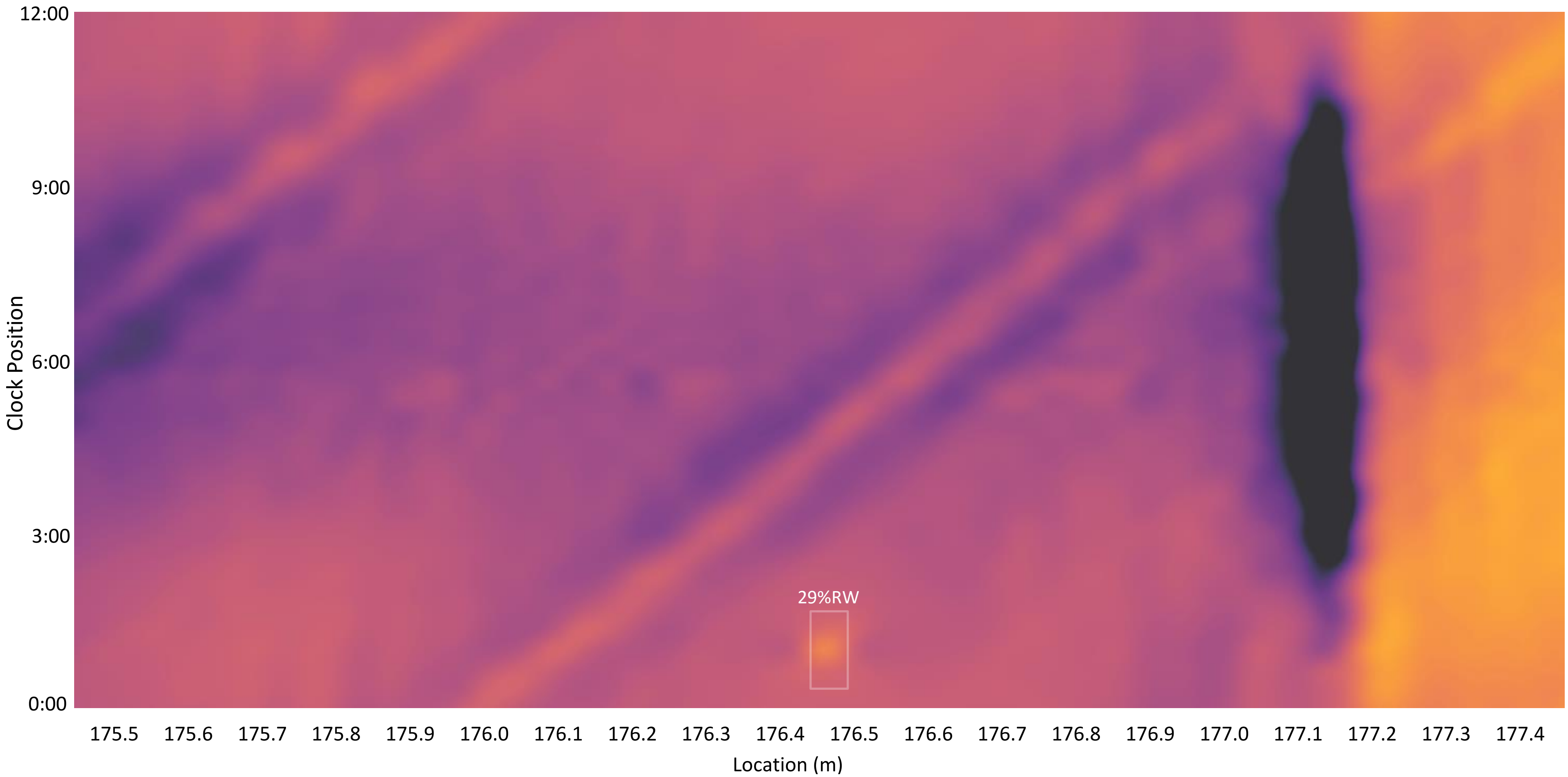


Figure 24. Pipe number 0240 colour map showing the single defect indication reported in this segment, along with a flange connection at 177.14m. This flange connection was noted to be leaking during installation on drawing 7183/64B in 1965. During analysis, special attention was given to this area near the flange connection, though no other wall loss indications were detected.

Appendix 2: RFT Tool Calibration

Prior to arriving on site, PICA performed a test run of the 24in Chimera tool in a 24in diameter, 9.5mm (0.375in) NWT, unlined spiral welded steel pipe to ensure that the tool was in proper working condition. The calibration pipe contained 5.1cm diameter circular flat bottom defects of varying wall loss percentages, and circular through holes (TH) of varying diameters. Machined defects were measured with an Ultrasonic Testing (UT) device to confirm final wall thicknesses. Table 5 provides an overview of the defects present in the calibration pipe.

Table 5. Defects machined into the 600mm diameter, 9.5mm (0.375in) NWT spiral welded steel calibration pipe

Defect Type	Remaining Wall:	Volume of Defect:
Circular Flat Bottom Defects 5.1 cm (2.0 in) diameter:	73%	5.2 cm^3 (0.3 in^3)
	53%	9.1 cm^3 (0.6 in^3)
	20%	15.4 cm^3 (0.9 in^3)
	Diameter of Defect:	**Volume of Defect:**
Circular Through Holes (0% RW):	1.3 cm (0.5in)	1.3 cm^3 (0.1 in^3)
	2.5 cm (1.0in)	4.7 cm^3 (0.3 in^3)
	5.1 cm (2.0in)	19.5 cm^3 (1.2 in^3)
	7.6 cm (3.0in)	43.2 cm^3 (2.7 in^3)
	10.2 cm (4.0in)	77.8 cm^3 (4.7 in^3)

Figure 25 show the circular flat bottom defects, and Figure 27 shows the circular through hole defects machined into the calibration pipe. All defects were visible in the RFT scan of the calibration pipe, including the 1.3cm (0.5in) diameter through hole and the 5.1cm x 73%RW flat bottom defect. Figure 26 shows the RFT scan of the calibration pipe. It is important to note that the results of the calibration may not be directly comparable to the 6.4 mm (0.250in) CML pipe used to construct the Charleswood-Assiniboia Feeder Main due to the differences in the grade and magnetic permeability of the steel pipe material.

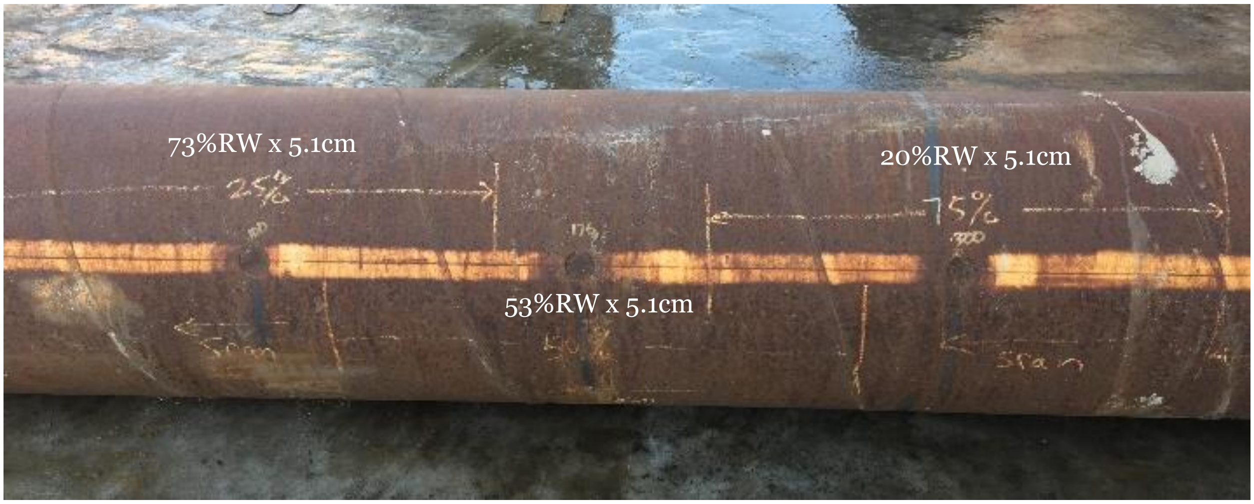


Figure 25. Calibration pipe with 73% RW, 53% RW, and 20% RW 5.1cm diameter flat bottom defects.

Figure 27. Through holes machined into the 24in calibration pipe. Defects measured 1.3 cm, 2.5 cm, 5.1 cm, 7.6 cm, and 10.2 cm (0.5in, 1.0in, 2.0in, 3.0in, and 4.0in) in diameter.

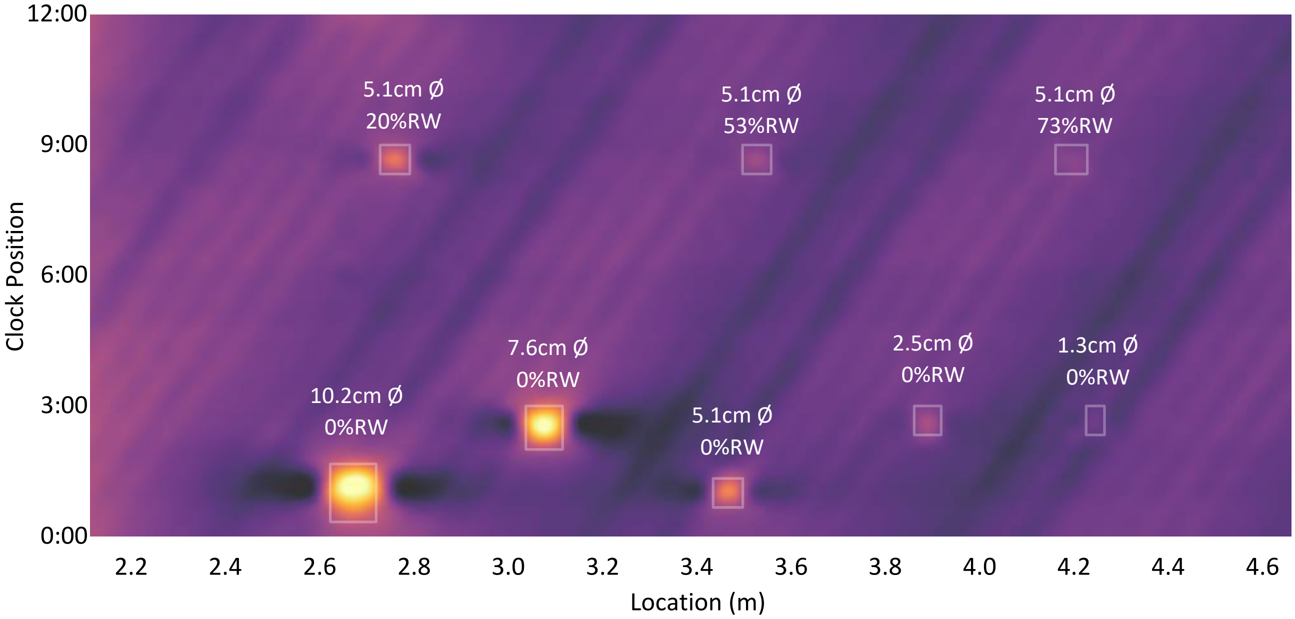


Figure 26. Colour map produced of scanned calibration pipe showing flat-bottom and through hole defects.

Appendix 3: Job Notes and Tool Log

Time (CST)	Operational Comments
Mar 26, 2019	
7:35	Inspection crew arrive on site at Kildonan-Redwood Feeder Main, West Chamber
7:40	First (mechanical Winch) skid steer off trailer
7:48	Laptop synced
8:20	Tool assembled
9:00	Odometer calibrated
9:14	Chimera tool powered up
9:32	Zeroed the wireline odometer with odometer wheel flush with pipe opening.
9:33	Tool scans at 7 Hz with detector leading as the tool begins its descent through the vertical piping
9:45:15	Tool reaches bottom of vertical piping, begin to pull tool back up
10:40	Doing rerun at 5hz with Exciter leading
11:12	Stopped recording
11:14	Restart tool at 5Hz for detector leading run
12:06	East side crew leaves with tool in preparation for inspection of horizontal section.
13:30	Wireline odometer zeroed when Exciter end plate is flush with white painted pipe opening at 13:30. Begin inspection with detector leading at 14 Hz. Start logging at 13:34.
15:13	AGM passage time (west side, AGM Q08874)
15:38	Tool arrived at 90 degrees elbow – start retrieve.
17:05	East side all packed up. Finished pulling tagline back into main from East to West.
17:20	Leave Site
Mar 27, 2019	
7:43	Arrival at Kildonan West side. Tail gate safety meeting.
8:02	Tool is brought to East Chamber
9:00	Tagline is pulled out on the East side, bringing up west winchline.
9:19	Chimera tool powered up. Begin inspection of horizontal portion of Feeder Main at 14Hz.
10:10	Tool passing AGM P40171 on East side of river.
11:21	Tool passing AGM Q08874 on West side of river.
11:49	Tool arrived at 90-degree elbow.
11:59	Tool passing AGM Q08874 on West side of river.
13:02	Data download complete.
13:14	Start up Chimera tool for last 10 Hz run.
13:31	Tool launch east to west at 10Hz.
14:12	Tool passes AGMs on West side of river.
14:41	Tool arrived at 90-degree elbow, start retrieve.
15:16	Tool arrives at East chamber.
16:45	Leave site.
Mar 28, 2019	
7:43	Arrive on site at Charleswood-Assiniboia, north excavation access.
10:25	Run Start north to south
11:02	Passage of AGM Q08874 on North side of river.
12:08	Tool arrives at south excavation
14:00	Winch slipped at South Chamber. Tool dropped and requiring repairs.
16:00	Zeroing wireline odometers with trailing conical pig 3 inch into the pipe.
16:08	Tool launch south to north.
16:38	Passage of AGM P40171 on South side of river.
17:21	Passage of AGM Q08874 on North side of river.
17:55	Tool arrives

Appendix I

Leakage Test Reports

AECOM
99 Commerce Drive
Winnipeg, MB, Canada R3P 0Y7
www.aecom.com

204 477 5381 tel
204 284 2040 fax

Inspection Report

Project:	Provision of Pipeline Access Modifications, Cleaning and Support Services for River Crossing Inspections – Phase Two - Bid Opportunity No. 492-2018	**Date:**	November 23, 2018
Contractor:	J-Con Civil Ltd. (J-Con)	**Project No.**	60549028 (601.7)
Owner:	The City of Winnipeg, Water & Waste Department	**Weather:**	

Present:			
	Gerry Jones	-	J-Con
	Justin Jones	-	J-Con
	Jaret	-	J-Con
	Dylan	-	J-Con
	Tanner Beavis	-	AECOM

Purpose: Inspection of Low Pressure Leakage Test on Newton Avenue Force Main

Friday, November 23, 2018:

- 12:35 p.m. - J-Con arrived on the west side of the crossing and began the installation of a blind flange with a ball valve on the force main. The flanges were installed so that the ball valves were located at the top of pipe. See attached diagram.

Figure 1: Blind flange with ball valve used for the leakage test.

- 2:24 p.m. - The crew had finished installation on the west and moved to the east side of the crossing.
  - A second blind flange was installed on the east side of the force main and a 2" hose serving as a standpipe was connected and tied off at the top of the manhole.
- 3:21 p.m. - J-Con decided to use a 2" submersible pump in the river to pump water to the standpipe.
  - The pump was used prior to arrival of the water truck and due to concerns that it did not contain enough water to fill the siphon in one load.
  - It was observed that the pressure in the hose at the time of pumping was minimal.

- 4:08 p.m. - Water was allowed to flush through the force main and out the west ball valve for 5 minutes.
- 4:13 p.m. - The pump was shut off and the valve on the west side was shut seconds after getting confirmation that the pump was off from Gerry Jones via cellphone.
- 4:16 p.m. - The crew began filling the standpipe using water from a 20 L container (jerry can) in an attempt to top up the pipe.
    - The jerry can was confirmed to be clean.
    - Two to three full jerry cans of water were poured into the standpipe and it was observed that the water level was not rising in the standpipe.
    - The leakage rate (rate in which make up water was required) was found to be in excess of what could be provided by manually adding with containers.
- 4:33 p.m. - The standpipe was detached from the manhole and brought up to the water truck outlet to increase the flow of water that could be poured into the line in an attempt to top up the standpipe.
    - The green 2” hose (pictured in Figure 2) was attached to the inlet of the water truck and the pump was used to replenish the water levels of the truck from the river.

Figure 2: J-Con using the water truck to fill the standpipe.

- 4:44 p.m. - The standpipe hose was swapped out to enable the use of a 4” reducer to be connected at the top of the hose at the manhole, serving as a funnel. A length of hose was connected to the water truck for the purpose of filling the standpipe. The top of the standpipe remained at ground elevation for the remainder of the test. See Figure 3.

Figure 3: The standpipe configuration with 4” reducer after switching hoses.

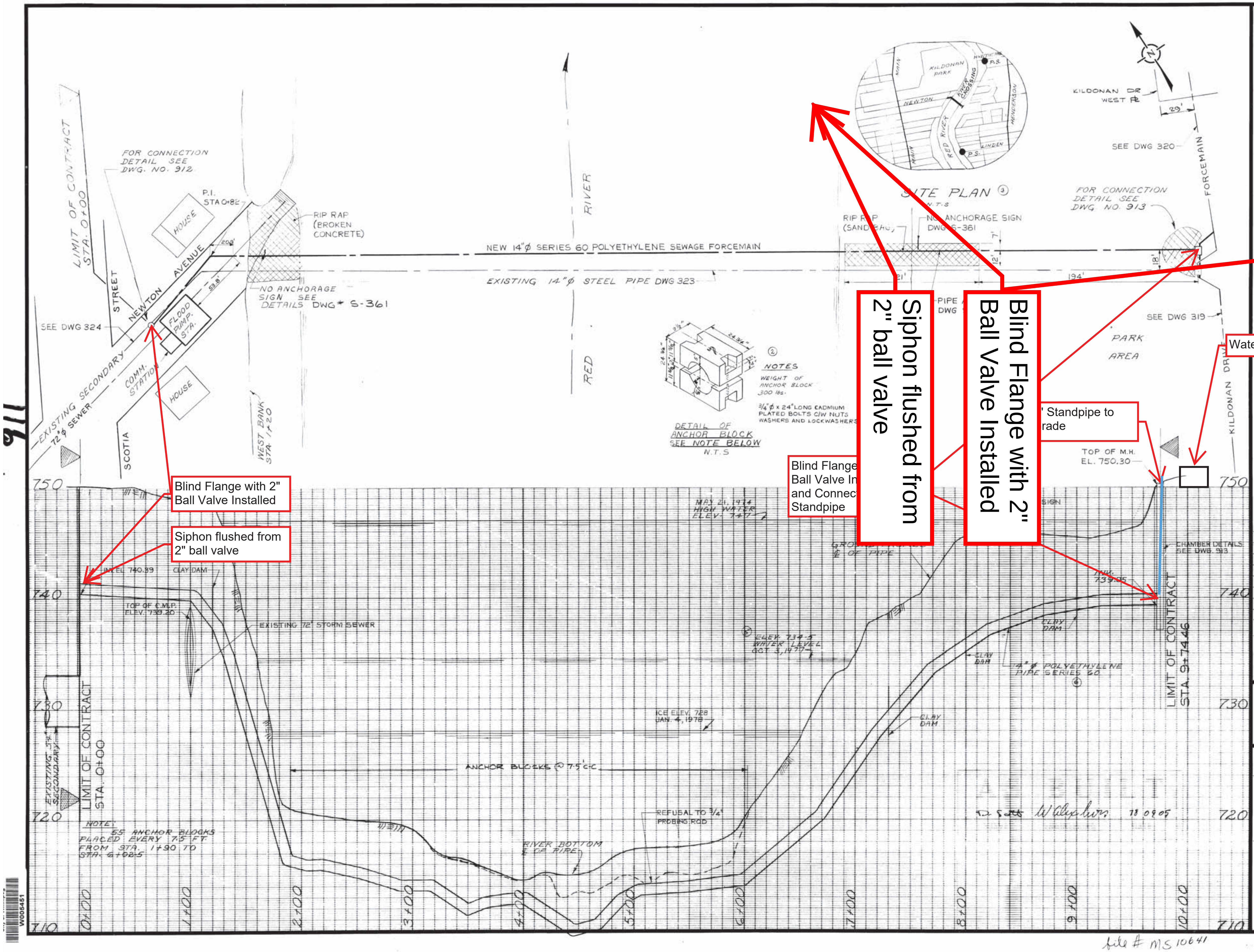
- 4:45 p.m. to 6:10 p.m.
  - Water was steadily poured from the water truck into the standpipe.
    - When the amount of flow was increased the water in the standpipe would reach the ground surface elevation, but continuously fell if the flow was shut off or reduced.
    - The water level was visible in the standpipe and it was observed to be fluctuating up and down when flow was shut off or reduced.
  - Approximately 25 minutes into testing the rate of flow from the water truck had been dialed back to match the rate at which water was leaving the standpipe. The elevation in the pipe remained relatively constant, but was observed to fluctuate up and down slightly.
  - Once the water elevation was observed to be relatively constant at the ground surface elevation, the hose was transferred into a 20 liter container and the time required to fill was measured with a stop watch.
    - The 20 liter container was filled in approximately 90 seconds.
  - The flanges and hoses were checked for leaks at both ends of the siphon during the beginning and midway points of the test
  - Approximately midway through the test period two crew members cracked the ball valve in the west manhole to check for trapped air in the siphon.
    - The pipe was found to be full of water. No air was observed or heard leaving the valve.
- 6:10 p.m. - The flow from the water truck was shut off and testing was concluded. J-Con was instructed to leave their equipment on site in case further testing was required the following week.

Monday, November 26, 2018:

- J-Con returned to measure where the water level in the standpipe had balanced out to over the weekend.
    - J-Con informed that they were not able to see the water level in the green standpipe but when they removed the pipe there was approximately 1 to 2 feet of water in the pipe.
    - It was later determined that this measurement was 1 to 2 feet of water in the pipe where it coiled horizontally from the flange, and not a vertical measurement of elevation.

for Tanner Beavis, E.I.T.
Municipal Engineer in Training
Conveyance
TJB/pab
Encl.

cc: A. Delaurier, WWD (adelaurier1@winnipeg.ca)
J. Jones, J-Con (JJones@J-Concivil.com)
M. McDonald, AECOM
N. Kehler, AECOM
A. Braun, AECOM



LEGEND

WATERWORKS, WASTE AND DISPOSAL : UNDERGROUND UTILITIES

- PROPOSED FORCEMAIN, INTERCEPTOR OR SECONDARY SEWER
- EXISTING FORCEMAIN, INTERCEPTOR OR SECONDARY SEWER — W.W.D. 24"Ø SEWER
- PROPOSED FEEDERMAIN
- EXISTING FEEDERMAIN — W.W.D. 24"Ø W.M.
- W.W.D. — WATERWORKS, WASTE AND DISPOSAL

OTHER UNDERGROUND UTILITIES

- COMMUNITY SEWER
- COMMUNITY WATERMAIN
- GAS
- HYDRO DUCT
- M.T.S. DUCT OR CABLE
- STREET LIGHTING, RAILWAY SIGNAL OR TRAFFIC SIGNAL CABLE

SURFACE UTILITIES

- COMMUNITY BOUNDARY LINE
- FENCE LINE
- POLE LINE
- RAILWAY LINE

MISCELLANEOUS SYMBOLS

- BASIN
- CULVERT
- HYDRANT
- MANHOLE
- HYDRO OR M.T.S. POLE
- LAMP STANDARD
- RAILWAY OR TRAFFIC SIGNAL STANDARD
- VALVE

NOTES

1.

NO.	DESCRIPTION	DATE	BY
⑦	REVISED TO AS BUILT	AUG 18/78	
⑥	SERIES 80 CHANGED TO 60	OCT 6/77	VD.
⑤	SUMMER WATER LEVEL ADDED	OCT. 6/77	VD.
④	NOTE - PIPE ALIGNMENT	OCT 6/77	VD
③	SITE PLAN REVISED	OCT. 6/77	V.D
②	ANCHOR BLOCK DIM. CHANGED, NOTE ADDED	OCT 6/77	V.D
①	RIVER CROSSING X-SECTION REMOVED	OCT-5/77	VD

REVISIONS

FIELD BOOK(S) NO. F 472
LAND SURVEY PLAN(S) NO.
B. M. GEO. B.M 12-005
SURVEYED BY R. TAPLIN
DATE Nov. 1, 1976

The City of Winnipeg
Waterworks, Waste and Disposal Division

ENGINEERING BRANCH
WASTE DISPOSAL

NEWTON RIVER X-ING
PLAN PROFILE

DESIGNED BY B. BRICK
DRAWN BY V. DUMORE
CHECKED BY Dan Weiler
DATE JUNE 1977
APPROVED BY
SCALE HOR. - 1" = 40' VERT. - 1" = 4'
DRAWING NO. 911

File # MS 10641

MICROFILMED MAY 11 1983

AECOM
99 Commerce Drive
Winnipeg, MB, Canada R3P 0Y7
www.aecom.com

204 477 5381 tel
204 284 2040 fax

Inspection Report

Project:	Provision of Pipeline Access Modifications, Cleaning and Support Services for River Crossing Inspections – Phase Two - Bid Opportunity No. 492-2018	**Date:**	November 28, 2018
Contractor:	J-Con Civil Ltd.	**Project No.**	60549028 (601.7)
Owner:	The City of Winnipeg, Water & Waste Department	**Weather:**	

Present:	Jaret	-	J-Con Civil Ltd. (J-Con)
	Dylan	-	J-Con Civil Ltd. (J-Con)
	Tanner Beavis	-	AECOM

Purpose: Inspection of Low Pressure Leakage Test on Newton Avenue Force Main

Wednesday, November 28, 2018:

- 1:55 p.m. - J-Con was on site with water truck upon AECOM inspector arrival to site. A clear 2” hose had been attached to the valve on the blind flange and tied at the manhole to serve as a standpipe for testing. A 4” reducer had been attached to the standpipe to serve as a funnel. See, Figure 1.

Figure 1: Clear standpipe assembly at the east chamber.

- 1:57 p.m. - J-Con began filling the standpipe with water from the water truck using a 2” hose.
  - The water “gargled” as it was initially poured down the standpipe but subsided within seconds, and the sound of air was not observed.
  - No leaks were observed in the chamber.
- 2:08 p.m. - water was observed approximately midway up the standpipe in the chamber. The water was turned off and it was measured that the water dropped steadily by 8 inches in 122 seconds.
- 2:20 p.m. to 3:30 p.m. - Filling of the standpipe recommenced and was adjusted from the water truck until the fill rate was approximately equivalent to the loss rate and the water elevation remained constant at the ground surface elevation.
  - Once the water elevation was observed to be relatively constant at the ground surface elevation, the hose was transferred into a measuring container and the time required to fill 8 liters was measured with a stop watch.
  - 2:30 p.m. - An 8 liter volume was filled in 32.5 seconds.
  - 3:28 p.m. - Another measurement was taken of the flow prior to conclusion of testing. An 8 liter volume was filled in 34.7 seconds.
  - The rate of flow being poured into the standpipe remained constant for the remainder of the test, which concluded at 3:30 p.m.
  - During the last half hour of testing, Jaret of J-Con drove to the west side of the crossing to check for leaks. Jaret was accompanied by the City’s Water and Waste Department.
    - No leaks were observed in the west manhole.
    - The valve was not cracked to check for trapped air during testing.
- 3:30 p.m. - The test was concluded and the test equipment was left on site overnight to allow the water level in the standpipe to stabilize.

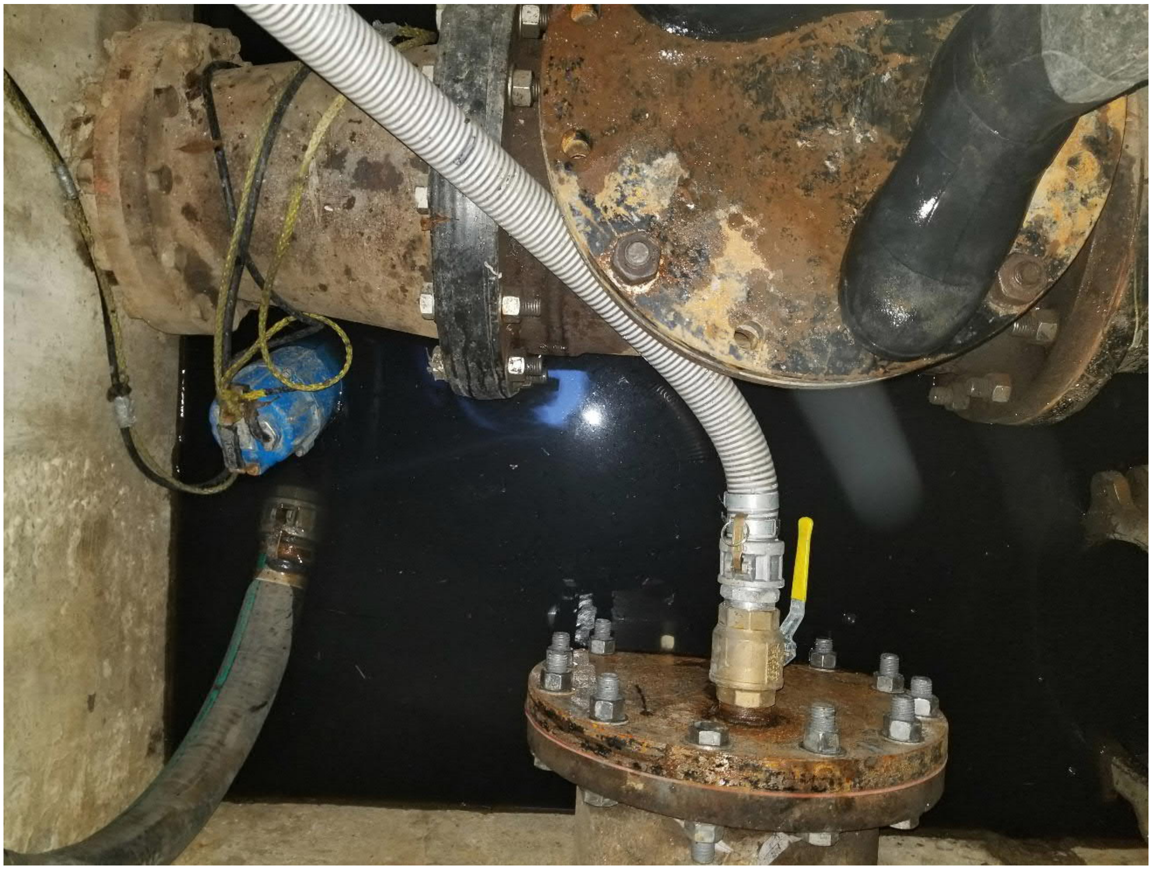


Figure 2: Top down view of the standpipe assembly. Black line made with marker shows water level after water was left to stabilize overnight.

Thursday, November 29, 2018:

- 8:03 a.m. - J-Con returned to site to measure the stabilized water level in the force main at the upstream chamber.
    - The elevation of the water in the standpipe was measured to be 566 mm above the invert of the force main.
- During disassembly of the blind flanges an air pocket was discovered in the force main on the west side of the river. The water level in the pipe was between the invert and the ball valve.
    - No water was present when the ball valve was opened.
    - Water was present in the pipe when the blind flange was removed. The exact depth is unknown.

for Tanner Beavis, E.I.T.
Municipal Engineer in Training
Conveyance
TJB/pab

cc: A. Delaurier, WWD (adelaurier1@winnipeg.ca)
J. Jones, J-Con (jjones@J-Concivil.com)
M. McDonald, AECOM
N. Kehler, AECOM
A. Braun, AECOM

AECOM
99 Commerce Drive
Winnipeg, MB, Canada R3P 0Y7
www.aecom.com

204 477 5381 tel
204 284 2040 fax

Inspection Report

Project:	Provision of Pipeline Access Modifications, Cleaning and Support Services for River Crossing Inspections – Phase Two – Bid Opportunity No. 492-2018	**Date:**	March 4, 2019
Contractor:	J-Con Civil Ltd. (J-Con)	**Project No.**	60549028 (601.7)
Owner:	The City of Winnipeg, Water & Waste Department	**Weather:**	

Present:			
	Gerry Jones	-	J-Con
	Bruno	-	J-Con
	Nick	-	J-Con
	Wendal	-	J-Con
	Jaret	-	J-Con
	Brad Kreitz	-	AECOM

Purpose: Inspection of Low Pressure Leakage Test on Newton Avenue Force Main

Monday March 4, 2019:

- J-Con mobilized to upstream valve chamber in Fraser’s Grove Park to prepare for the low pressure leakage test. Bruno and Nick on site at 08:00 to set up pump and hoses. Water pump would not start and they were required to wait for a new pump to be delivered from their shop.
- The hoses to be used to fill the water tank from the river were found to be blocked and frozen, requiring the use of a tiger torch to thaw them. Wendal arrived with the replacement pump at the upstream chamber at 12:30 to help with setup of the new water pump and to assist in thawing hoses. Jaret was stationed at downstream manhole on Scotia St to monitor the valve during the test.
- At 12:45 J-Con had finished thawing the hoses and they began to fill the water tank on the back of the water truck. At this time, they began filling the pipe by allowing water to flow from the water tank into the vertical standpipe connected to the blind flange/ball valve in the upstream valve. Workers at both ends of the pipe verified that both ball valves were in the fully open position and unobstructed.
- At 14:00 it was noticed that the rate at which the level of the water tank was dropping had reduced to almost zero. To investigate, J-Con decided to disconnect the hose running from the water tank to the top of the vertical standpipe at grade. Water was immediately observed flowing back up out of the standpipe, stabilizing at ground level after several minutes. A worker stationed at the downstream end of the pipe confirmed that there was still no water exiting the pipe at the downstream valve.
- This condition remained constant from 14:00 to approximately 15:00 while J-Con worked to identify the source of the problem. At 15:00 the valves at both ends of the pipe were double checked for defects and to ensure there were no physical blockages. None were found and it was concluded that the pipe itself must be blocked. Due to the water level in the standpipe remaining constant at grade during the investigation, it was also concluded that the section of pipe between the upstream chamber and the blockage was not leaking noticeably.
- The site was secured for the day at 15:15 and J-Con confirmed that they would be back on Wednesday March 6 to remove the blind flanges from both ends of the pipe.

Site Photos:

Photo 1 - Blind flanges with ball valves already installed on both ends of force main

Photo 2 - Setting up water pump and thawing hoses

Photo 3 - Preparing to begin filling pipe at upstream valve chamber in Fraser's Grove Park

for Brad Kreitz
Municipal Technologist
Conveyance
BK/pab

cc: A. Delaurier, WWD (adelaurier1@winnipeg.ca)
J. Jones, J-Con (jjones@j-concivil.com)
M. McDonald, AECOM
N. Kehler, AECOM
A. Braun, AECOM

AECOM
AECOM
99 Commerce Drive
Winnipeg, MB, Canada R3P 0Y7
www.aecom.com
204 477 5381 tel
204 284 2040 fax

Inspection Report

Project: Provision of Pipeline Access Modifications, Cleaning and Support Services for River Crossing Inspections – Phase Two – Bid Opportunity No. 492-2018

Date: April 9, 2019

Contractor: J-Con Civil Ltd. (J-Con)

Project No. 60549028 (601.7)

Owner: The City of Winnipeg, Water & Waste Department

Weather:

Present: 2 Crew members	-	J-Con
2 Crew members	-	McDougall's Enterprise
Brad Kreitz	-	AECOM

Purpose: Inspection of Leakage Test on Kildonan-Redwood FDM

Tuesday April 9, 2019:

- J-Con and McDougall mobilized to Kildonan-Redwood FDM to conduct leakage test at 08:00. Water was introduced to the feeder main via the new 300 mm gate valve adjacent to the valve chamber on the east side of the river near the intersection of Hespeler Avenue and Glenwood Crescent. (hereafter referred to as *the east chamber*) McDougall's pressure testing trailer was connected to the 75 mm blow-off valve on the top of the new cross piece in the east chamber when the inspector arrived on site at 09:00. (See Figure 1)
- McDougall reportedly flushed all of the air out of the same 75 mm blow-off valve in the east chamber before connecting hose to trailer. Air was also reportedly flushed out of the temporary blind flange with ball valve at the top of the vertical pipe in the west chamber.
- A small leak was reported by J-Con personnel on the connection between the blind flange and the ball valve in the west chamber. This leak was estimated to be only approximately 0.5 L in 5 minutes. Alex McDougall recommended proceeding with the test.
- The feeder main was charged to city pressure (74 psi) by opening the new 300 mm gate valve. The 300 mm gate valve was then closed and the test began at approximately 09:15. The pressure immediately began to drop and Alex McDougall measured a leak of approximately 78 L over 15 minutes. The test was then aborted to investigate possible causes of error or locations where leakage might be occurring. No leaks were observed in the east chamber and Alex deemed the leak on the blind flange in the west chamber to be too small to account for the test results. Communicated results of the test to the city to seek direction on how to proceed.

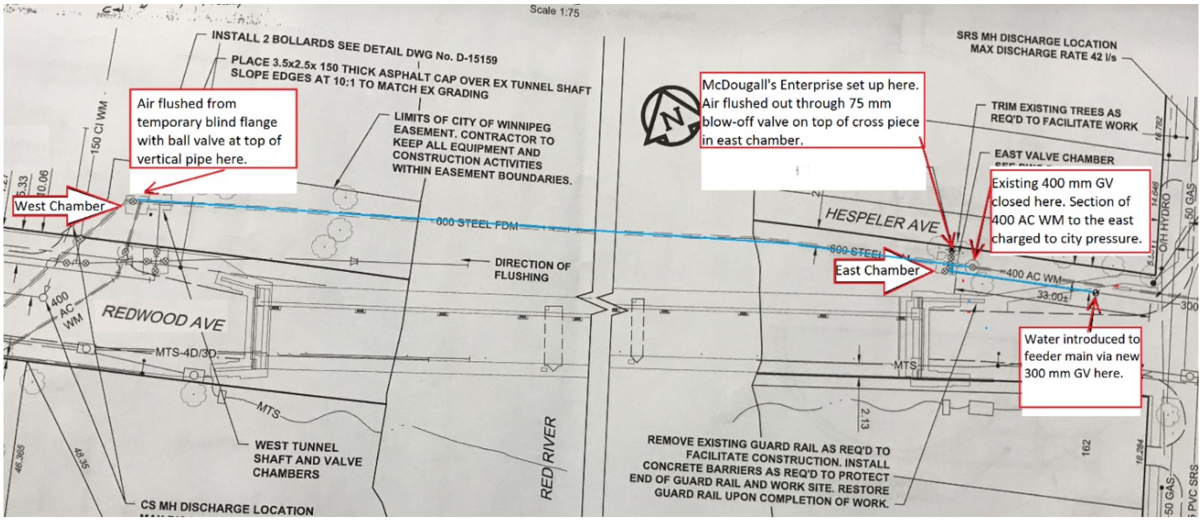


Figure 1-Diagram of leakage test set-up

Site Photos:

Photo 1 - McDougall operating new 300 mm valve to charge pipe to city pressure

Photo 2 - Hoarded hoses connecting McDougall pressure testing trailer to 75 mm blow-off on east chamber cross piece

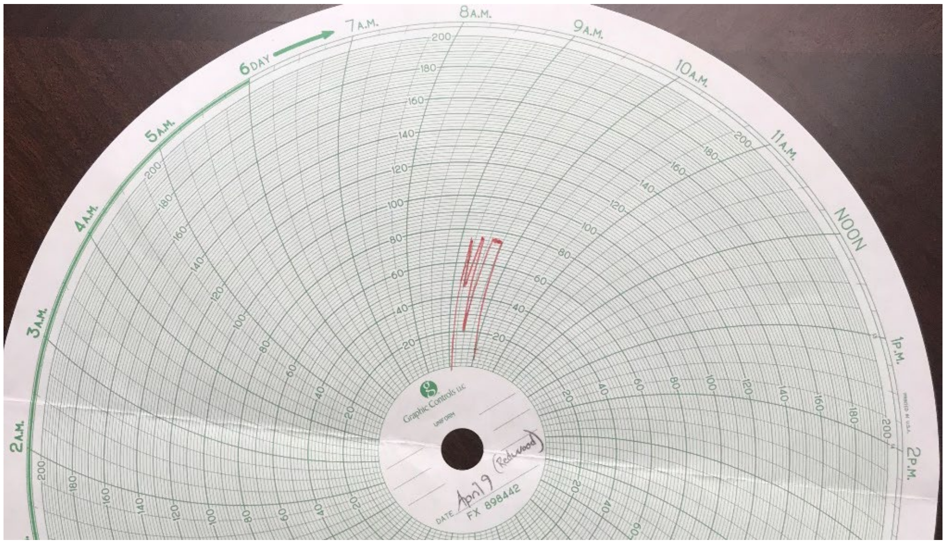
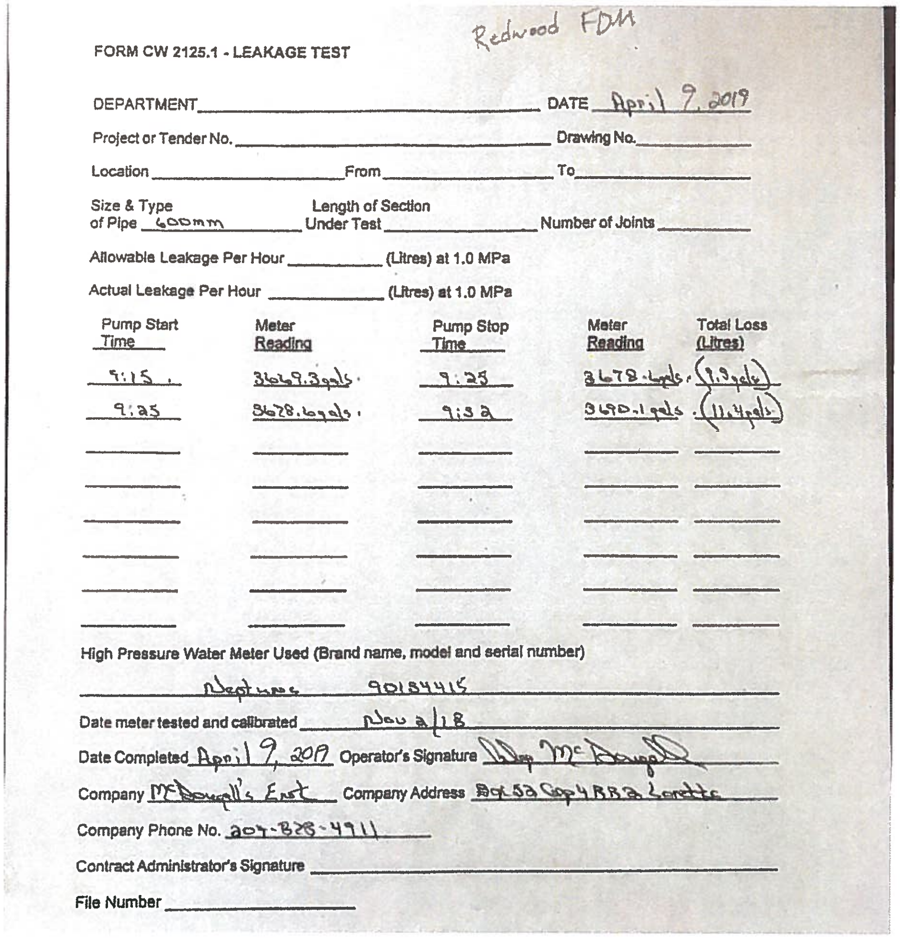


Photo 3 - McDougall's Enterprise Pressure Test Chart



Redwood FDM

FORM CW 2125.1 - LEAKAGE TEST

DEPARTMENT ____ DATE April 9, 2019

Project or Tender No. ____ Drawing No. ____

Location ____ From ____ To ____

Size & Type of Pipe 600mm Length of Section Under Test ____ Number of Joints ____

Allowable Leakage Per Hour ____ (Litres) at 1.0 MPa

Actual Leakage Per Hour ____ (Litres) at 1.0 MPa

Pump Start Time	Meter Reading	Pump Stop Time	Meter Reading	Total Loss (Litres)
9:15	3669.3gals	9:25	3678.6gals	(9.3gals)
9:25	3678.6gals	9:32	3690.1gals	(11.4gals)

High Pressure Water Meter Used (Brand name, model and serial number)

Neptune 90184415

Date meter tested and calibrated Nov 2/18

Date Completed April 9, 2019 Operator's Signature McDougall

Company McDougall's Ent Company Address Box 52 Grp 4 RR 2 Lorette

Company Phone No. 204-878-4911

Contract Administrator's Signature ____

File Number ____

Photo 4 - McDougall's Enterprise Leakage Test Report

for

Brad Kreitz
Municipal Technologist
Conveyance
BK/pab

cc: A. Delaurier, WWD (adelaurier1@winnipeg.ca)
J. Jones, J-Con (jjones@j-concivil.com)
M. McDonald, AECOM
N. Kehler, AECOM
A. Braun, AECOM

AECOM
99 Commerce Drive
Winnipeg, MB, Canada R3P 0Y7
www.aecom.com

204 477 5381 tel
204 284 2040 fax

Inspection Report

Project: Provision of Pipeline Access Modifications, Cleaning and Support Services for River Crossing Inspections – Phase Two – Bid Opportunity No. 492-2018

Date: April 11, 2019

Contractor: J-Con Civil Ltd. (J-Con)

Project No. 60549028 (601.7)

Owner: The City of Winnipeg, Water & Waste Department

Weather:

Present:

2 Crew members	-	J-Con
2 Crew members	-	McDougall's Enterprise
Brad Kreitz	-	AECOM

Purpose: Inspection of Leakage Test on Charleswood-Assiniboia FDM

Thursday April 11, 2019:

- J-Con and McDougall mobilized to Charleswood-Assiniboia FDM at 11:00 to conduct leakage testing on the feeder main. Hydrant at Southboine Dr. and Berkley St. used to fill McDougall's pressure testing trailer tank. Trailer connected to 50 mm blow-off valve on the feeder main in the valve chamber on the south side of the river. (See Figure 1) McDougall was not equipped to directly measure city pressure prior to the test.
- At 12:17 McDougall charged the line to 63 psi, which he estimated to be approximately 5 psi below city pressure. By 12:24 the pressure had dropped to 45 psi. Alex charged the line back up to 62 psi and measured a leak of 109 L over 20 minutes.
- It was determined that the most probable reason for the drop in pressure was bypassing of the feeder main valve in the south valve chamber. Marcel of City of Winnipeg Emergency Services confirmed that this feeder main valve in the south chamber was suspected to be bypassing and that the adjacent section of feeder main south of the south valve chamber was currently dewatered. (See Figure 1- secondary section referred to above highlighted in green) For this reason, the test was aborted to allow time for city personnel to re-pressurize this secondary section of feeder main.
- J-Con and McDougall to return on Monday, April 15, 2019 to run the test again once secondary section of feeder main south of the crossing is returned to city pressure.

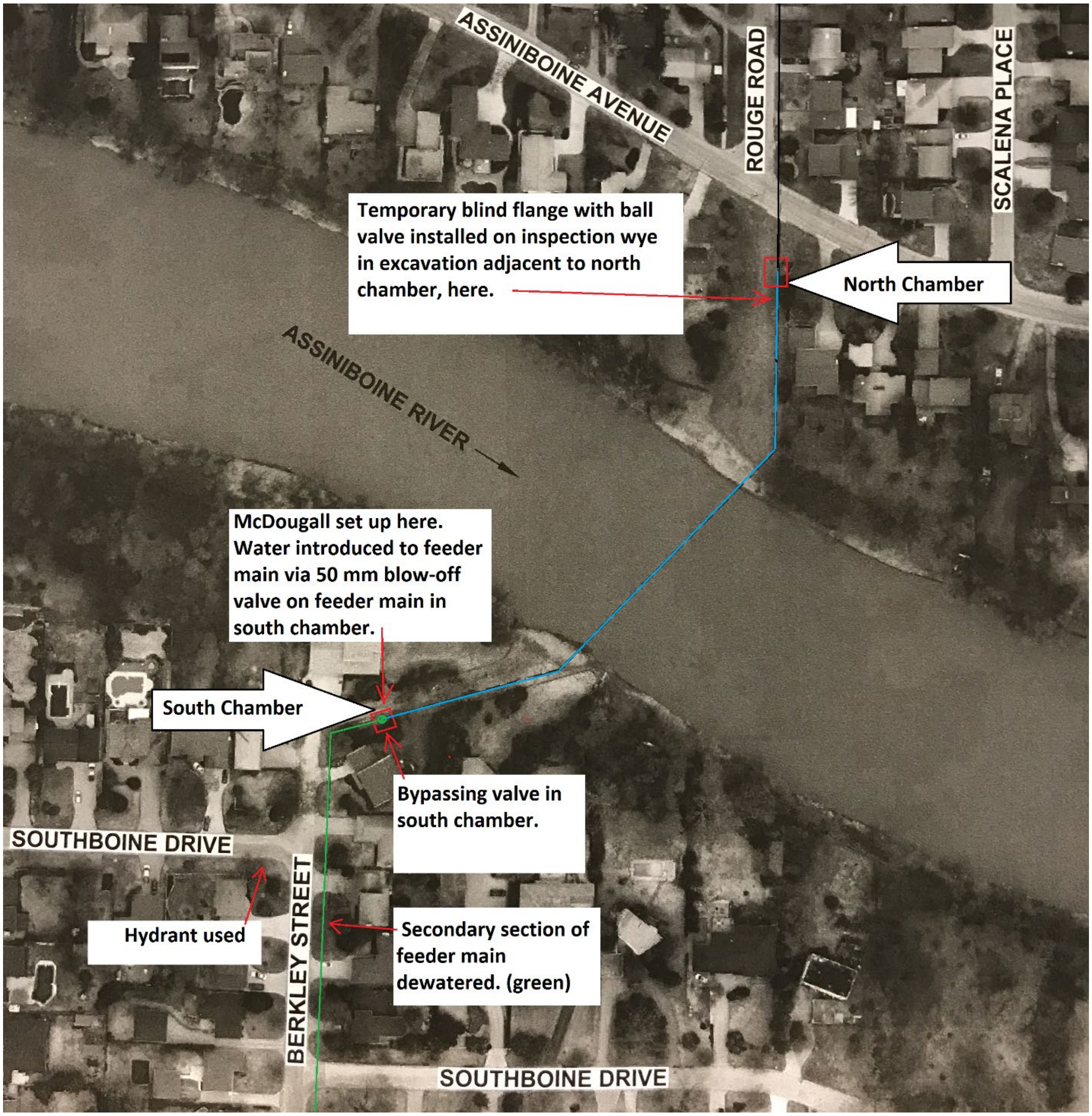


Figure 1 - Diagram of leakage test set-up.

Site Photos:

Photo 1 - McDougall's pressure testing trailer connected to 50 mm blow-off in south chamber

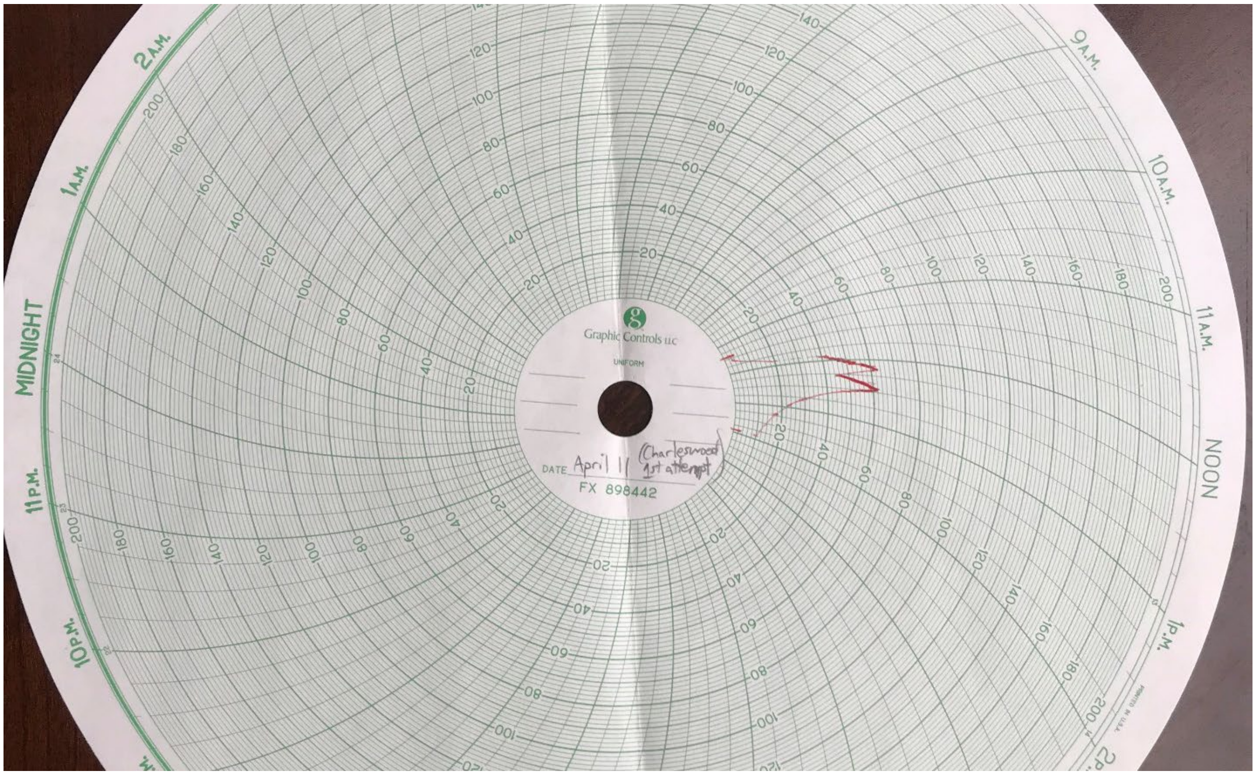
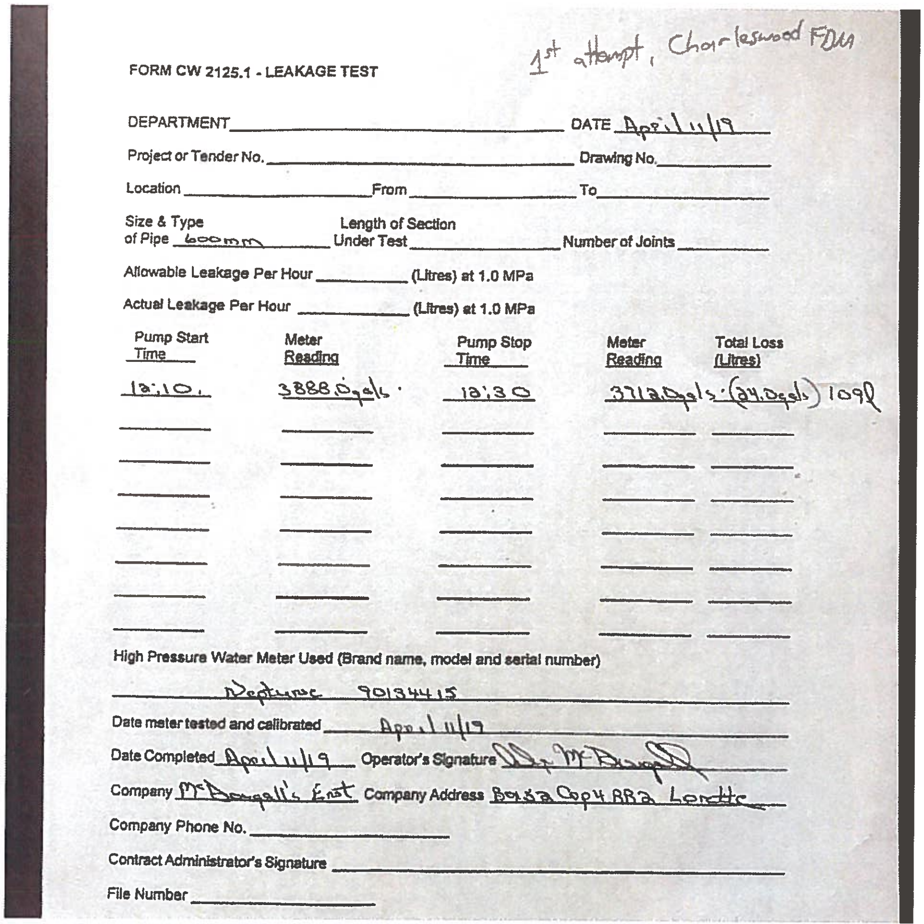


Photo 2 - McDougall's Enterprise Pressure Test Chart



FORM CW 2125.1 - LEAKAGE TEST

1st attempt, Charleswood FDM

DEPARTMENT ____ DATE April 11/19

Project or Tender No. ____ Drawing No. ____

Location ____ From ____ To ____

Size & Type of Pipe 600mm Length of Section Under Test ____ Number of Joints ____

Allowable Leakage Per Hour ____ (Litres) at 1.0 MPa

Actual Leakage Per Hour ____ (Litres) at 1.0 MPa

Pump Start Time	Meter Reading	Pump Stop Time	Meter Reading	Total Loss (Litres)
12:10.	3888.0gals.	12:30	3712.0gals.	(24.0gals) 109l

High Pressure Water Meter Used (Brand name, model and serial number)

Neptune 90134415

Date meter tested and calibrated April 11/19

Date Completed April 11/19 Operator's Signature McDougall

Company McDougall's Ent Company Address Box 32 Gp4 RR2 Lorette

Company Phone No. ____

Contract Administrator's Signature ____

File Number ____

Photo 3 - McDougall's Enterprise Leakage Test Report

for

Brad Kreitz
Municipal Technologist
Conveyance
BK/pab

cc: A. Delaurier, WWD (adelaurier1@winnipeg.ca)
J. Jones, J-Con (jjones@j-concivil.com)
M. McDonald, AECOM
N. Kehler, AECOM
A. Braun, AECOM

AECOM
AECOM
99 Commerce Drive
Winnipeg, MB, Canada R3P 0Y7
www.aecom.com
204 477 5381 tel
204 284 2040 fax

Inspection Report

Project:	Provision of Pipeline Access Modifications, Cleaning and Support Services for River Crossing Inspections – Phase Two – Bid Opportunity No. 492-2018	**Date:**	April 15, 2019
Contractor:	J-Con Civil Ltd. (J-Con)	**Project No.**	60549028 (601.7)
Owner:	The City of Winnipeg, Water & Waste Department	**Weather:**	

Present:	2 Crew members	-	J-Con
	2 Crew members	-	McDougall's Enterprise
	Brad Kreitz	-	AECOM

Purpose: Inspection of Leakage Test on Charleswood-Assiniboia FDM

Monday April 15, 2019:

- Marcel from City of Winnipeg Emergency Services confirmed that the secondary section of feeder main to the south of the south chamber had been successfully re-pressurized. Main line valve in south chamber still closed. (See Figure 1)
- J-Con and McDougall mobilized to Charleswood-Assiniboia FDM at 13:00 to conduct leakage testing on the feeder main.
- Pressure gauge attached to city hydrant at Southboine Dr and Berkley St to determine city pressure. (62 psi – See Photo 1)
- Hose connected to hydrant to fill McDougall's water tank. Trailer connected to crossing pipe via hose to 50 mm blow-off on the feeder main in the south chamber. At approximately 13:15 the pressure in the feeder main crossing was bled off to zero and observed to gradually rise back to approximately 62 psi over the course of approximately 1.5 hours. (See Photo 2)
- Results:
    - The south main line feeder main valve is bypassing slightly.
    - No apparent leaks on the feeder main crossing.

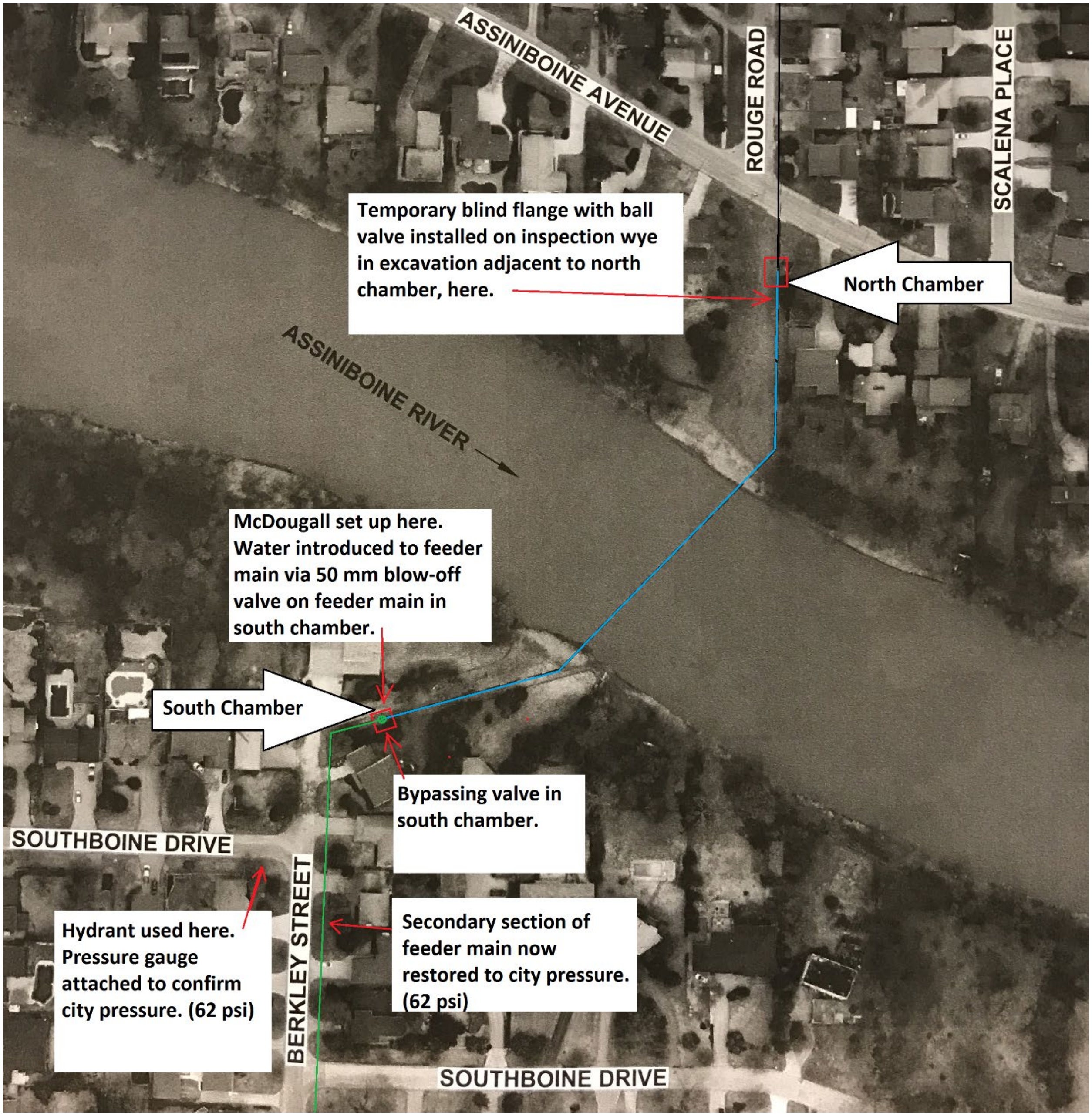


Figure 1 - Diagram of leakage test setup

Site Photos:

Photo 1 - Gauge measuring city pressure on hydrant at intersection of Southboine Dr and Berkley St

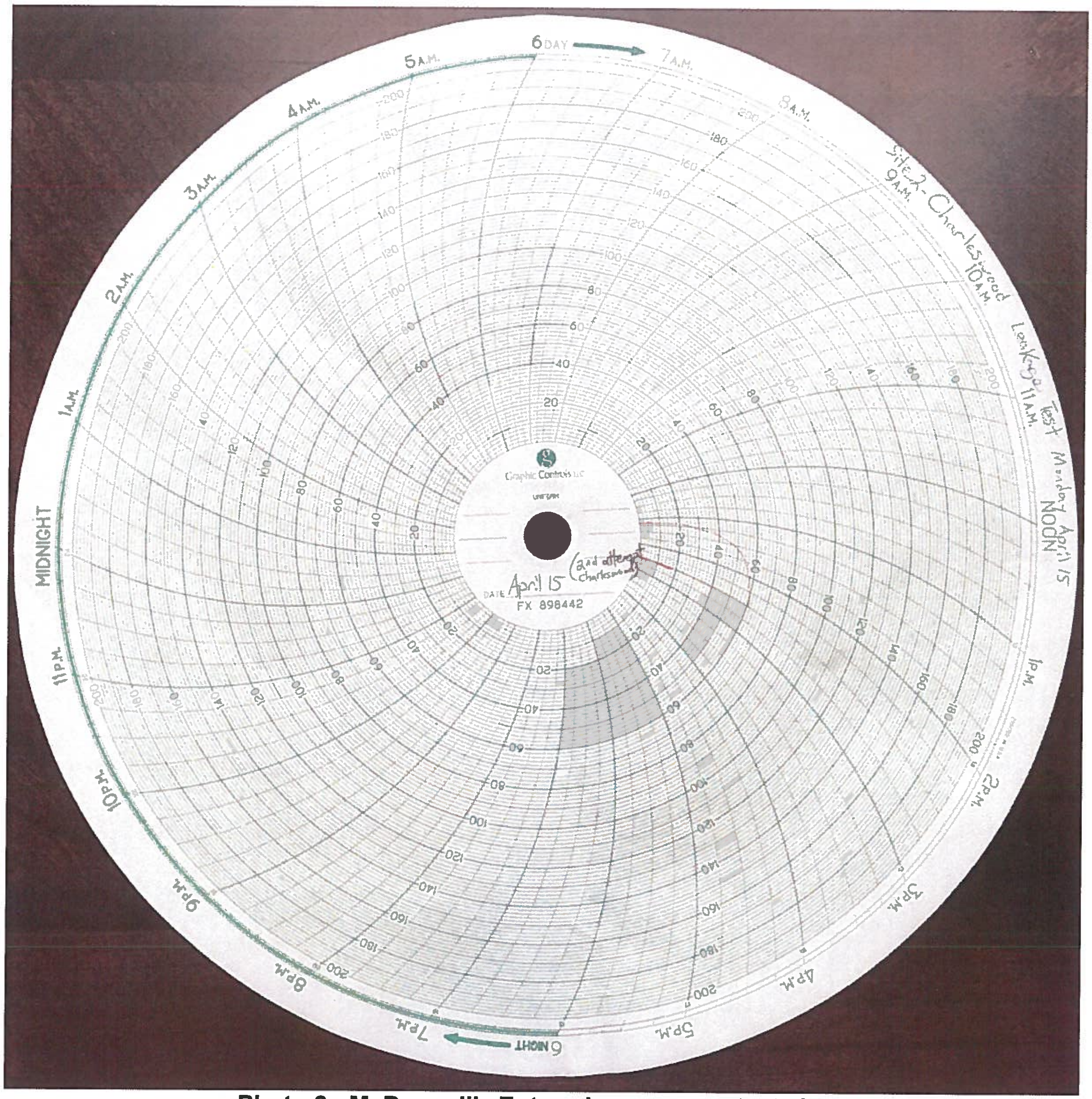


Photo 2 - McDougall's Enterprise pressure test chart

for Brad Kreitz
Municipal Technologist
Conveyance
BK/pab

cc: A. Delaurier, WWD (adelaurier1@winnipeg.ca)
J. Jones, J-Con (jjones@j-concivil.com)
M. McDonald, AECOM
N. Kehler, AECOM
A. Braun, AECOM

AECOM
99 Commerce Drive
Winnipeg, MB, Canada R3P 0Y7
www.aecom.com

204 477 5381 tel
204 284 2040 fax

Inspection Report

Project:	Provision of Pipeline Access Modifications, Cleaning and Support Services for River Crossing Inspections – Phase Two – Bid Opportunity No. 492-2018	**Date:**	April 17, 2019
Contractor:	J-Con Civil Ltd. (J-Con)	**Project No.**	60549028 (601.7)
Owner:	The City of Winnipeg, Water & Waste Department	**Weather:**	

Present:			
	5 Crew members	-	J-Con
	2 Crew members	-	McDougall's Enterprise
	3 Crew members	-	City of Winnipeg Emergency Services
	Brad Kreitz	-	AECOM

Purpose: Inspection of Leakage Test on Kildonan-Redwood FDM

Wednesday April 17, 2019:

- J-Con and McDougall's Enterprise mobilized to Kildonan-Redwood FDM to conduct second leakage test at 08:00. McDougall's Enterprise set up on the east side of the river just outside the valve chamber near the intersection of Hespeler Ave. and Glenwood Cres. (hereafter referred to as the *east valve chamber*) (See Figure 1)
- Water was introduced to the feeder main via the new 300 mm gate valve installed approximately 25 m east of the east valve chamber. Air was flushed from feeder main via McDougall's hose connected to the 75 mm blow-off valve on the top of the cross piece in the east chamber, as well as from the temporary blind flange with ball valve installed at the top of the vertical pipe in the west chamber. (See Figure 1, Photo 1,Photo 2 and Photo 3)
- At approximately 09:15 McDougall's pressure testing trailer was then connected to the hose attached to the 75 mm blow-off valve in the east chamber.
- Confirmed that J-Con had successfully fixed the small leak that was first observed in the west chamber at the interface between the blind flange and the ball valve during the first test.
- At 09:30 the feeder main was charged to city pressure (68 psi) by opening the new 300 mm gate valve. The 300 mm gate valve was then closed and the test began. By 09:45 Alex McDougall noted that the pressure had dropped to 44 psi and measured a leak of approximately 100 L over 15 minutes. (See Photo 4 & Photo 5)
- While the crossing was under city pressure J-Con personnel entered the tunnel in the west chamber to inspect for leaks visually and with the use of a sonoscope. This inspection was then repeated by Gerry Jones to ensure that nothing had been missed. No signs of leaks were detected in the tunnel, the west chamber, or the east chamber.
- At this point, with the known possible sources of error eliminated, it was determined that there was a leak somewhere between the west chamber and the new 300 mm gate valve at Hespeler Ave. and Glenwood Cres.
- City of Winnipeg personnel attempted to use an acoustic correlator to locate the leak but were missing vital calibration information and were unable to proceed. AECOM to provide information on crossing pipe properties to aid in correlator calibration. City personnel to return on April 18 to make a second attempt, J-Con to provide supporting services for confined space entry if necessary.

Thursday April 18, 2019:

- J-Con and City of Winnipeg Emergency Services returned to Kildonan-Redwood FDM to attempt to use acoustic correlator to locate the leak. The correlator was connected to the top of the vertical pipe in the west chamber and the top of the cross piece in the east chamber.

- The new 300 mm gate valve was fully opened and a pressure gauge was attached to the 75 mm blow-off valve on the top of the cross piece in the east chamber to confirm that the feeder main was at city pressure during the testing.
- The results of the first setup indicated a possible leak at approximately 10-20 m west of the east chamber but the results were not considered to be conclusive.
- The correlator was then set up to test the segment of 300 mm WM between the cross in the east chamber and the new 300 mm gate valve at Hespeler Ave. and Glenwood Cres. No indications of leaks were detected.

Results:

- Apparent leak of 400 L/hr based on leakage test.
- Visual inspection indicates that the leak is not present within the tunnel or tunnel shaft.
- Correlator inspection indicates leak is present on the feeder main crossing itself (between the valve chambers) and possibly 10 to 20 m west of the east valve chamber (towards the river).

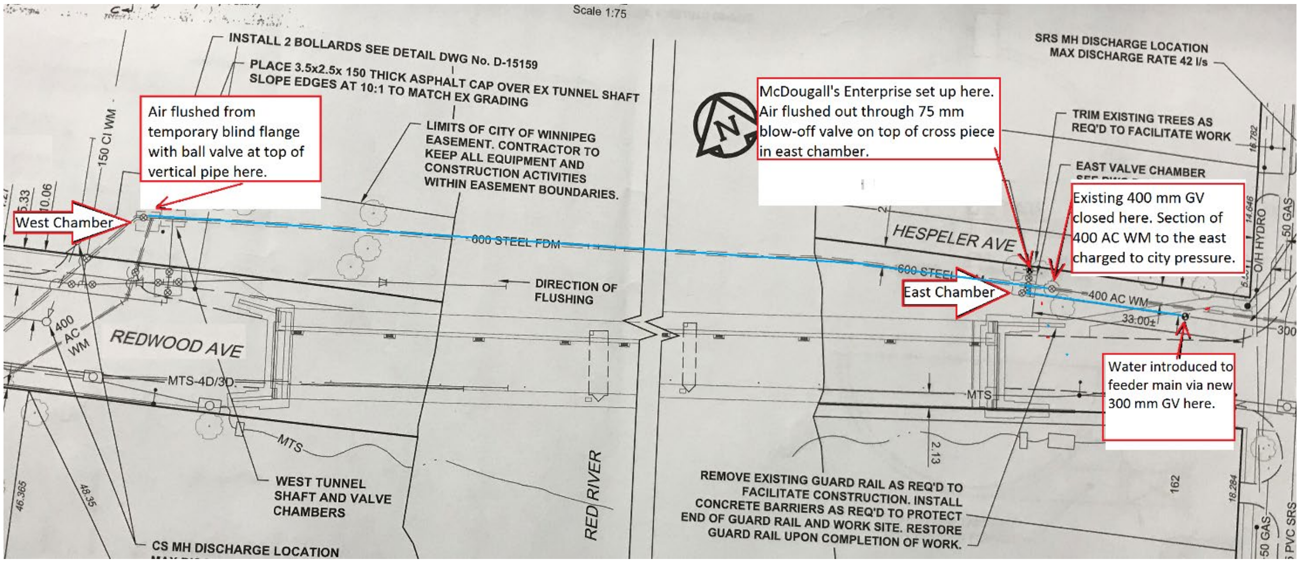


Figure 1 - Diagram of leakage test setup.

Site Photos:

Photo 1 - McDougall's hose connected to 75 mm blow-off in east chamber

Photo 2 - Water observed flowing from hose connected to 75 mm blow-off in east chamber after flushing air (prior to connecting to pressure testing trailer)

Photo 3 - Water observed flowing from blind flange with ball valve after flushing out air in west chamber

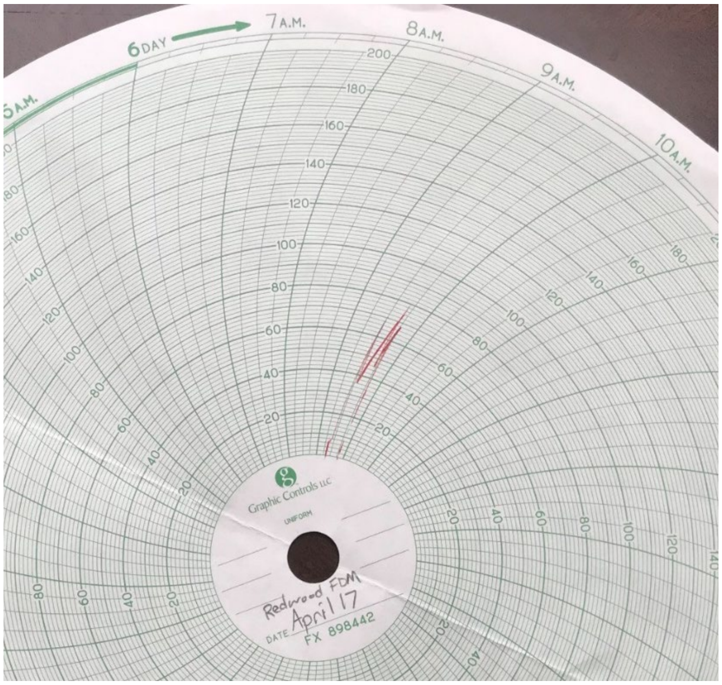
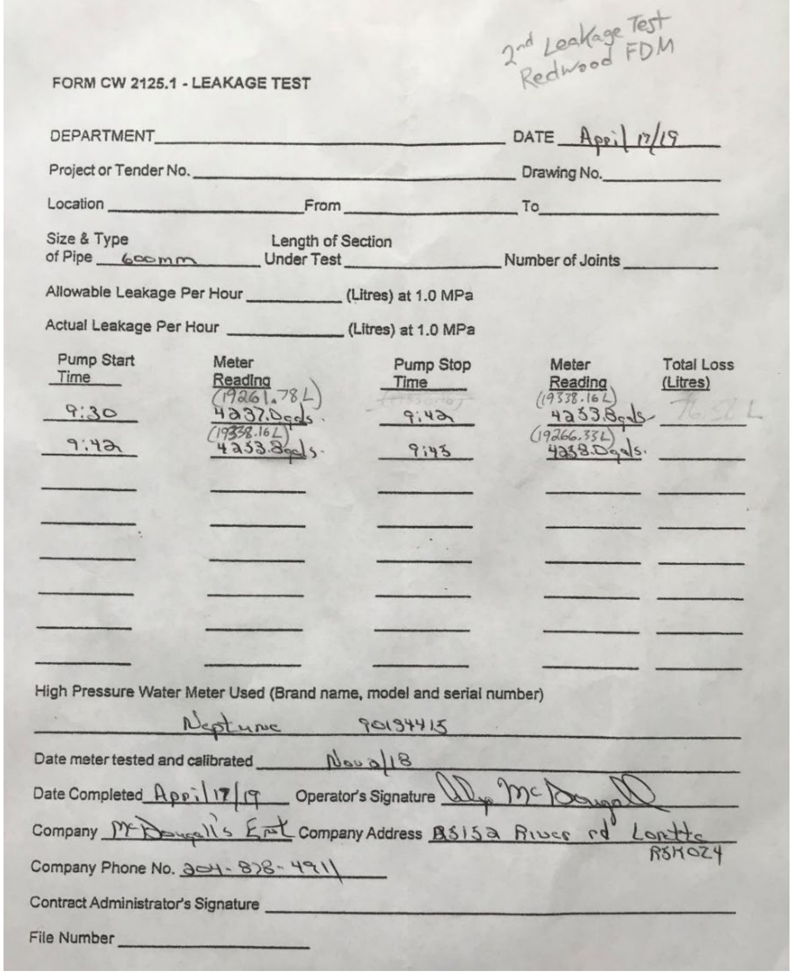


Photo 4 - McDougall's Enterprise pressure test chart



2nd Leakage Test
Redwood FDM

FORM CW 2125.1 - LEAKAGE TEST

DEPARTMENT ______ DATE April 17/19

Project or Tender No. ______ Drawing No. ______

Location ______ From ______ To ______

Size & Type of Pipe 600mm Length of Section Under Test ______ Number of Joints ______

Allowable Leakage Per Hour ______ (Litres) at 1.0 MPa

Actual Leakage Per Hour ______ (Litres) at 1.0 MPa

Pump Start Time	Meter Reading	Pump Stop Time	Meter Reading	Total Loss (Litres)
9:30	(19261.78L) 4237.0gals.	9:42	(19338.16L) 4253.8gals	76.38 L
9:42	(19338.16L) 4253.8gals.	9:43	(19266.33L) 4238.0gals.	

High Pressure Water Meter Used (Brand name, model and serial number)
Neptune 90194415

Date meter tested and calibrated Nov 2/18

Date Completed April 17/19 Operator's Signature McDougall

Company McDougall's Ent Company Address 85152 River rd Lorette R5K0Z4

Company Phone No. 204-878-4911

Contract Administrator's Signature ______

File Number ______

Photo 5 - McDougall's Enterprise leakage test report

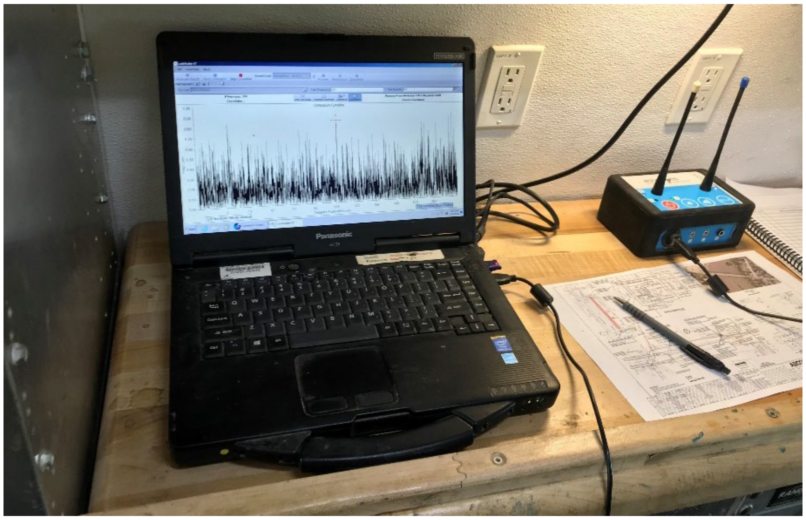


Photo 6 - Acoustic correlator read-out during first setup on April 18

Photo 7 - Acoustic correlator set up on 300 mm gate valve during second setup on April 18

Photo 8 - Pressure gauge to confirm city pressure on crossing pipe during correlator use

for Brad Kreitz
Municipal Technologist
Conveyance
BK/pab

cc: A. Delaurier, WWD (adelaurier1@winnipeg.ca)
J. Jones, J-Con (jjones@j-concivil.com)
M. McDonald, AECOM
N. Kehler, AECOM
A. Braun, AECOM

AECOM
99 Commerce Drive
Winnipeg, MB, Canada R3P 0Y7
www.aecom.com

204 477 5381 tel
204 284 2040 fax

Inspection Report

Project:	Provision of Pipeline Access Modifications, Cleaning and Support Services for River Crossing Inspections – Phase Two – Bid Opportunity No. 492-2018	**Date:**	May 21, 2019
Contractor:	J-Con Civil Ltd. (J-Con)	**Project No.**	60549028 (601.7)
Owner:	The City of Winnipeg, Water & Waste Department	**Weather:**	

Present:	5 Crew members	-	J-Con
	1 Crew members	-	McDougall's Enterprise
	2 Crew members	-	City of Winnipeg Emergency Services
	2 Crew Members	-	2 Vac Truck for Contingency Plan
	Brad Kreitz	-	AECOM

Purpose: Inspection of Leakage Test on Heritage Park Force Main

Tuesday, May 21, 2019 / Wednesday, May 22, 2019:

- 22:30 J-Con and two vac trucks mobilized to Heritage Park Force Main to conduct leakage test. Set-up 50 mm water pump to supply water for flushing from Sturgeon Creek. Connected to 50 mm port previously identified within pumping station. (See Photo 1)
- 23:15 J-Con realized that the inflatable plug on site did not match the one in the shop drawings, having a maximum back/test pressure of only 20 psi. (See Photo 2) Due to the time required to order a new plug it was decided to attempt to brace the plug securely against the opposite wall of the downstream manhole and to keep all personnel out of the manhole while the line was pressurized as a safety precaution.
- 23:55 City personnel turned off the station pumps and J-Con proceeded to install inflatable plug with 50 mm port in the downstream manhole at the intersection of Ness Ave. and School Rd.
- 00:25 J-Con commenced flushing.
- 00:30 McDougall's Enterprise arrived on site and began setting up leakage test apparatus in the pump station (pressure gauge, flow meter, backflow valve, leakage test graphing machine).
- 00:40 J-Con personnel at the downstream manhole reported no more air exiting the 25 mm port on the inflatable plug and concluded that the air had been successfully flushed out of the force main. J-Con then closed port on the inflatable plug and the 50 mm threaded port in the pumping station, disconnected the flushing hose, and connected McDougall's leakage testing apparatus to the same 50 mm threaded port. Water to charge the line to 50 psi (and make-up water) supplied through a tap within the pumping station.
- 00:45 McDougall's Enterprise began charging the force main up to 50 psi.
- 00:50 Force main charged to 50 psi, test started.
- City personnel continually monitoring levels in the upstream manhole (S-MH20000040) just outside the pumping station to ensure critical elevation of 231.37 m was not reached.
- 01:00 Pressure dropped to 40 psi. J-Con and McDougall inspecting valves and 50 mm port connection in pumping station for leaks. A small drip was found at the connection between the 50 mm port and the hose running to McDougall's test apparatus. This was quickly eliminated by additional tightening of the fitting. It was also observed at this time that three of the closed valves had a very small amount of water weeping up around the spindles. A stethoscope was used to sound the valves but J-Con and McDougall were both unable to hear any evidence of bypassing in any of the valves. Since no bypassing was heard using the stethoscope, City personnel recommended not attempting to reseat or tighten the valves further due to their unknown and possibly deteriorated condition (see Photo 3 and Photo 4).

- 01:05 J-Con directed their personnel at the downstream manhole at Ness Ave. and School Rd. to inspect the inflatable plug for leaks and/or movement. J-Con visually inspected and listened for leaks from the top of the manhole. No leaks or movement were observed throughout the duration of the test.
- 01:15 With sources of possible leakage within the station and downstream manhole checked/eliminated, the test continued with the pressure slowly dropping at a decreasing rate. The pressure was allowed to continue to drop for the remainder of the test rather than pumping it back up to 50 psi regularly so that the graph would clearly show how quickly the rate at which the pressure was dropping was decreasing.
- 01:50 One-hour test completed, pressure dropped to 16 psi.
- 02:05 McDougall pumped the line back-up to 50 psi and measured 17.5 L of total loss over one hour (see Photo 5 and Photo 6).
- 02:37 J-Con successfully removed inflatable plug from downstream manhole.
- 02:40 Both vac truck drivers left site. McDougall and J-Con began packing up equipment.
- 02:45 City personnel restored pumping station to service.

Site Photos:

Photo 1 - 50 mm threaded port for flushing / connecting leakage testing equipment

Photo 2 - Incorrect inflatable plug, rated for 20 psi, 25 mm port

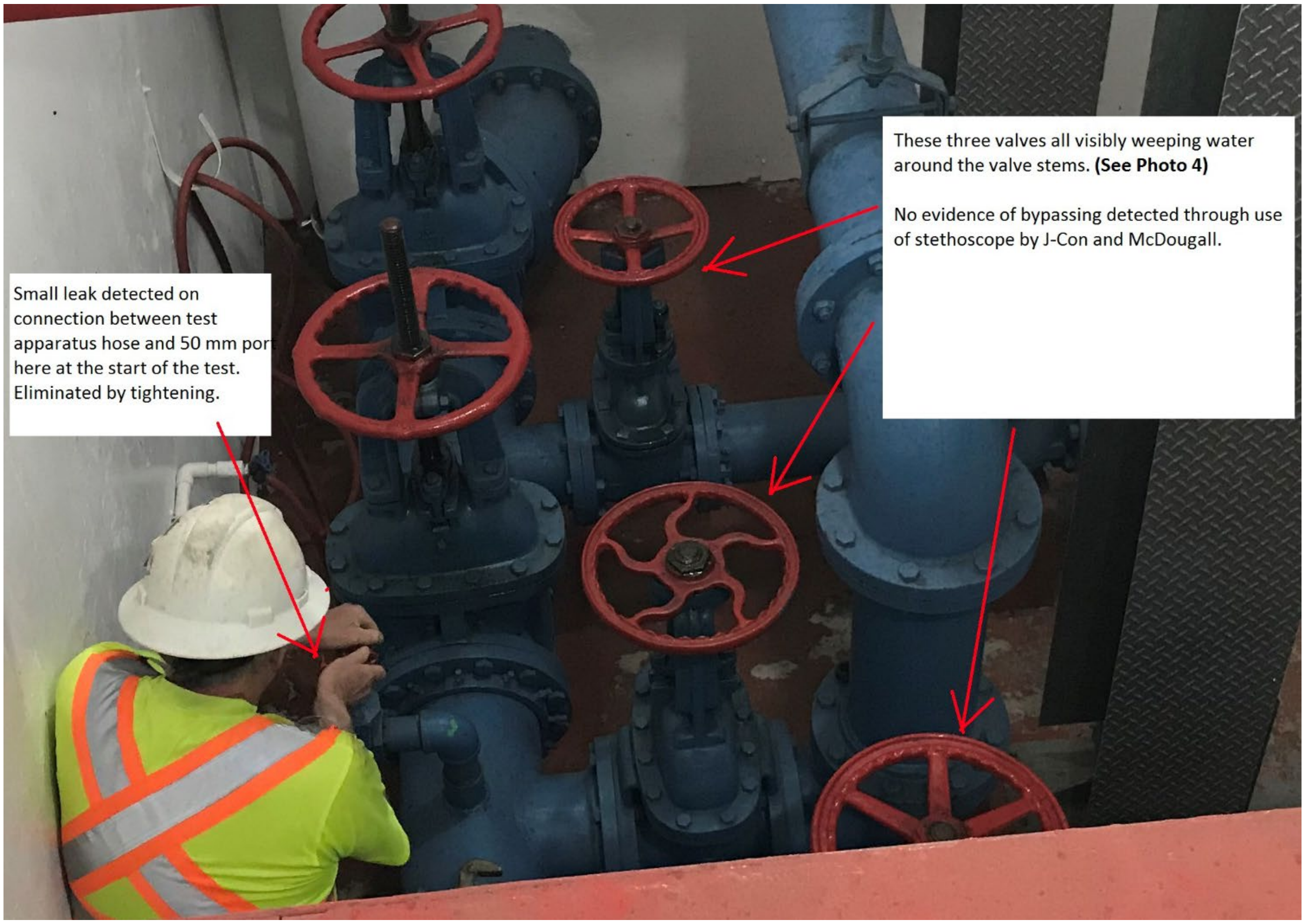
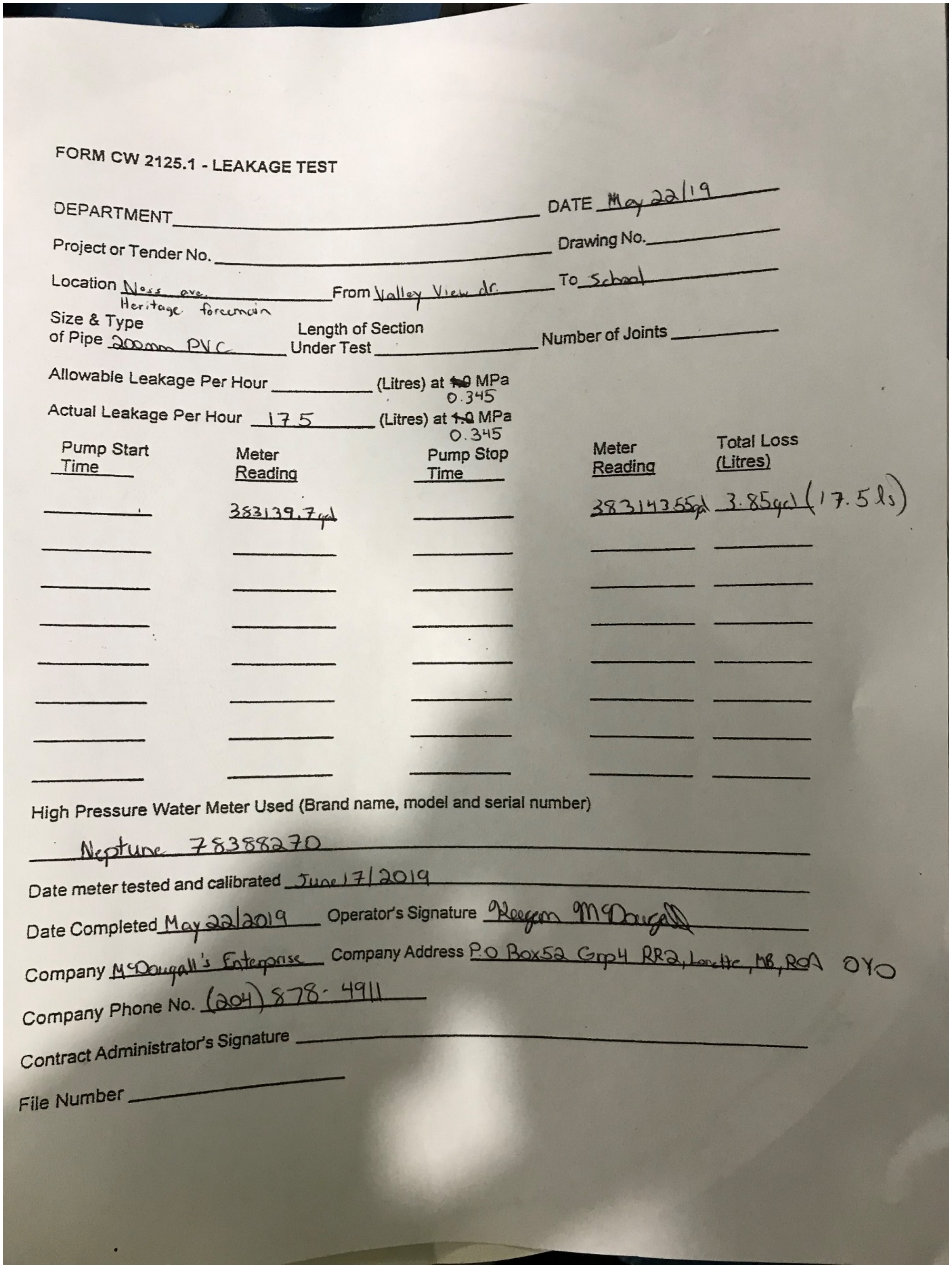


Photo 3 - Eliminating potential sources of leakage at the beginning of the test

Photo 4 - Slight weeping/leaking detected around valve stems of the three valves indicated in Photo 3.



FORM CW 2125.1 - LEAKAGE TEST

DEPARTMENT ______ DATE May 22/19

Project or Tender No. ______ Drawing No. ______

Location Ness ave. Heritage forcemain From Valley View dr. To School

Size & Type of Pipe 200mm PVC Length of Section Under Test ______ Number of Joints ______

Allowable Leakage Per Hour ______ (Litres) at ~~1.0~~ 0.345 MPa

Actual Leakage Per Hour 17.5 (Litres) at ~~1.0~~ 0.345 MPa

Pump Start Time	Meter Reading	Pump Stop Time	Meter Reading	Total Loss (Litres)
	383139.7gal		383143.55gal	3.85gal (17.5 ls)

High Pressure Water Meter Used (Brand name, model and serial number)

Neptune 78388270

Date meter tested and calibrated June 17/2019

Date Completed May 22/2019 Operator's Signature Keegan McDougall

Company McDougall's Enterprise Company Address P.O Box52 Grp4 RR2, Lorette, MB, R0A 0Y0

Company Phone No. (204) 878-4911

Contract Administrator's Signature ______

File Number ______

Photo 5 - McDougall's Enterprise leakage test report

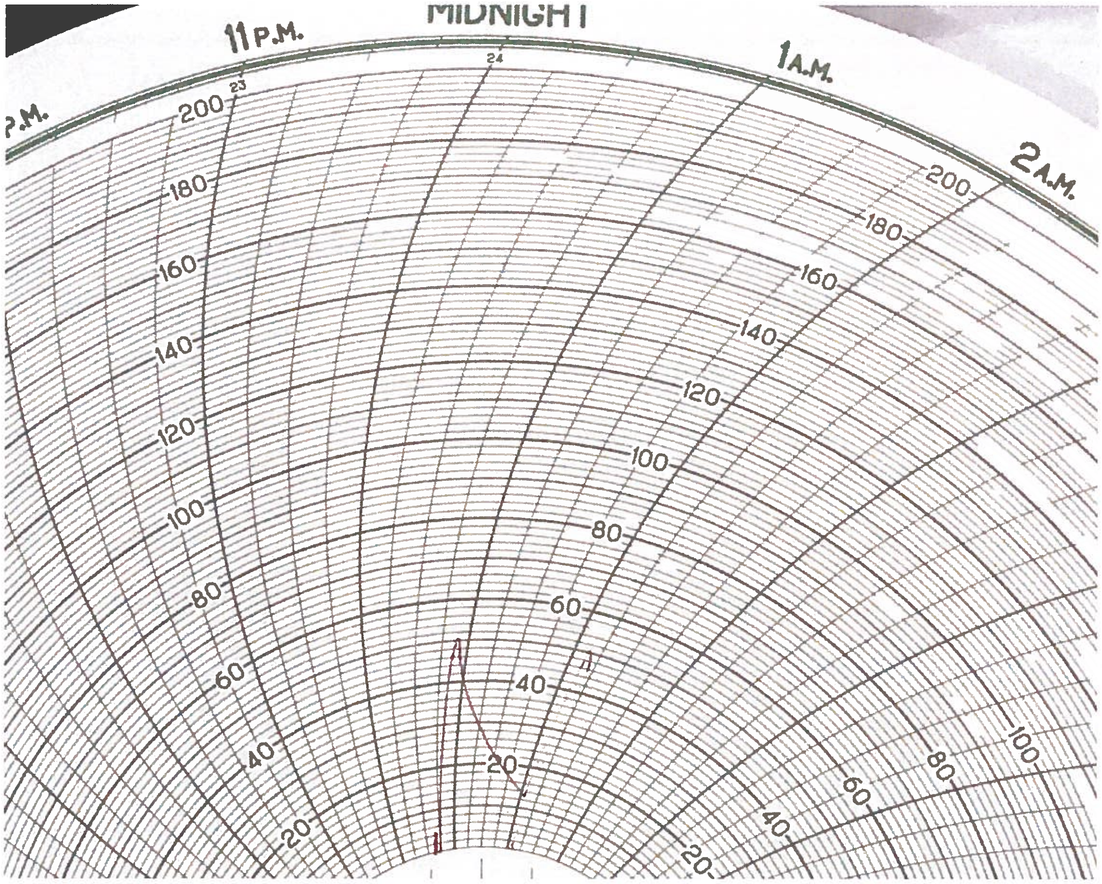


Photo 6 - McDougall's Enterprise leakage test graph

for

Brad Kreitz
Municipal Technologist
Conveyance
BK/pab

cc: A. Delaurier, WWD (adelaurier1@winnipeg.ca)
J. Jones, J-Con (jjones@j-concivil.com)
M. McDonald, AECOM
N. Kehler, AECOM
A. Braun, AECOM

AECOM
99 Commerce Drive
Winnipeg, MB, Canada R3P 0Y7
www.aecom.com

204 477 5381 tel
204 284 2040 fax

Inspection Report

Project:	Provision of Pipeline Access Modifications, Cleaning and Support Services for River Crossing Inspections – Phase Two – Bid Opportunity No. 492-2018	**Date:**	June 25, 2019
Contractor:	J-Con Civil Ltd. (J-Con)	**Project No.**	60549028 (601.7)
Owner:	The City of Winnipeg, Water & Waste Department	**Weather:**	Warm, Partly Cloudy

---

Present:

3 Crew Members	-	J-Con
3 Crew Members	-	City of Winnipeg
Tanner Beavis	-	AECOM

Purpose: Inspection of low pressure leakage test on Newton Avenue Force Main.

Tuesday, June 25, 2019

- 8:30 a.m. – City personnel confirmed that center gate valve between the two chambers at Fraser's Grove Park was open and closed the knife gate located on the HDPE force main.
- 8:45 a.m. – J-Con arrived at the downstream discharge manhole on Scotia Street and began installation of blind flange equipped with 2-inch port.

Figure 1 - Installation of blind flange with 2" port at downstream manhole.

- 9:42 a.m. – J-Con completed installation of blind flange and City personnel cracked the upstream knife gate to allow flow into the force main. Sewage was observed discharging from the 2-inch port on two separate occasions but quickly receded prior to City personnel closing the knife gate and J-Con being able to access the port (less than 5 seconds).
- 10:50 a.m. – A steady stream of sewage was observed discharging from the 2-inch port and City personnel closed the knife gate at the Fraser's Grove Park chamber. J-Con connected the 2-inch standpipe to the port and tied it off at the top of the discharge manhole. J-Con began filling the stand pipe

with water using a water truck and a half inch hose. The standpipe “burped” air as the water entered the pipe.

Figure 2 - J-Con filling the standpipe with water.

- 11:14 a.m. – The water level in the standpipe was observed to have stabilized approximately 9 inches above the rim of the manhole and marked using a felt pen.

Figure 3 - Mark made on the standpipe at the start of the leakage test.

- 11:15 a.m. – The leakage test was initiated.
- 11:45 a.m. – The water level in the standpipe was marked. It was measured that the water level had dropped approximately 4.13 cm since the start of the test.
- 12:00 p.m. – The water level in the standpipe was marked. It was measured that the water level had dropped an additional 1.53 cm for a total drop of 5.66 cm.
- 12:15 p.m. – The leakage test was concluded. The final level in the standpipe was marked. It was measured that the level had dropped by an additional 2.01 cm for a total drop of 7.67 cm.

Figure 4 - Measurement of markings on standpipe after removal.

- 12:17 p.m. – J-Con poured water into the standpipe using a measuring cup until the water level stabilized at the starting elevation. 5.05 L of water was required to bring the water level back to where it started prior to the 1-hour leakage test.

Figure 5 - Water container and measuring cup used to fill the standpipe after completion of leakage test.

- 1:11 p.m. – The standpipe and blind flange were removed from the downstream chamber and City personnel re-opened the knife gate located in Fraser's Grove Park.

Tanner Beavis, E.I.T.
Municipal Engineer in Training
Conveyance
TJB/pab

cc: A. Delaurier, WWD (adelaurier1@winnipeg.ca)
J. Jones, J-Con (jjones@j-concivil.com)
M. McDonald, AECOM
A. Braun, AECOM
B. Kreitz, AECOM
N. Kehler, AECOM

AECOM
99 Commerce Drive
Winnipeg, MB, Canada R3P 0Y7
www.aecom.com

204 477 5381 tel
204 284 2040 fax

Inspection Report

Project:	Provision of Pipeline Access Modifications, Cleaning and Support Services for River Crossing Inspections – Phase Two – Bid Opportunity No. 492-2018	**Date:**	June 10, 2019
Contractor:	J-Con Civil Ltd. (J-Con)	**Project No.**	60549028 (601.7)
Owner:	The City of Winnipeg, Water & Waste Department	**Weather:**	

Present:

Gerry Jones	-	J-Con
5 Crew members	-	J-Con
1 Crew member	-	McDougall's Enterprise
1 Crew member	-	Uni-Jet
3 Crew members	-	City of Winnipeg Emergency Services
Andrew McMillan	-	City of Winnipeg, Wastewater Operations
Adam Braun	-	AECOM

Purpose: Inspection of Leakage Test on Heritage Park Force Main

Monday, June 10, 2019 / Tuesday, June 11, 2019:

- 23:00 - J-Con and City on site:
  - Setting up traffic control
  - Prepping to install plug (See Figure 1)

- 23:15 - Uni-Jet on site

- 23:30 - Meeting on site with all parties

- 23:45 – Station drawn down and turned off
  - Plug installed at downstream MH (See Figure 2)

- 00:15 – Commencement of flushing
  - Force main flushed through 50 mm port on the discharge header within the station.

- 00:45 – Force main pressurized to 58 psi
  - Pressure dropping slowly, approximately 10 psi in 8 min
  - Within the station (refer to Figure 3):
    - Discharge header valve 1 closed.
    - Discharge header valve 2 open.
    - All other pump valves closed
  - Minor seepage past plug as witnessed by Uni-Jet. Seepage subsided after a few minutes.

- 1:15 – Plug moved under pressure.
  - Plug reset
  - Force main re-flushed
  - Plug pressurized to 70 psi

- 1-45 – Plug moved under pressure
  - Plug reset with reconfigured bracing. See discussion in issues section.
  - Force main re-flushed

- 2:05 – Force main pressured to 51 psi
    - Test started
    - Within the station (refer to Figure 3):
        - Discharge header valve 1 closed.
        - Discharge header valve 2 open.
        - All other pump valves closed.
        - 50 mm port opened with significant bypassing. Water collected in 20L pails, see Figure 4.
    - No visible seepage from test plug.

- 2:10 – Valve 2 closed.
    - Bypassing through 50 mm port reduced

- 2:20 – Valves within station sounded:
    - Valve 1 – Audible bypassing
    - First 6” pump valve – No Audible bypassing

- 3:15 – Pressure reduced to 15 psi
    - Bypassing from 50 mm port continued

- 3:35 – Test Ended (See Figure 5)

- 3:40 – Test plug removed and station valves reopened

- 3:50 – Station online

- 4:00 – Cleanup and demobilization

City crews monitored upstream wastewater levels throughout the test.

Results:

- Total makeup water measured by McDougall: 5.9 Imperial Gallons (26.82L)

- Total water measured within station: 25 to 26 L

Issues:

- Gerry Jones and J-Con were unaware of the modified testing procedure and were not prepared to induce pressure from the downstream plug location. J-Con managed to find appropriate fittings to test through the bleed port on the plug vs. the testing port as requested.

- McDougal did not bring full trailer setup as discussed with J-Con and thus did not have a water tank for testing. McDougal opted to utilize a hydrant for the water feed at the corner of Ness Ave and School Road.

- For the first two plug setup’s J-Con utilized an insufficient blocking configuration which allowed the plug to move. For the third attempt they employed blocking consistent with the first test which held.

Site Photos:

Figure 1: Setup at Discharge Manhole

Figure 2: Pressure Plug and Blocking

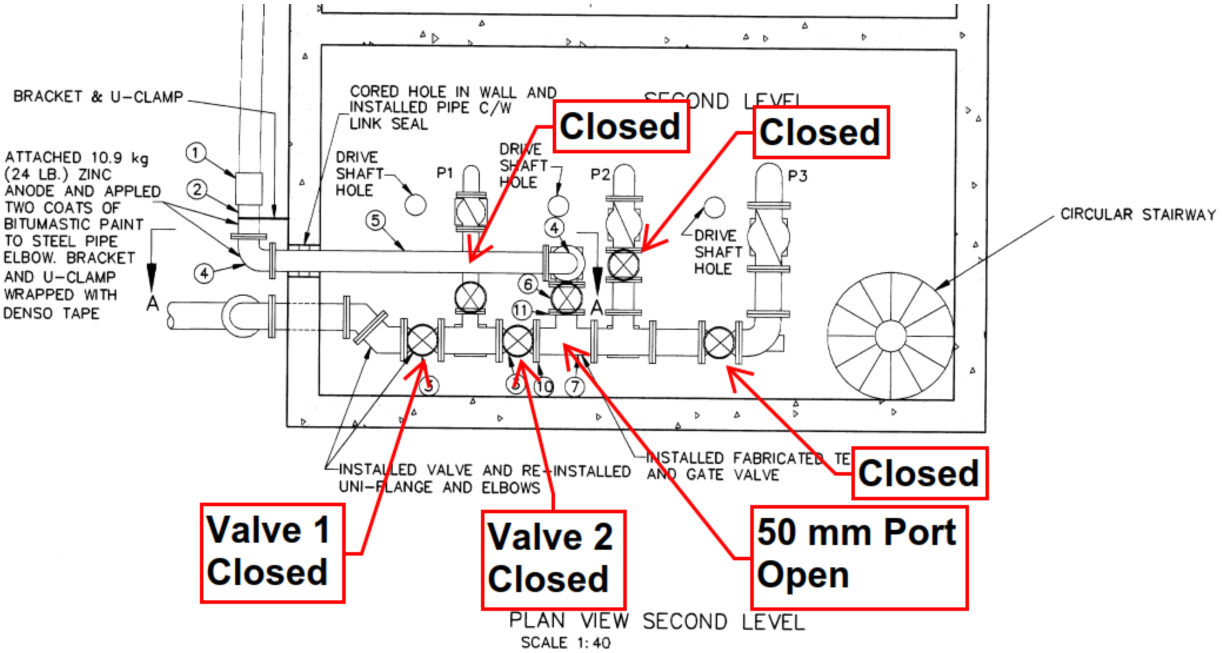


Figure 3: Valve Configuration in Station

Figure 4: Bypassing Measurement in Station

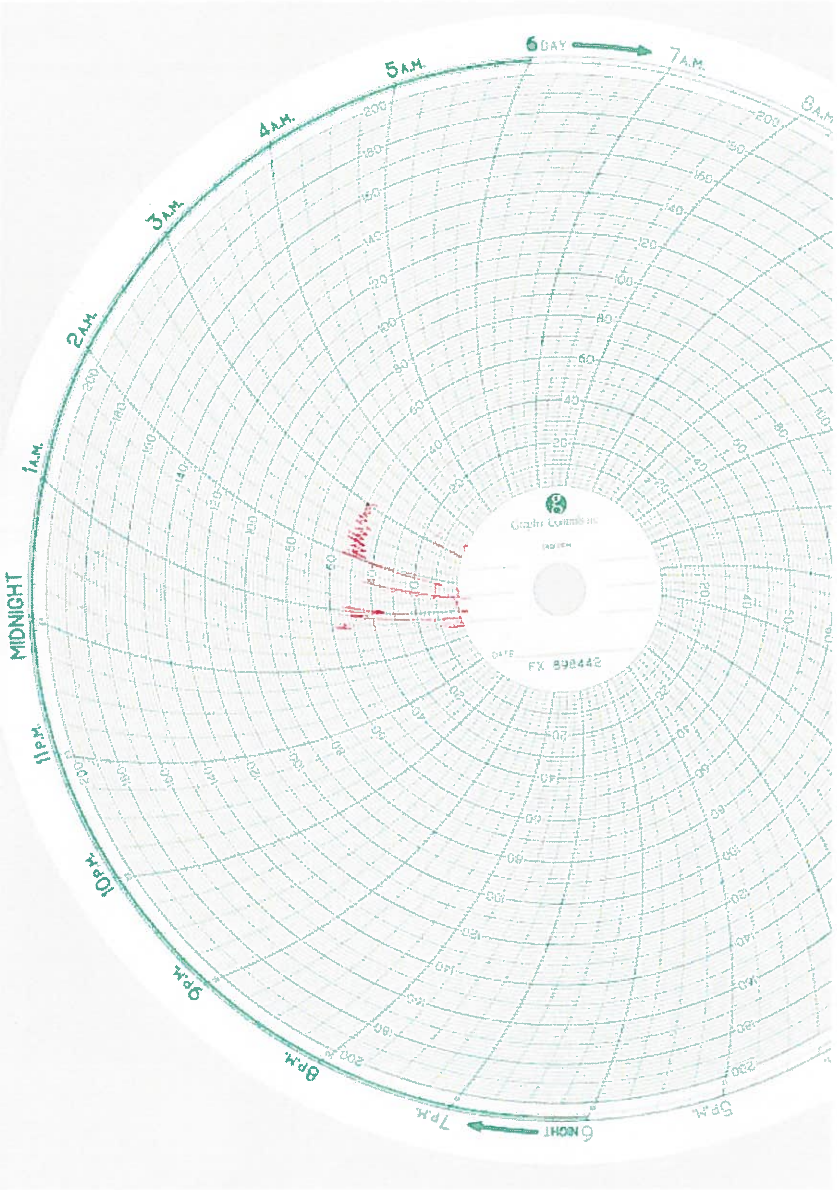


Figure 5: Test Record

Adam Braun
Municipal Engineer
Conveyance
ADB/gms

cc: A. Delaurier, WWD (adelaurier1@winnipeg.ca)
J. Jones, J-Con (jjones@j-concivil.com)
M. McDonald, AECOM
N. Kehler, AECOM
B. Kreitz, AECOM
T. Beavis, AECOM

AECOM
99 Commerce Drive
Winnipeg, MB, Canada R3P 0Y7
www.aecom.com

204 477 5381 tel
204 284 2040 fax

Inspection Report

Project:	Provision of Pipeline Access Modifications, Cleaning and Support Services for River Crossing Inspections – Phase Two – Bid Opportunity No. 492-2018	**Date:**	September 6, 2019
Contractor:	J-Con Civil Ltd. (J-Con)	**Project No.**	60549028 (601.7)
Owner:	The City of Winnipeg, Water & Waste Department	**Weather:**	Sunny

---

Present:	Jarrett Phillips and 2 crew members	-	J-Con
	Marcel Gervais and 2 crew members	-	City of Winnipeg Emergency Services
	Armand Delaurier	-	City of Winnipeg
	Zeljko Bodiroga and various staff	-	City of Winnipeg Water Services
	Adam Braun	-	AECOM
	Nathan Kehler	-	AECOM
	Brad Kreitz	-	AECOM

Purpose: Redwood Correlator Test to Locate Leak on Kildonan-Redwood Feeder Main (2nd attempt).

Friday, September 6, 2019:

- J-Con on site at approximately 8:30 a.m. to prepare for flushing.
- Water was introduced via the 300 mm gate valve approximately 25 m east of the east valve chamber.
- Water was allowed to exit the feeder main via a hose connected to the 150 mm port on the temporary blind flange at the top of the vertical standpipe in the west chamber and running directly to the river. Flushing reportedly began at 9:00 a.m. and continued until the city crew and AECOM staff arrived on site at approximately 9:45 a.m.
- At 10:10 a.m. it was discovered that J-Con had failed to flush the air from the system at the high point in the east chamber. Marcel of Emergency Services opened the 75 mm blow-off in the east chamber. A significant quantity or air was heard exiting the blow-off, which was left open for several minutes before the flow of air ceased and only water was observed. The blow-off was then closed. Flushing continued for several more minutes while waiting for the laptop for the correlator test to arrive.
- At 10:15 a.m. the required laptop arrived, and Jarrett Phillips of J-Con closed the port on the temporary blind flange in the west chamber while City of Winnipeg personnel finished setting up the correlator. The 300 mm gate valve near the east chamber (supplying the feeder main being tested) was left fully open for the duration of the test. Sensor wires were connected to the top of the vertical standpipe in the west chamber and to the 600 mm feeder main in the east chamber near the interface between the pipe and the west wall. The laptop on which the test results were being observed was set up on the walkway in the middle of the Harry Lazarenko Bridge.

Results:

- The results of the correlator test were inconclusive. No definite indication of a leak was found.
- After the test concluded it was decided that the leak should be confirmed once again. A pressure gauge was affixed to the 75 mm blow-off in the east chamber and at 10:55 a.m. the 300 mm gate valve near the east chamber was closed. Marcel monitored the pressure gauge and reported a starting pressure of 75-78 psi.
- By 10:57 a.m. the pressure had dropped to 70 psi.
- By 11:03 a.m. the pressure had dropped to 60 psi, then suddenly dropped from 60 psi to 40 psi in a matter of seconds. The sudden drop in pressure was accompanied by a loud gurgling sound in the east chamber.
- At 11:08 a.m. the pressure had dropped to 38 psi and the test was concluded.

Site Photos:

Photo 1: City of Winnipeg personnel preparing to connect correlator wires to feeder main in the east chamber

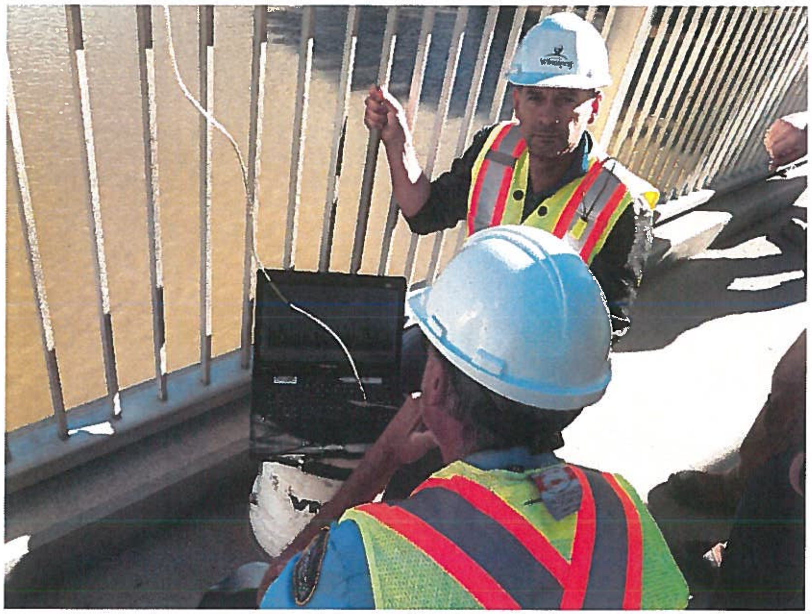


Photo 2: Observing results of correlator test on the Harry Lazarenko Bridge

For B. Kreitz
Municipal Technologist
Conveyance

cc: A. Delaurier, WWD (adelaurier1@winnipeg.ca)
J. Jones, J-Con (jjones@j-concivil.com)
M. McDonald, AECOM
N. Kehler, AECOM
T. Beavis, AECOM

Appendix J

Site 3 – UT Inspection Report

AECOM
99 Commerce Drive 204 477 5381 tel
Winnipeg, MB, Canada R3P 0Y7 204 284 2040 fax
www.aecom.com

Project:	**Baltimore Force Main - Bridge Pipe Inspection**	Date:	November 2, 2018
Contractor:	N/A	Project #:	60549028
Owner:	City of Winnipeg	Weather:	N/A

Inspection Report

On October 26, 2018, Marshall Gibbons of AECOM performed ultrasonic thickness testing on the steel pipe Baltimore Forcemain that is mounted beneath the St. Vital Bridge. The purpose of this work was to determine the remaining wall thickness of the force main at various points along the bridge and was undertaken in response to leak failures that occurred in August and October of this year. Access to the work sites was afforded by workers and scaffolding provided by J-Con.

The approximate locations of the test sites are shown on Figure A 1. These sites were initially selected by AECOM for electromagnetic (EM) thickness inspection by PICA under Bid Opportunity 495-2018. While the Civil Contractor for that project, J-Con, was preparing those sites for inspection (i.e. exposing the steel pipe by removing the metal cladding and foam insulation) a leak occurred due to substantial pipe wall thinning at Site 4.

To quickly gain an understanding of remaining wall thicknesses of the pipe across the bridge, measurements were taken primarily at the cardinal clock points around the pipe circumference, and at points along the invert. When it was discovered that the invert of the pipe was substantially thinner than other locations around the circumference, the circumferential extent of the thinning was investigated by testing across the invert at a test spacing of about 1 cm. Sites that measured less than one millimetre remaining thickness were further probed to determine minimum thickness at that site.

Testing commenced at the south end of the force main, first at the October 2018 leak site (Site 4) then near the force main drain (Site 5), and then proceeded to the north end of the force main. For record purposes, test sites were numbered increasing from north to south to be consistent with the bridge pier numbering, which increases in a southerly direction.

Testing at each site consisted of the following, and results are attached at the end of this report:

- Site 1: Circumferential bands on each side of the girth weld (weld 31), at the limit of factory-applied insulation 200mm south of the girth weld, and at the limits of insulation removal 0.4m north and 2.0m south of the weld.
- Site 2: Short strips across the across the invert on the north side of the girth weld (weld 30) and at 2.63m north of the weld at the limit of insulation removal.
- Site 3: Circumferential bands on each side of the girth weld (weld 29), a strip across the invert on the north side of the weld, and at points along the invert from 0.67m north of the weld to 0.60m south of the weld.
- Site 4: A strip across the invert on the north side of the leak repair coupling, at points along the invert south of the coupling, and circumferential bands at the following locations:
    - Immediately north and south of the repair coupling;
    - 0.83m south of the coupling, at the southern limit of insulation removal;

  - 60mm north of the coupling, near the limit of field-applied insulation;
  - 115mm north of the coupling, near the limit of factory-applied insulation;
  - 1.31m north of the coupling, at the northern limit of insulation removal.
- Site 5: Circumferential bands immediately north and south of the forcemain drain pipe, at points along the invert from 240mm south of the drain pipe to 190mm north of the drain pipe (at the north limit of insulation removal), and across the invert at the north limit of insulation removal.

Visual inspection and thickness testing revealed the following about the bridge pipe:

- At all test sites, wall thicknesses around the pipe circumference generally ranged between 7mm and 9mm except at the invert. Maximum wall thicknesses measured were 9.31mm and 9.07mm at Site 1.
- At the invert, pipe wall thickness generally decreased in a southerly direction. The highest invert thicknesses were measured at Site 1 and ranged between about 6mm and 7mm. At Sites 2 and 3, invert wall thickness ranged between about 2mm and 3mm, and at Sites 4 and 5 thicknesses were generally below 2mm.
- Lowest invert wall thicknesses were measured at Site 4 (0.87mm and 0.90mm).
- Cross-invert measurements revealed that substantial thinning occurs within about 20mm to 30mm each side of absolute invert (6:00 clock reference). During these inspections, particles could be heard rolling or bouncing along the invert of the pipe while the force main was in operation.
- Most of the exterior steel pipe surfaces at the girth weld areas, which were covered with field-applied insulation and cladding, exhibited only minor surface corrosion that would not contribute significantly to the invert perforation failures. External surfaces around the leak location at Site 4 and the drain connection at site 5 exhibited slightly more advanced corrosion.
- Outside pipe diameter was measured to be 510mm at Site 4.

According to the as-built drawings, the bridge pipe was installed in 1988-1989. The tender document specified the force main pipe to conform to the requirements of AWWA C200 and to have an outside diameter of 508mm and minimum wall thickness of 9.525mm. No protective coatings or linings appear to have been specified. While exclusion of an exterior coating does not appear to have compromised the pipe's condition due to the effectiveness of the factory applied insulation, the exclusion of a lining would be construed as a factor that could have severely compromised the design life of the pipe given that the fluid being transported was wastewater (a balance of wastewater collected in both dry weather and wet weather conditions).

While observations of very little exterior face corrosion are consistent with the manner in which the system was built (i.e. as a pre-insulated pipe) significant internal corrosion processes are present. As nearly all wall thickness measurements were less than 8.5mm it is high likely that a protective lining was not installed (if one was installed, it has completely failed). This suggests the observations of preferential invert corrosion in the pipe are likely widespread due to a combination of natural pitting and erosion corrosion processes. The severity of the pitting is also extreme and remediation plans should be rationalized for the pipe in the short term.

Marshall Gibbons, C.E.T.
Senior Municipal Technologist
Conveyance
MAG/CCM/pab
Encl.

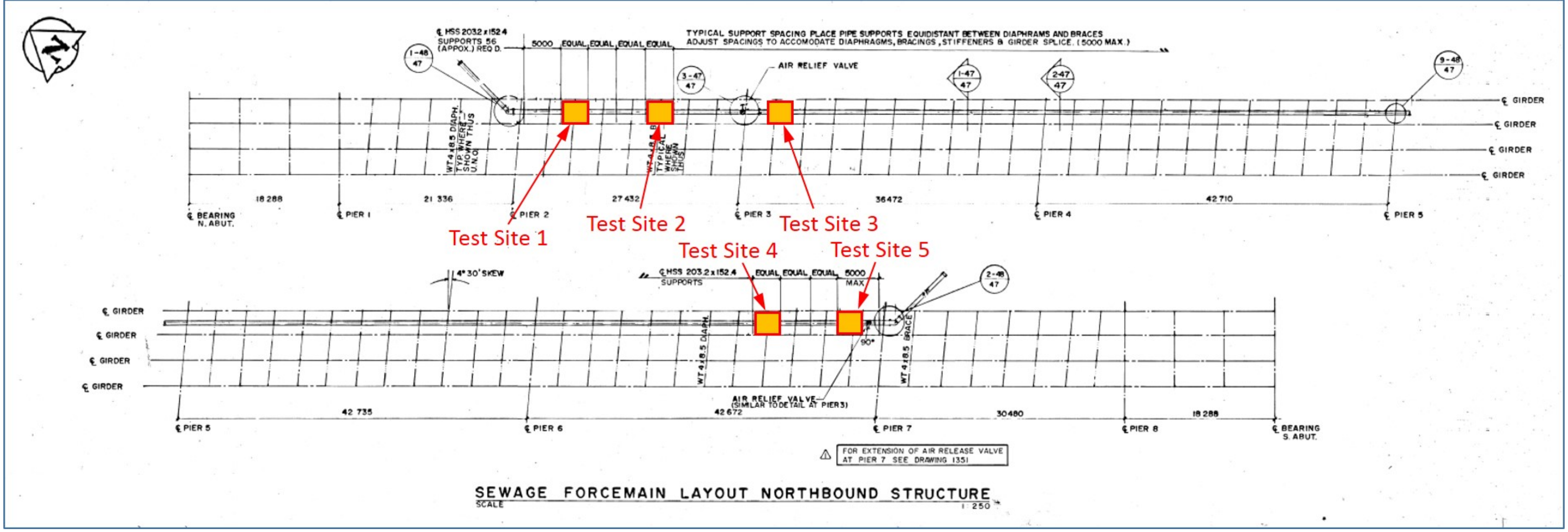


Figure A 1 – Approximate Locations of Ultrasonic Test Sites along Baltimore Force Main Bridge Pipe.

Figure A 2 - Leak Site in Baltimore Force Main, August 10, 2018.
Triangular hole through patch welded to the bottom of the pipe.

Figure A 3 – Leak Site in Baltimore Force Main, October, 2018.

Figure A 4 – Test Sites 1 and 2 between Piers 2 and 3 on north side of Red River.

Figure A 5 – Test Site 1 on north side of Red River.
Girth weld location on right (darker band of insulation near bridge drainage chute).

**Figure A 6 – Test Site 1 on north side of Red River.
Top of pipe showing minor surface corrosion at girth weld (X-ray 31)
and ultrasonic test points on each side of girth weld.**

**Figure A 7 – Test Site 2 on north side of Red River.
Girth weld location on left, at exposed insulation.**

**Figure A 8 – Test Site 2 on north side of Red River.
Girth weld location on left.**

Figure A 9 - Test Sites 2 and 3 near Pier 3 on north side of Red River.

Figure A 10 – Test Site 3 south of Pier 3 on north side of Red River.

**Figure A 11 – Test Site 3 south of Pier 3 on north side of Red River.
Top of pipe showing minor surface corrosion at girth weld (X-ray 29).**

Figure A 12 – Test Site 3 south of Pier 3 on north side of Red River. Bottom of pipe showing test locations at girth weld and on invert.

Figure A 13 - Test Sites 4 and 5 between Piers 6 and 7 on south side of Red River.

Figure A 14 - Test Site 4 at October 2018 leak site on south side of Red River.

Figure A 15 – Test Site 4 north of leak repair on south side of Red River. Note ultrasonic test points near repair clamp at left edge of photo.

Figure A 16 – Test Site 4 north of leak repair on south side of Red River. Note ultrasonic test points along invert and around pipe circumference.

Figure A 17 – Test Site 4 south of leak repair on south side of Red River. Note ultrasonic test points along invert and around pipe circumference.

Figure A 18 - Test Site 5 at Force Main Drain on south side of Red River. Note ultrasonic test points around pipe circumference.

Figure A 19 - Test Site 5 at Force Main Drain on south side of Red River. Note ultrasonic test points around pipe circumference.

Figure A 20 – Test Site 5: Thickness Test Points on invert and circumference. Note ultrasonic test points along invert and around pipe circumference.

Baltimore Force Main - St. Vital Bridge
Test Site 1 - Ultrasonic Thickness Testing

Gauge Velocity Setting = 5898 m/s, set using 7.5mm step on calibration block

Probe = FH2E-D, 7.5 MHz

All thickness measurements are in millimetres

Circumferencial Thickness Readings (clock ref. looking downstream)

Clock Point	0.0m Cladding N End	0.01m S	0.39m S N Side of Weld	0.43m S S Side of Weld	0.61m S Edge of Shop Insul.	2.40m S At Cladding	2.41m S Cladding S End
12:00		8.33	7.86	8.55	7.97	7.66	
1:00		8.02	8.11	8.42	7.54	7.97	
2:00		7.71	7.69	8.45	8.51	8.59	
3:00		7.79	7.55	8.07	8.55	8.18	
4:00		8.16	8.32	8.47	8.52	8.49	
5:00		8.03	8.17	8.24	8.49	8.28	
6:00		6.52	6.09	6.70	6.28	5.83	
7:00		7.10	7.31	8.16	8.02	7.88	
8:00		8.25	8.11	8.39	9.07	8.39	
9:00		7.81	8.26	8.24	9.31	8.36	
10:00		7.83	8.32	8.47	8.95	8.56	
11:00		8.12	8.26	8.64	7.91	8.28	

Baltimore Force Main - St. Vital Bridge
Test Site 2 - Ultrasonic Thickness Testing

Gauge Velocity Setting = 5949 m/s, set using 7.5mm step on calibration block

Probe = FH2E-D, 7.5 MHz

All thickness measurements are in millimetres

Thickness across invert at ~1 cm intervals, 2.36m N of girth weld at N Limit of insulation removal

East		Invert		West
5.37	2.80	3.07	5.11	6.85

Thickness across invert at ~1 cm intervals, N side of girth weld

East			Invert			West
6.32	5.27	4.27	2.06	2.88	4.60	6.08

Baltimore Force Main - St. Vital Bridge
Test Site 3 - Ultrasonic Thickness Testing

Gauge Velocity Setting = 5906 m/s, set using 7.5mm step on calibration block

Probe = FH2E-D, 7.5 MHz

All thickness measurements are in millimetres

Circumferencial Thickness Readings (clock ref. looking downstream)

Clock Point	N Side of Weld	S Side of Weld
12:00	8.14	8.27
1:00	8.18	8.70
2:00	8.16	8.08
3:00	8.46	8.36
4:00	8.09	8.46
5:00	8.43	8.37
6:00	2.35	2.52
7:00	8.47	8.61
8:00	8.07	8.26
9:00	8.20	8.28
10:00	8.18	8.22
11:00	8.51	8.45

Thickness across invert at ~1 cm intervals on N side of girth weld

East				Invert				West
7.49	5.76	3.92	2.04	1.62	1.73	2.82	4.80	6.48

Thickness along invert

0.68m N	0.43m N	0.14m N	0.0m	0.15m S	0.20m S	0.56m S
2.34	2.05	1.53	**Weld**	2.64	1.63	2.13

Baltimore Force Main - St. Vital Bridge
Test Site 4 - Ultrasonic Thickness Testing

Gauge Velocity noted below each data set

Probe = FH2E-D, 7.5 MHz

All thickness measurements are in millimetres

Circumferencial Thickness Readings (clock ref. looking downstream)

Clock Point	1.32m N Cladding	1.31m N	114mm N	65mm N	12mm N	N Edge of Clamp 0.0	Clamp 405mm wide	S Edge of Clamp 0.0	12mm S	0.30m S	0.61m S	0.83m S	0.84m S Cladding
12:00		8.17	8.75	7.61	7.86				8.12			7.99	
1:00		7.24	7.69	8.15	8.19				7.44			7.68	
2:00		8.44	7.84	8.28	8.13				7.84			8.28	
3:00		8.26	8.35	8.48	8.47				8.26			7.57	
4:00		7.95	8.54	8.16	8.19				8.29			7.98	
5:00		8.10	8.46	8.34	8.20				8.38			8.32	
6:00		0.87	1.60	1.40	1.01				1.67	1.81	1.65	1.84	
7:00		7.80	8.40	8.44	8.34				8.15			8.22	
8:00		7.69	8.50	8.42	8.59				8.18			8.32	
9:00		8.03	8.11	7.31	7.46				8.46			8.41	
10:00		7.81	8.81	7.43	7.53				8.29			8.25	
11:00		7.27	8.11	8.29	8.85				8.65			7.76	

Gauge Velocity Setting = 5909 m/s, set using 7.5mm step on calibration block

Gauge Velocity Setting = 5910 m/s, set using 7.5mm step on calibration block

Thickness across invert at ~1 cm intervals on N side of repair clamp

				Invert				
7.49	6.93	5.29	2.07	0.90	2.65	4.76	6.31	6.70

Gauge Velocity Setting = 5909 m/s, set using 7.5mm step on calibration block

Baltimore Force Main - St. Vital Bridge
Test Site 5 - Ultrasonic Thickness Testing

Gauge Velocity Setting = 5945 m/s, set using 7.5mm step on calibration block

Probe = FH2E-D, 7.5 MHz

All thickness measurements are in millimetres

Circumferencial Thickness Readings (clock ref. looking downstream)

Clock Point	N of Drain	S of Drain
12:00	7.68	8.19
1:00	7.34	8.11
2:00	8.15	7.03
3:00	8.11	8.09
4:00	8.20	8.23
5:00	8.24	8.19
6:00	1.86	1.74
7:00	8.07	8.12
8:00	8.37	8.27
9:00	8.30	8.26
10:00	8.19	6.95
11:00	7.07	8.65

Thickness across invert at ~1 cm intervals on N side of drain, at cladding

East				Invert				West
7.60	7.10	5.47	2.76	2.04	2.06	4.05	5.97	7.21

Thickness along invert at forcemain drain

200mm N	190mm N	135mm N	95mm N	65mm N	40mm N	0mm N	~2.5" dia	0mm S	45mm S	75mm S	115mm S	150mm S	190mm S	240mm S
Edge of Cladding	2.05	2.00	2.00	2.01	2.22	**N Edge of Drain Pipe**	**Drain Pipe**	**S Edge of Drain Pipe**	2.73	2.53	2.63	2.67	2.25	2.30

Appendix K

Pure Inspection Report

SAHARA®

INSPECTION REPORT

Charleswood Feeder Main and Fort Garry-St. Vital Feeder Main

Report Prepared for:

City of Winnipeg

By:

Pure Technologies, a Xylem brand

(November 23, 2018)

Sahara Inspection Report

Charleswood Feeder Main and Fort Garry-St. Vital Feeder Main

Prepared for

City of Winnipeg

Prepared by

Pure Technologies, a Xylem brand

November 23, 2018

Quality Assurance and Quality Control Statement

By my signature, I attest that this report has been prepared and reviewed in accordance with the Pure Technologies Quality Assurance and Quality Control procedures:

Victor Bernal, Project Manager Date: November 23, 2018

Ramzi Khalaf, Senior Program Manager Date: November 23, 2018

NOTICE

This report contains confidential commercial information regarding proprietary equipment, methods, and data analysis, which is the property of Pure Technologies It is for the sole use of the City of Winnipeg and its consultants.

DISCLAIMER

The information contained in this report is provided 'as is' without warranty of any kind, either express or implied. Pure Technologies is not liable for any lost profits, lost savings, or other incidental, special, or consequential damage arising out of the inspection results or the information contained in this report. Please refer to the terms and conditions attached to the Sahara Agreement and Pure Technologies' Technical Support Agreement for further details.

Contents

1. Executive Summary

The City of Winnipeg (CoW) retained the services of Pure Technologies, a Xylem brand. (Pure Technologies) to inspect one (1) section of the Charleswood Feeder Main (CFM), a 750-mm diameter potable water pipeline and one section of the Fort Garry St. Vital Feeder Main (FGFM), a 600-mm diameter potable water pipeline. Pure Technologies inspected the pipelines using the Sahara leak and air pocket detection platform.

The two (2) inspections took place from October 23 to October 25, 2018 at the CFM and FGFM for approximately 875m combined distance.

The purpose of the survey was to assess the condition of the feeder mains in two specific areas:

1. Identifying and accurately locating leaks and pockets of trapped air utilizing the Sahara system's acoustic capabilities.
2. Identifying any visual anomalies or other visual points of interest utilizing the Sahara system's Closed-Circuit Television (CCTV) capabilities.

In addition to this report, CCTV data collected in the inspection will be provided to the CoW in digital format.

Table 1.1 summarizes the Sahara inspection scope and results for the inspected sections of the Aqueduct.

Table 1.1: Summary of Sahara Inspection Results

Insertion	Pipeline	Survey Date	Start Location	End Location	Encoder Distance (m)	Leaks	Air Pockets	Visual Anomalies
1	FGFM	October 23-24, 2018	West Chamber	307m downstream	307	0	0	0
2	CFM	October 25, 2018	C-Pit	568m downstream	568	0	0	0
Total					**875**	**0**	**0**	**0**

2. Introduction

2.1 Project Background

The City of Winnipeg (CoW) retained the services of Pure Technologies, a Xylem brand. (Pure Technologies) to inspect one section of the Charleswood Feeder Main (CFM), a 750-mm diameter potable water pipeline and one section of the Fort Garry-St. Vital Feeder Main (FGFM), a 600-mm diameter potable water pipeline. Pure Technologies inspected the pipelines using the Sahara leak and air pocket detection platform.

The two (2) inspections took place from October 23 to October 25, 2018 at the CFM and FGFM for approximately 875m combined distance.

Figures 2.1 and 2.2 show the approximate locations of the sections inspected.

Figure 2.1: FGFM – West Chamber (307m)

Figure 2.2: CFM – C Pit (568m)

2.2 Description of Sahara Technology

The Sahara inspection platform is an acoustic-based, non-destructive condition assessment technology that detects acoustic activity associated with leaks[1] or pockets of trapped air, and potential structural defects via CCTV in pressurized water pipelines 12-inch (300-mm) in diameter and larger[2] of all construction types and materials. The Sahara inspection platform is composed of the following:

- a sensor with acoustic and video components (including LED lighting)
- a system for tracking the sensor from above ground
- an insertion assembly for inserting the sensor into a live pipeline
- a cable drum containing the communication umbilical for the sensor
- a rack of electronic instrumentation for the processing of acoustic and visual data

---

[1] The audible sound of a leak and the ability to detect leaks acoustically vary with pipeline pressure, leak volume, and leak shape. Sahara has detected leaks as small as 0.037 liters per minute (l/min) at pipeline pressures of 90 pounds per square inch (psi) or higher, and leaks as small as 0.37l/min at 25 psi. 15 psi is the minimum recommended pressure differential for acoustic leak detection.

[2] Down to 6-inch (150-mm) with special considerations.

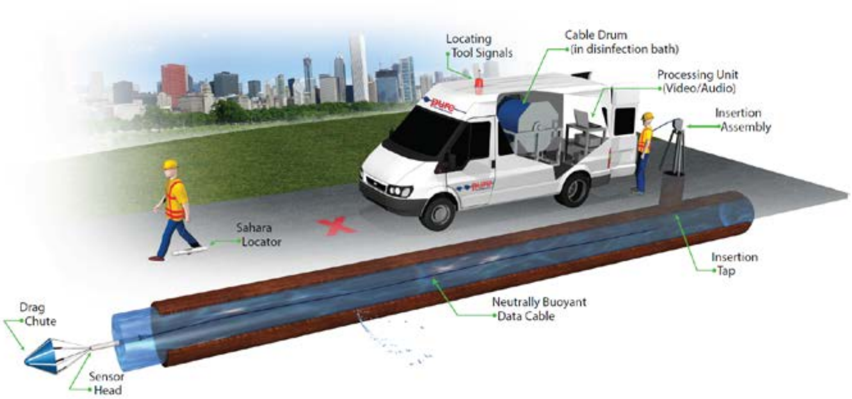


Figure 2.3: Sahara System Overview Diagram

The system is inserted into a live pipeline through a tap with a minimum diameter of 2-inch (50-mm). The sensor is propelled by the water flow using a drag chute that carries the tethered sensor head through the pipe for distances up to 1,800m per insertion as the data cable is unreeled from the cable drum[3] . Figure 2.3 depicts the typical Sahara system configuration.

2.3 Typical Insertion Requirements

Sahara requires a standard full bore 2-inch (50-mm) valve at each insertion point. Alternatively, for valves larger than 2-inch, a blind flange tapped with 2-inch national pipe thread (NPT)[4] female thread is required. Figure 2.4 shows the typical insertion set-up with a 2-inch ball valve.

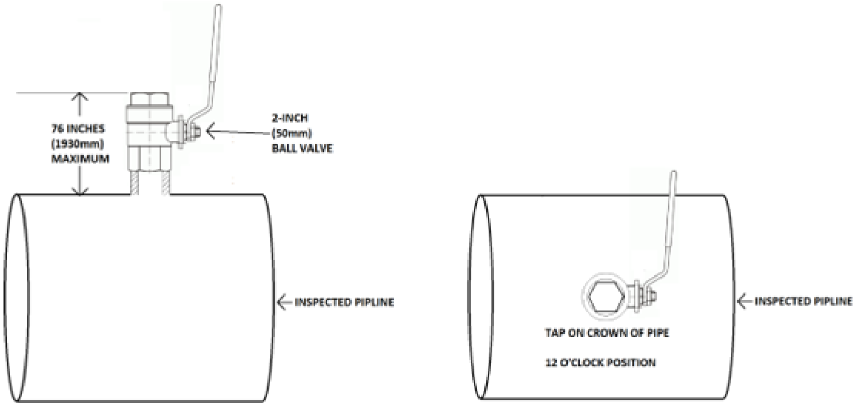


Figure 2.4: Typical Insertion Set-up: Profile and Plan View

---

[3] The distance achieved in each insertion is determined by pipeline material, flow velocity, and the cumulative degree of bends encountered.

[4] National Pipe Thread (NPT) Taper is a U.S. standard for tapered threads used on threaded pipes and fittings. In contrast to straight threads that are found on a bolt, a taper thread will pull tight and therefore make a fluid-tight seal.

2.4 Sahara Tracking and Location System

The Sahara sensor is tracked from above ground during the inspection at set intervals and at select points of interest. The sensor is tracked using the Sahara Locator®. The frequency used by the locator allows accurate through-pipe communication, even within metallic mains with a ground cover of up to 10 meters.

Accuracy of ground location is typically +/- 500 mm; however, location accuracy can be affected by the presence of large amounts of steel in or on the ground (such as railroad tracks, rebar, or unusually thick metallic pipe walls), steep slopes, or heavily wooded areas.

A technician follows the sensor head above ground, locating the sensor when requested by the Sahara operator—typically at a leak, air pocket, or other location of interest. The Sahara Locator can also be used to locate the sensor at set intervals to determine the alignment of a given section of pipeline.

2.5 Locating Leaks and Air Pockets

The audible sound of a leak and accordingly the ability to detect leaks acoustically vary with pipeline pressure, leak volume, pipe defect shape, and pipe surround/bedding material and configuration. Sahara has detected leaks with sizes as small as 0.037 liters per minute (l/min) at pipeline internal pressures of 90 pounds per square inch (psi) or higher, and leaks as small as 0.37 l/min at 25 psi. Although leaks have been detected in pipelines with pressures between 4-10 psi, 15 psi is the minimum recommended pressure differential for acoustic leak detection. At this pressure of 15 psi, leaks have been located in pipes of various materials from 400mm up to 2400mm in diameter.

The audible noise emitted by the leaks depends on different factors such as:

- Pipeline pressure – lower pressure puts less strain on the pipe wall and cause the pressure drop across the leak surface to have a lower audible noise
- Pipeline diameter – larger pipes tend to have thicker pipe walls that attenuate the noise traveling from the point of pressure drop to the sensor location inside the pipe
- Pipe material – the audible noise, travelling from the point of pressure drop to the sensor location inside the pipe, can be attenuated based on the type of material of the pipeline

In addition to the Sahara leak detection experience; leaks can be simulated on any pipeline as a means to confirm the technology's ability to detect leaks in the given pipeline under inspection. The leak simulation allows the operator to confirm what the "typical" leak sound is for the given pipeline conditions, establish what is the lower detection threshold for the given pipeline conditions, and to demonstrate technology capabilities.

The acoustic signal processor equipment and software provides the primary operator with the ability to monitor and analyze the data collected from the Sahara sensor in real time. This allows for real-time reporting of acoustic and visual events found in the pressure pipeline.

The acoustic signal processor software also converts the audio signal into visual form, displaying the signal amplitude, frequencies, head position, and velocity. The Sahara operator can isolate acoustic event locations, estimate leak magnitude qualitatively, and identify the limits of pockets of trapped air.

The precise location of an acoustic event is identified by positioning the sensor within the pipe and simultaneously positioning the Sahara Locator directly above the sensor head, allowing for above ground location.

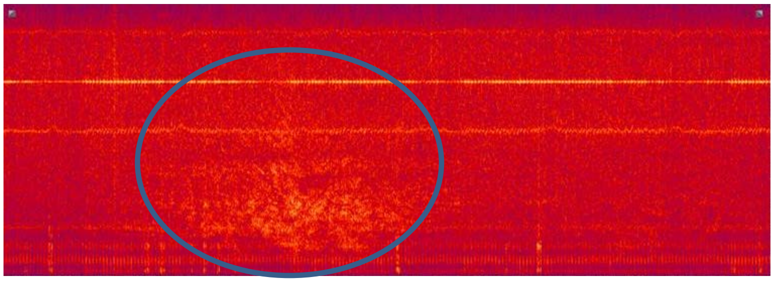


Figure 2.5: Sahara leak spectral

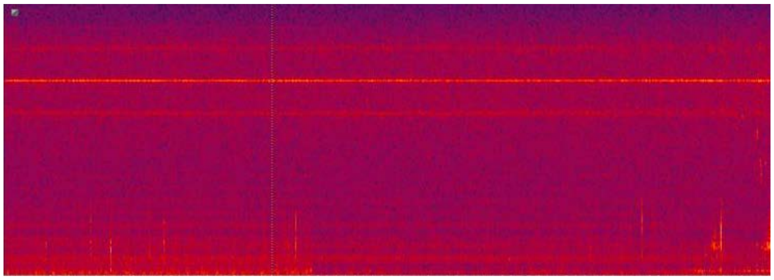


Figure 2.6: Sahara no leak spectral

2.6 Closed-Circuit Televising (CCTV)

In addition, the operator can distinguish pipeline features or other points of interest using the Sahara platform’s closed-circuit televising (CCTV) capability. As with leaks, the Sahara operator can indicate above ground the position of visual points of interest by positioning the sensor within the pipe and simultaneously positioning the Sahara Locator directly above the sensor head. Clarity of the video can be affected negatively by high turbidity, turbulent flow, surface condition of the pipe wall, and when inspecting pipes over 48-inch (1200-mm) in diameter.

3. Sahara Leak Detection and CCTV Results

3.1 Sahara Inspection Results Summary

The Sahara inspections were completed from October 23 to October 25, 2018. Table 3.1 displays the inspections results. Zero (0) leaks and zero (0) air pockets were identified during the inspections of the FGFM and the CFM; with zero (0) visual defects located above ground for further investigation or repair. Sections 3.3 and 3.4 detail the inspected sections.

Table 3.1: Summary of Sahara Inspection Results

Insertion	Pipeline	Survey Date	Start Location	End Location	Encoder Distance (m)	Leaks	Air Pockets	Visual Anomalies
1	FGFM	October 23-24, 2018	West Chamber	307m downstream	307	0	0	0
2	CFM	October 25, 2018	C-Pit	568m downstream	568	0	0	0
Total					**875**	**0**	**0**	**0**

Figure 3.1: Insertion 1 Start Location (West Chamber)

Figure 3.2: Insertion 2 Start Location (C-Pit)

3.2 FGFM Inspection 1 – October 23-24, 2018

The 600-mm diameter FGFM section between West Chamber and East Chamber was inspected on October 23-24, 2018. Figure 3.3 displays an aerial view of the FGFM Inspection 1 with the inspection scope highlighted in blue. A total inspection distance of 307m was completed. There were zero (0) leaks and zero (0) air pockets detected during the inspection. Table 3.2 provides details on the Inspection 1.

Figure 3.3: Aerial View of FGFM Inspection 1

Table 3.2: Aqueduct Sahara Inspection 1 Details	
Pipeline	FGFM
Diameter	600-mm
Material	Cast Iron
Insertion Date	October 23-24, 2018
Insertion Location	ARV 116
Insertion Feature	4-inch valve with 2-inch NPT port
Inspected Distance	307m
Measured Flow	0.45m/s approximate
Measured Pressure	3.10 bar (at insertion point)
Survey Observations	0 visual defects located. No leaks or air pockets were detected.

3.3 CFM Inspection 2 – October 25, 2018

The 750-mm diameter CFM section was inspected on October 25, 2018. Figure 3.4 displays an aerial view of the CFM Inspection 2 with the inspection scope highlighted in blue. A total inspection distance of 568m was completed. There were zero (0) leaks and zero (0) air pockets detected during the inspection. Table 3.3 provides details on the Inspection 2.

Figure 3.4: Aerial View of CFM Inspection 2

Table 3.3: Aqueduct Sahara Inspection 2 Details	
Pipeline	CFM
Diameter	750-mm
Material	AC
Insertion Date	October 25, 2018
Insertion Location	C-Pit
Insertion Feature	2-inch Valve
Inspected Distance	568m
Measured Flow	0.6m/s approximate
Measured Pressure	4.82 bar (At insertion point)
Survey Observations	0 visual defects located. No leaks or air pockets were detected.

4. Summary

Pure Technologies' evaluation of the FGFM and the CFM summary:

- The 600-mm FGFM was inspected for a total distance of 307m. The inspected section contained no leaks or air pockets at the time of the inspection.
- The 750-mm CFM was inspected for a total distance of 568m. The inspected section contained no leaks or air pockets at the time of the inspection.
- No visual defects were located during the inspections. Video files will be delivered to CoW.

Appendix L

Material Testing Reports

PSILab
Testing • Forensics • Consulting

117 South Sunset St., Suite I
Longmont, CO 80501
(720) 204-1529
info@psilab.net
www.psilab.net

PROJECT REPORT

REPORT NUMBER: AEC-011619-1-RP1

DATE: March 4, 2019

CLIENT INFORMATION

AECOM
99 Commerce Drive
Winnipeg, Manitoba R3P 0Y7 Canada
Attention: Adam Braun
AECOM Project No.: 60549028 (100)

SAMPLE DESCRIPTION

One (1) PVC pipe sample was submitted by the client. The sample was approximately 20 inches in length and was received on 2/1/2019. The sample was identified by the client as follows:

1. Sample #1: Heritage Park Force Main

A digital image of the sample follows in Figure 1.

Figure 1. Sample #1: Heritage Park Force Main

TEST PERFORMED

ASTM D1784 Cell Classification Testing:

The sample was tested in general accordance with the test methods and requirements of ASTM D1784-11. Specifically, tensile and impact test specimens were removed from the sample and tested.

The testing performed and reported herein did not include chemical identification of the compound, DTUL testing, nor flammability testing.

1. Izod Impact testing was conducted in accordance with ASTM D256-10, Method A. Notches and impact direction were oriented perpendicular to the profile extrusion direction. Specimens 0.5 inches by 0.125 inches were tested.

2. Tensile Properties testing was conducted in accordance with ASTM D638-14. Testing was conducted using modified Type IV specimens oriented along the profile extrusion direction at a test speed of 0.2 inches/minute. Full wall thickness test specimens 0.25 inches in width were tested.

All test specimens were conditioned in accordance with ASTM D618-13, Procedure A (minimum of 40 hours at 23C/50% RH) prior to testing.

Additional Tests:

Dimensions:

The sample was tested per ASTM D2122-16, *Standard Test Method for Determining Dimensions of Thermoplastic Pipe and Fittings*. Specifically, the sample was tested for average outside diameter and minimum wall thickness.

Heat Reversion:

The sample was tested per ASTM F1057-17, *Standard Practice for Estimating the Quality of Extruded Poly(Vinyl Chloride) (PVC) Pipe by the Heat Reversion Technique*. The test sample was exposed in an air oven at 180±5°C for 30 minutes after recovery.

Acetone Immersion:

The sample was tested per ASTM D2152-17, *Standard Test Method for Adequacy of Fusion of Extruded Poly(Vinyl Chloride) (PVC) Pipe and Molded Fittings by Acetone Immersion*. The test sample was exposed to anhydrous acetone for 20 minutes.

Loss on Ignition (LOI):

The sample was tested per ASTM D2584-18, *Standard Test Method for Ignition Loss of Cured Reinforced Resins*. Triplicate test specimens were removed from the sample and tested. Test specimens approximately 5g were tested. Each specimen was placed in a 565°C muffle furnace for greater than 8 hours duration. This test determined the residue upon ashing of the as-produced PVC compound.

Thermal Analysis by Differential Scanning Calorimetry (DSC):
The sample was tested in general accordance with ISO 18373-1, *Rigid PVC pipes -- Differential scanning calorimetry (DSC) method -- Part 1: Measurement of the processing temperature* and ISO 18373-2, *Rigid PVC pipes -- Differential scanning calorimetry (DSC) method -- Part 2: Measurement of the enthalpy of fusion of crystallites*. Modifications to the test method included performing the testing using a ramp rate of 25°C/min, a purge gas flow rate of 50ml/min, and an end temperature of approximately 232°C. Duplicate test specimens were removed from the sample, specifically from the inner, middle, and outer wall locations. The maximum processing temperature Tp (i.e. the "Interpeak Melt") was determined for each specimen, as well as enthalpies of the "A" and "B" segments of the DSC thermograms. Test specimens were obtained using a micro-saw and scalpel.

RESULTS

The synopsis test results for all testing excepting the Thermal Analysis are summarized in Table 1 below.

Thermal Analysis test results are tabulated in Table 2.

Post-test digital images of the heat reversion test samples are displayed in Figures 2 - 4

DISCUSSION:

Upon performance of the heat reversion test the interior surface of Sample #1 suffered from significant blistering and/or fish scaling.

Report Written by:	**Report Reviewed by:**
Steve Ferry Laboratory Director	Steve Lam President

	Sample ID
Properties	Sample 1
Address	Heritage Park
IZOD Impact Resistance (ft-lb/in)	1.270
Tensile Strength @ Yield (psi)	7,168
Tensile Modulus (psi)	446,070
Color	Blue
Outside Diameter Ave. (in)	11.110
Minimum Wall Thickness (in)	0.6216
Heat Reversion	FAIL
Acetone Immersion	<1% / No Reaction
Ash Content (%)	6.04%

Table 1. Synopsis test results.

Sample 1	Mass (mg)	T sub g	Melt Onset	A Endotherm	Interpeak Melt	B Endotherm	% Gelation	End Temp
Inner-1	11.1	85.47	103.70	2.860	186.69	2.362	54.8%	232.66
Inner-2	10.9	85.43	103.43	2.647	186.24	2.282	53.7%	232.66
Inner Ave.		85.45	103.57	2.754	186.47	2.322	**54.2%**	232.66
Mid-1	11.3	85.42	97.83	3.260	184.61	2.158	60.2%	232.69
Mid-2	11.2	85.77	98.36	2.887	184.78	2.216	56.6%	232.71
Mid Ave.		85.60	98.10	3.074	184.70	2.187	**58.4%**	232.70
Outer-1	12.7	86.18	109.21	5.142	189.99	1.390	78.7%	232.62
Outer-2	11.9	85.84	108.83	5.890	195.34	0.9112	86.6%	232.66
Outer Ave.		86.01	109.02	5.516	192.67	1.151	**82.7%**	232.64

Table 2. Thermal Analysis test results for Sample 1. All temperatures are in degrees Celsius (°C), and Endotherms are in J/g.

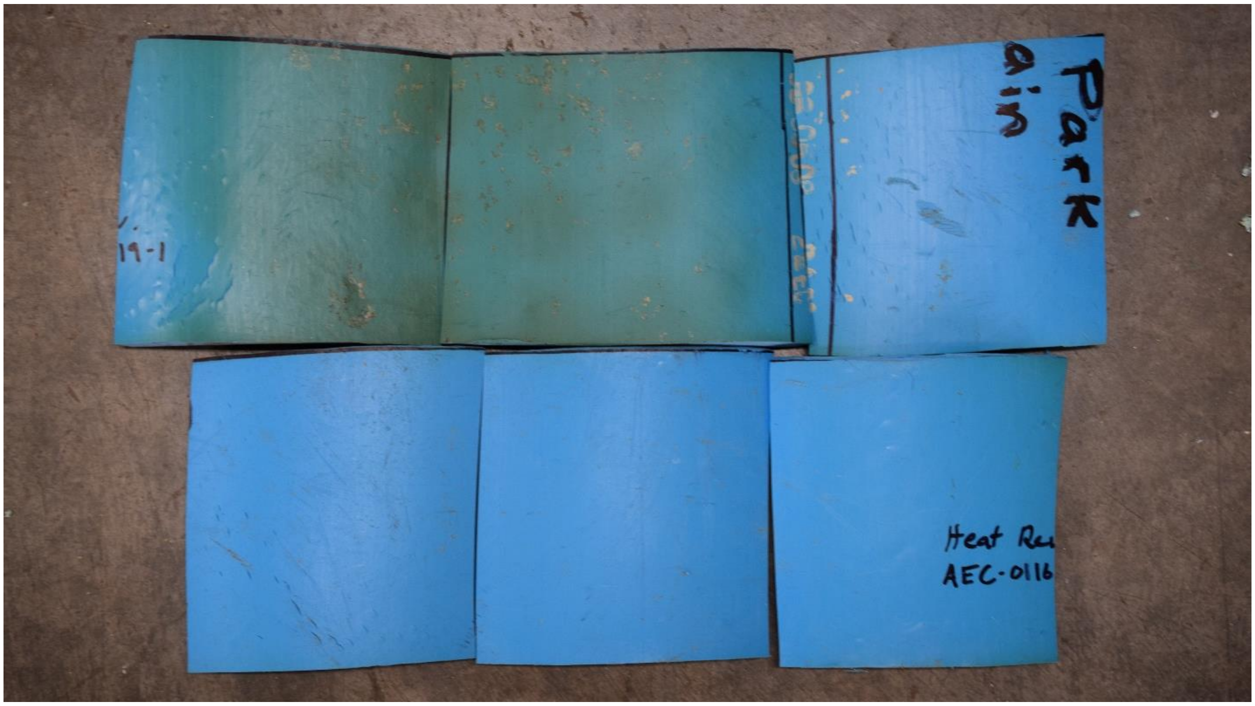


Figure 2. Post-test heat reversion test specimen from Sample #1: Note sun-bleached surface of the pipe which turns brown during heat reversion testing.

Figure 3. Post-test heat reversion test specimen from Sample #1: Note significant blistering and/or fish scaling.

Figure 4. Post-test heat reversion test specimen from Sample #1: Note significant wall separation.

AECOM
99 Commerce Drive
Winnipeg, MB, Canada R3P 0Y7
www.aecom.com

204 477 5381 tel
204 284 2040 fax

Technical Memorandum

To Armand Delaurier, C.E.T.

CC Marv McDonald, C.E.T.

Subject **HRRC: HDPE Testing Update - Newton Force Main and Fort Garry-St. Vital Interceptor**

From Adam Braun, P. Eng., Chris Macey, P. Eng.

Date July 14, 2017

Project Number 60270487 (500)

Subsequent to issuing of the High Risk River Crossings Condition Assessment Report – Sewer Crossings in September of 2016, HDPE samples from the Newton Force Main and Fort Garry-St. Vital (FGSV) interceptor were sent for testing at the NSF Canada labs in Aurora, Ontario to characterize the material cell classification and slow crack growth potential. This technical memorandum discusses the findings of the testing, effects on the condition assessment completed by AECOM during the program, and recommendations for proceeding with the upcoming condition assessment of the Newton Force Main crossing.

Test Results

Below is a summary of the testing results reported by NSF. A copy of the final testing report has been attached in Appendix A.

	FGSV Interceptor (Sample 16-349)	Newton Force Main (Sample 16-350)
Cell Classification	PE3255XX	PE3354XX
Density (g/cm^3)	0.947	0.945
Melt Index (g/10 min)	0.601	0.293
Flexural Modulus (MPa)	874	850
Tensile (Yield) Strength (MPa)	24.3	22.3
Slow Crack Growth Resistance (PENT)	Brittle Failure at 0:20 and 1:23	Brittle Failure at 0:21 and 1:14

Both pipelines failed under the slow crack growth resistance testing (PENT tests) well before the minimum 10 hours of testing listed in ASTM D3350 for cell classification. This indicates that both pipelines are sensitive to long term stresses imparted by either manufacturing (internal) flaws or external flaws (e.g. improper installation).

It is believed that both pipelines were supplied by the same manufacturer (DuPont). However, both samples have radically different melt index values. Variation in melt indexes can be an indication of poor extrusion consistency and quality control processes.

Structural Analysis under HRRC Program

AECOM's structural analysis for the 350 mm HDPE Newton Force Main (Site 3b) assumed a PE2XXX series resin and associated material properties for flexural and tensile strength and flexural modulus. Based on the results of the testing program, the constituent HDPE resins are consistent with PE3XXX series resins which exhibit higher densities and flexural properties. Thus, the structural checks completed which identified factors of safety ranging from 2.94 to 3.47 against external hydrostatic pressure and soil loading respectively, are conservative in nature and do not require revisiting.

Significance of Testing Results

The lifetime of a HDPE pipe is controlled by material, environment, and loading factors.

The cell classifications of each material were determined to be a true High Density PE as opposed to low or medium density material which is a positive aspect of the testing. The testing yielded mechanical properties of PE3255 and PE3354 for the FGSV interceptor and Newton Force Main, respectively. These resin classifications are typical of many pre-1980 resins (see Figure 1) which typically exhibit much poorer resistance to slow crack growth than common pressure pipe resins manufactured post 1980. The newer (post 1980) PE3408 resin is known to have a high enough resistance to slow crack growth (SCG) such that SCG does not control design life under the vast majority of circumstances.

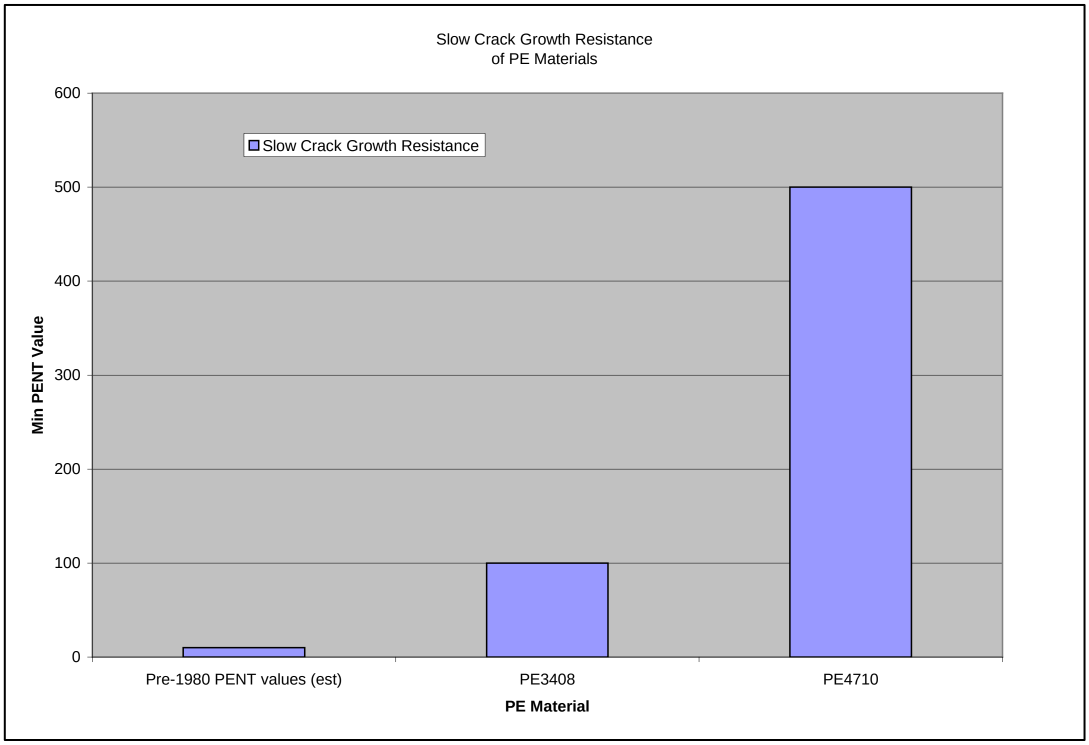


Figure 1: Typical Slow Crack Growth Resistance by Resin Type

The primary environmental exposure concern for HDPE pipelines is exposure to extreme oxidizing agents (e.g. chlorine) that can break down inferior HDPE resins. The exposure conditions for both the FGSV Interceptor and the Newton Force Main are generally considered to be relatively benign to HDPE, as municipal waste streams are not commonly rich in HDPE oxidizing agents.

Polyethylene (PE) materials have three modes of failure, depending on the stress level and chemical resistance evident in the material as shown in Figure 2. Stage I failures are ductile in nature, and only occur at very high stress levels. Based on the structural analysis carried out and known mechanical properties of the pipes, the stress levels in the pipe walls are very low. Neither pipe is considered to be at risk to a Stage I failure mode.

Stage II failures, however, are brittle types of fractures and can occur at moderate to low stress levels. Stage II failures are associated with slow crack growth (SCG) and the resistance of the pipe to SCG can usually be assessed in a PENT test. PENT test values lower than 10 hours are indicative of the material with a much higher risk factor to incur brittle fractures over time at increasing lower stress levels.

Stage III failures occur as a result of chemical degradation of the material, and the steep curve associated with Stage III failures indicates that the material no longer has the capacity to withstand any load at all. Based on the resin classifications used and the exposure conditions for the pipes, they are very unlikely to experience Stage III failure modes.

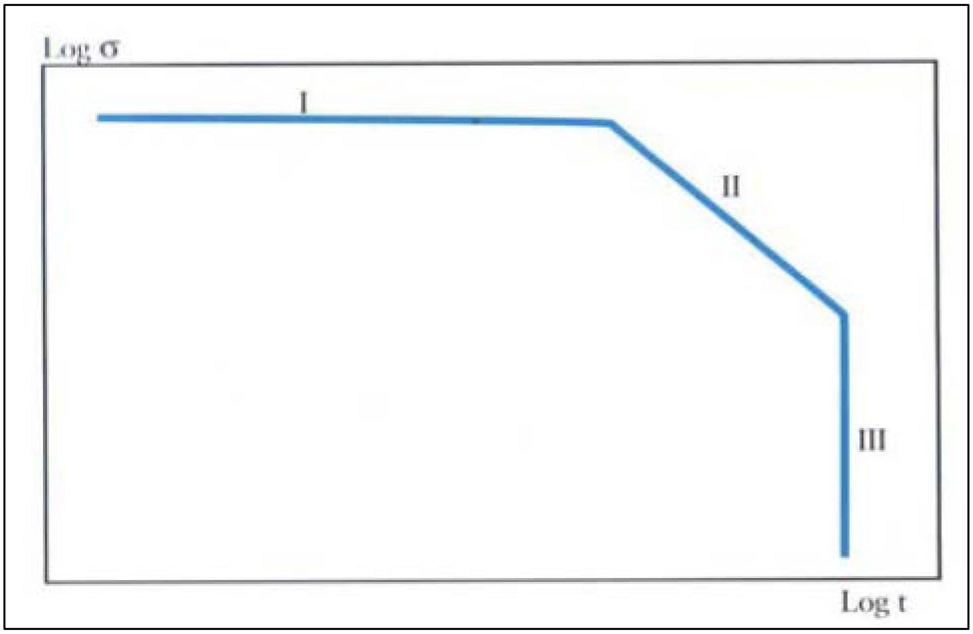


Figure 2: Relationship between Stress Level and Time to Failure for HDPE

Both samples exhibit very low PENT values in testing which is an indication that:

- The pipe materials will become increasing less ductile over time, and
- The pipes will continue to have increasing risk profiles for brittle failures.

The elevated internal pressures associated with the Newton Force Main and application of cyclic loads from pumping put it at higher risk than the FGSV Interceptor.

Recommendations

The poor resistance to slow crack growth raises concerns over the long term viability of both river crossings. The internal pressure profile and nature of the loading on the Newton Force Main places it at a higher risk than the FGSV Interceptor crossing.

The extremely varied installation and operating conditions typical to river crossings can result in locations of higher than normal combined stresses in the hoop and longitudinal directions. Coupled with poor bedding and backfill methods utilized in the early 1970's for HDPE installations, these stress concentrations could lead to failure of the pipe.

Experience with the City's HDPE force main and gravity sewer river crossings indicates they have a finite lifespan. Examples include, failure of the Crescent Drive Interceptor and damage to the FGSV Interceptor. The buckling failure found on the FGSV interceptor siphon via sonar inspection was not truly indicative of SCG alone and likely resulted from original construction practices, despite the presence of both invert and obvert cracking, an indication of SCG related deterioration. When exposed, the bedding for the HDPE pipe at this location was found to be a mixture of sand bedding and native material containing a high degree of foreign material including rock and large tree branches. The presence of foreign material within the pipe bedding causes stress concentrations which exacerbate slow crack growth over time. Based on the performance history of early 1970's HDPE river crossings in the City, it is recommended that replacement or trenchless rehabilitation be considered in the medium term.

The upcoming High Risk River Crossing, Phase Two program includes a sonar inspection of the HDPE Newton Force Main. While eventual replacement/rehabilitation of this crossing is inevitable, inspection of the crossing using sonar will allow us to assess the structural state of the crossing and risk level for failure due to slow crack growth. Risk factors for slow crack growth would include excessive deformation, reverse curvature, etc.; all conditions that result in high material stresses. Sonar inspection in some instances can also confirm the presence of cracks and active leakage.

In addition to the sonar inspection (if it does not provide clear evidence of leakage) we would recommend a leakage test be completed in conjunction with the sonar inspection to confirm the hydrostatic integrity of the crossing. The leakage test may be able to be carried out with the aid of low head pressure test by filling up the line with water less than its current operating pressure and monitoring any change in water levels or with the City's own LeakFinderRT platform using the testing protocol previous developed by AECOM and Echologics under the original wastewater river crossing screening program.

Given the age of both the Newton Crossing and the FGSV Interceptor and their low resistance to slow crack growth, some consideration will need to be given to establish a more definitive timeline for replacement of both crossings. Assuming current stress levels are low and a number of years of remaining useful service may be assumed, however, their ultimate useful life will be finite and they both could require replacement/rehabilitation in the next 5-10 year period.

The outcome of the upcoming inspection program will dictate replacement/rehabilitation of the Newton Force Main crossing. Should inspection of the line under the upcoming inspection program find the pipe to be intact, a continued leak detection program can be initiated to look for signs of failure. Due to the modifications completed during the 2014 HRRC inspection program, the City now has the ability to isolate the Newton Force Main should the line fail, allowing time for replacement or rehabilitation work to be completed.

We trust this information meets your requirements on this matter. Should you have any queries or require further information or clarification, please do not hesitate to contact Adam Braun of this office.

Sincerely,
AECOM Canada Ltd.

Adam Braun, P. Eng.
Municipal Engineer
Conveyance
ADB/CCM/pab

FOR ✓
Chris Macey, P. Eng.
Americas Technical Practice Leader
Condition Assessment and Rehabilitation

PROVINCE OF MANITOBA
A.D. BRAUN
Member
24453
REGISTERED PROFESSIONAL ENGINEER

Appendix A

PROJECT 16-3307
FINAL REPORT

Cell Classification Testing on Two PE Samples

January 26, 2017

Executive Summary

The purpose of Project 16-3307 was to conduct testing on two samples of 16" PE pipe in accordance with ASTM D3350-14 to determine cell classification.

Based on the samples provided and the testing performed in this project, the following conclusions are made:

- The material cell classification of sample 16-349 is PE3255XY
- The material cell classification of sample 16-350 is PE3354XY
- The slow crack growth resistance as determined by PENT, designated X, was below the minimum requirement of 10 h for cell class classification. Both samples failed in less than two hours.
- The hydrostatic design basis at 23°C, designated Y in the material cell classification, was not determined in this project due to limited sample size.

Client:

AECOM Canada Ltd.
99 Commerce Drive
Winnipeg, MB R3P 0Y7
Canada

P.O. Number: 60270487 – Adam Braun

1.0 Purpose of Test

The purpose of Project 16-3307 was to conduct testing on two samples of 16" PE pipe in accordance with ASTM D3350-14 to determine cell classification.

2.0 Test Item Identification and Description

The following samples, as shown in **Table 1**, were provided by the Client. No further details of the samples were provided.

Table 1: Sample Description

Sample ID	Description	Markings
16-349	Black HDPE sample, approximately 17" x 2" x 13"	FGSV INTERCEPTOR
16-350	Black HDPE sample, approximately 13" x 1" x 12"	Newton FM

3.0 Test Methods

The following cell classification testing was conducted in accordance with ASTM D3350-14[1]:

- Density per ASTM D792-13[2], Method A
- Carbon Black and Ash Content per ASTM D1603-14[3] and ASTM D5630-13[4], respectively
- Melt Index per ASTM D1238-13[5] at 190°C/2.16 kg
- Flexural Modulus per ASTM D790-15e2[6]
- Tensile Strength at Yield and Elongation at Break per ASTM D638-14[7]
- PENT per ASTM F1473-16[8] on two (2) specimens at 80°C and 2.4 MPa

Testing to ASTM D792, D1603, D5630, D1238, D790, and D638 were performed at an approved subcontract facility. These tests are within the subcontract facility's ISO 17025 scope of accreditation.

Testing to ASTM F1473 is within NSF Canada's ISO 17025 scope of accreditation (I.A.S. TL-256).

4.0 Test Details

Table 2 and **3** summarize the Cell Classification test results for the PE samples. Detailed test methodology and results are provided **Appendices A** through **F**.

---

[1] ASTM D3350-14 *Standard Specification for Polyethylene Plastics Pipe and Fittings Materials*
[2] ASTM D792-13 *Standard Test Methods for Density and Specific Gravity (Relative Density) of Plastics by Displacement*
[3] ASTM D1603-14 *Standard Test Method for Carbon Black Content in Olefin Plastics*
[4] ASTM D5630-13 *Standard Test Method for Ash Content in Plastics*
[5] ASTM D1238-13 *Standard Test Method for Melt Flow Rates of Thermoplastics by Extrusion Plastometer*
[6] ASTM D790-15e2 *Standard Test Methods for Flexural Properties of Unreinforced and Reinforced Plastics and Electrical Insulating Materials*
[7] ASTM D638-14 *Standard Test Method for Tensile Properties of Plastics*
[8] ASTM F1473-16 *Standard Test Method for Notch Tensile Test to Measure the Resistance to Slow Crack Growth of Polyethylene Pipes and Resins*

Table 2: Summary Cell Classification Test Results for Sample 16-349

Test	Standard	Results		Cell Classification
Carbon Black Content	ASTM D1603-14	Average: 1.97%		
Ash Content	ASTM D5630-13	Average: 0.04%		
Density	ASTM D792-13	Average: 0.947 g/cm^3		3
Melt Index	ASTM D1238-13	190°C/2.16 kg	Average: 0.601 g/10 min	2
Flexural Modulus	ASTM D790-10	Average Flexural Secant Modulus calculated at 2% strain: 874 MPa		5
Tensile Strength	ASTM D638-14	Average Strength at Yield: 24.3 MPa Average Elongation at Break: 681.2%		5
Slow Crack Growth Resistance (PENT)	ASTM F1473-16	Brittle Failure at 0:20[a] and 1:23[a]		0
Hydrostatic Strength Classification	ASTM D2837-13e1	Not Assessed		-

[a] hh:mm

Table 3: Summary Cell Classification Test Results for Sample 16-350

Test	Standard	Results		Cell Classification
Carbon Black Content	ASTM D1603-14	Average: 2.18%		
Ash Content	ASTM D5630-13	Average: 0.02%		
Density	ASTM D792-13	Average: 0.945 g/cm^3		3
Melt Index	ASTM D1238-13	190°C/2.16 kg	Average: 0.293 g/10 min	3
Flexural Modulus	ASTM D790-10	Average Flexural Secant Modulus calculated at 2% strain: 850 MPa		5
Tensile Strength	ASTM D638-14	Average Strength at Yield: 22.3 MPa Average Elongation at Break: 476.2%		4
Slow Crack Growth Resistance (PENT)	ASTM F1473-16	Brittle Failure at 0:21[a] and 1:14[a]		0
Hydrostatic Strength Classification	ASTM D2837-13e1	Not Assessed		-

[a] hh:mm

5.0 Results

Per Section 6.5 of ASTM D3350-14, the reported density refers to the average density of the PE base resin. This value is found after determining the carbon black and ash content and removing their impact from the sample density. Detailed results for each test are provided in **Appendix A** and **B**.

Melt index testing was performed only at 190°C/2.16 kg, as per Section 10.1.4.1 of ASTM D3350-14.

The observed elongation at break of 681.2% and 476.2% for samples 16-349 and 16-350, respectively, both meet the minimum requirement of 400% in Section 6.7 of ASTM D3350-14.

For Sample 16-349, brittle failure at 0:20[9] and 1:23[9] were observed for the PENT test. For Sample 16-350, brittle failures at 0:21[9] and 1:14[9] were observed for the PENT test. These failure times are below the minimum requirement of 10 h for cell classification in ASTM D3350-14.

Hydrostatic testing was excluded due to the limited available sample.

6.0 Conclusions

Based on the samples provided and the testing performed in this project, the following conclusions are made:

- The material cell classification of sample 16-349 is PE3255XY
- The material cell classification of sample 16-350 is PE3354XY
- The slow crack growth resistance as determined by PENT, designated X, was below the minimum requirement of 10 h for cell class classification. Both samples failed in less than two hours.
- The hydrostatic design basis at 23°C, designated Y in the material cell classification, was not determined in this project due to limited sample size.

---

[9] hh:mm

Reviewed by

Ali Ladhani, B.Eng.
Engineering Specialist

Authorized by

Michael Conrad, Ph.D., P.Eng.
Laboratory Manager

Appendix A

Carbon Black and Ash Content

Carbon black content testing was conducted in accordance with ASTM D1603-14 on December 2, 2016. The carbon black content includes residual inorganic matter. Testing was conducted in duplicate. Test results are provided in **Table A1**.

Table A1: Carbon Black Content Test Results

Sample ID	Carbon Black Content (%)		Average Carbon Black Content (%)
	No. 1	No. 2	
16-349	1.92	2.01	1.97
16-350	2.12	2.23	2.18

Ash content testing was conducted in accordance with ASTM D5630-13, Procedure B on December 7 and 8, 2016. The furnace temperature was 800°C. The test duration was between 2.5 and 3.3 h. Testing was conducted in duplicate. Test results are provided in **Table A2**.

Table A2: Ash Content Test Results

Sample ID	Ash Content (%)		Average Ash Content (%)
	No. 1	No. 2	
16-349	0.03	0.04	0.04
16-350	0.02	0.02	0.02

Appendix B

Density

Density testing was conducted in accordance with ASTM D792-13, Method A on December 5, 2016. Specimens were conditioned at 23°C and 50% relative humidity and tested at 20.6°C. Testing was conducted in duplicate. Test results refer to PE resin density, which is calculated per Equation 1 below, and are provided in **Table B1**.

$$Dr = Dp - 0.0044C \qquad (1)$$

where Dr is the resin density
Dp is the compound density
C is the weight percent of carbon black

Table B1: Density Test Results

Sample ID	Density (g/cm^3)		Average Density (g/cm^3)
	No. 1	No. 2	
16-349	0.948	0.946	0.947
16-350	0.945	0.945	0.945

Appendix C

Melt Index

The Melt Flow Rate was determined in accordance with ASTM D1238-13, Procedure A, on December 2, 2016 (190°C/2.16 kg). Three (3) replicates were tested per pipe sample. Test results are provided in **Table C1.**

Table C1: Melt Index Test Results

Sample ID	Test Condition	Melt Flow Rate (g/10 min)			Average Melt Flow Rate (g/10 min)
		No. 1	No. 2	No. 3	
16-349	190°C/2.16 kg	0.603	0.610	0.590	0.601
16-350		0.287	0.294	0.299	0.293

Appendix D

Flexural Modulus

Flexural modulus was conducted in accordance with ASTM D790-10, Method 1, Procedure B, and a 50 mm test span, on December 7, 2016. Samples were prepared in accordance with ASTM D4703-10a. Five (5) replicates were tested per sample, each with a thickness of 3.2 mm ± 0.3 mm "flatwise" at a crosshead speed of 12.7 mm/min. The test specimen width was 12.7 mm. Specimens were conditioned for 40 hours at 23 ± 2°C and 50 ± 10% relative humidity prior to testing. Test results are provided in **Table D1.**

Table D1: Flexural Modulus Test Results

Sample ID	Flexural Secant Modulus calculated at 2% strain (MPa)					Average Flexural Secant Modulus calculated at 2% strain (MPa)
	No. 1	No. 2	No. 3	No. 4	No. 5	
16-349	882	898	947	840	803	874
16-350	811	798	876	915	848	850

Appendix E

Tensile Strength

The Tensile Strength was determined in general accordance with ASTM D638-14 on December 7, 2016.

Five (5) Type IV replicate specimens per pipe sample, prepared in accordance with ASTM D4703-10a were tested. Specimens were conditioned at 23 ± 2°C and 50 ± 10% relative humidity prior to testing at these same conditions. The apparatus used was a dynamometer with a constant rate of extension and testing was performed with an extensometer. The gage length used to calculate elongation percentages at yield point and at break was 25 mm. The test speed was 50 mm/min. Test results are provided in **Tables E1** and **E2**.

Table E1: Tensile Strength Test Results for Sample 16-349

Parameter	Sample 16-349					Average	Standard Deviation
	No. 1	No. 2	No. 3	No. 4	No. 5		
Thickness (mm)	3.46	3.46	3.44	3.49	3.50	3.47	0.02
Width (mm)	6.40	6.28	6.43	6.60	6.40	6.42	0.11
Stress at Yield (MPa)	24.5	24.3	23.2	24.7	24.9	24.3	0.7
Elongation at Yield (%)	11.1	11.3	10.7	10.4	11.0	10.9	0.4
Stress at Break (MPa)	14.6	16.1	13.4	15.8	16.3	15.2	1.2
Elongation at Break (%)	632.5	711.6	447.9	802.9	811.3	681.2	149.6

Table E2: Tensile Strength Test Results for Sample 16-350

Parameter	Sample 16-350					Average	Standard Deviation
	No. 1	No. 2	No. 3	No. 4	No. 5		
Thickness (mm)	3.51	3.54	3.54	3.51	3.52	3.52	0.02
Width (mm)	6.44	6.39	6.47	6.57	6.49	6.47	0.07
Stress at Yield (MPa)	21.7	22.6	22.5	22.2	22.4	22.3	0.4
Elongation at Yield (%)	11.4	10.5	10.9	11.0	10.6	10.9	0.4
Stress at Break (MPa)	12.8	12.9	12.7	13.0	12.7	12.8	0.1
Elongation at Break (%)	569.9	494.8	482.6	593.4	240.3	476.2	140.1

Appendix F

Pennsylvania Notch Test (PENT)

PENT was conducted in accordance with ASTM F1473-16. Testing was performed on January 10, 2017. Resin mix was roll milled, compression molded and machined to manufacture the test specimens. Testing was conducted in duplicates at 80°C and at a stress of 2.4 MPa based on the un-notched area. Test results are provided in **Tables F1** and **F2.**

Table F1: PENT Test Results for Sample 16-349

Sample ID	**16-349**	
Average Time to Failure (hh:mm)	00:51	
Specimen No.	01	02
Specimen Width (mm)	25.02	25.13
Specimen Thickness (mm)	10.04	10.06
Test Temperature (°C)	80	80
Stress (MPa)	2.4	2.4
Main Notch Depth (mm)	3.51	3.51
Side Groove Depth (mm)	1.00 / 1.00	1.00 / 1.00
Calculated Load (N)	603	607
Status	Brittle Failure	Brittle Failure
Time to Failure (hh:mm)	01:23	00:20

Table F2: PENT Test Results for Sample 16-350

Sample ID	**16-350**	
Average Time to Failure (hh:mm)	00:47	
Specimen No.	01	02
Specimen Width (mm)	25.10	25.13
Specimen Thickness (mm)	10.01	10.06
Test Temperature (°C)	80	80
Stress (MPa)	2.4	2.4
Main Notch Depth (mm)	3.50	3.51
Side Groove Depth (mm)	1.00 / 1.00	1.00 / 1.00
Calculated Load (N)	603	607
Status	Brittle Failure	Brittle Failure
Time to Failure (hh:mm)	00:21	01:14