The Corporation of the City of Sault Ste. Marie

The Fort Creek Aqueduct
Watershed Appraisal and Hydraulic Assessment

July 2016

15-1192

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July 29th, 2016
15-1192.01

The Corporation of the City of Sault Ste. Marie
Engineering Department
Level 5 - Civic Centre
99 Foster Drive,
P.O. Box 580, Sault Ste. Marie, ON
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Attn: **Mr. Carl Rumiel, P.Eng.**
Design and Construction Engineer

Re: **Fort Creek Aqueduct Watershed Appraisal and Hydraulic Assessment**
Sault Ste. Marie, Ontario

Dear Mr. Rumiel:

Please find enclosed our final report titled "Fort Creek Aqueduct Watershed Appraisal and Hydraulic Assessment". The report describes the current drainage characteristics, assesses the capacity of the current drainage infrastructure and makes recommendations for conveyance improvements.

We trust you will find the information presented acceptable. This report has been prepared in part for the information and inclusion into the Environmental Assessment currently being undertaken for this project. Should you have any questions, please do not hesitate to contact the undersigned at your convenience.

Sincerely,

TULLOCH Engineering Inc.

John McDonald, P.Eng.
Project Manager

JM/bt

Encl.

TULLOCH ENGINEERING

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2	1	The Corporation of the City of Sault Ste. Marie
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Revision Log

Revision #	Issued By	Date	Issue / Revision Description
0	JVM	April 14/2016	Draft - Issued for Client Review
1	JVM	July 29/ 2016	Final Report – Issued for EA Project File

Engineers Seal

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1. INTRODUCTION

1.1 Purpose of the Appraisal

The City of Sault Ste. Marie has initiated a Class Environmental Assessment (Class EA) to identify and evaluate alternative ways to make improvements to the underground portion of the Fort Creek drainage system (the Aqueduct) between its inlet at Carmen's Way and its outlet at the Canadian Pacific Railway (CPR) crossing just north of Edinburgh Street.

Originally constructed between 1912 and 1917, concerns have been raised about its structural adequacy and its hydraulic capacity to carry flows from major rainfall and runoff events. The purpose of this appraisal is to hydraulically assess the identified preferred alternative through the Steelton area derived from the Class EA process and including the 'Do Nothing' alternative. This report has been prepared in part for inclusion into the Class EA project file and recognizes that in order to hydraulically assess the drainage in the Steelton area the study area must be expanded and includes the entire Fort Creek watershed and both its major and minor drainage systems.

1.2 Study Approach

The study methodology generally follows a linear approach by first hydrologically evaluating the existing drainage area, then performing a hydraulic analysis of the minor drainage system, followed by a final integrated hydraulic analysis of the minor and major drainage system as a whole. Utilizing the results of these analyses, and based upon direction received from the Class EA framework to which this analysis forms a part of, this study then hydraulically analyzed a series of alternatives to address identified deficiencies within the existing underground aqueduct and major drainage system.

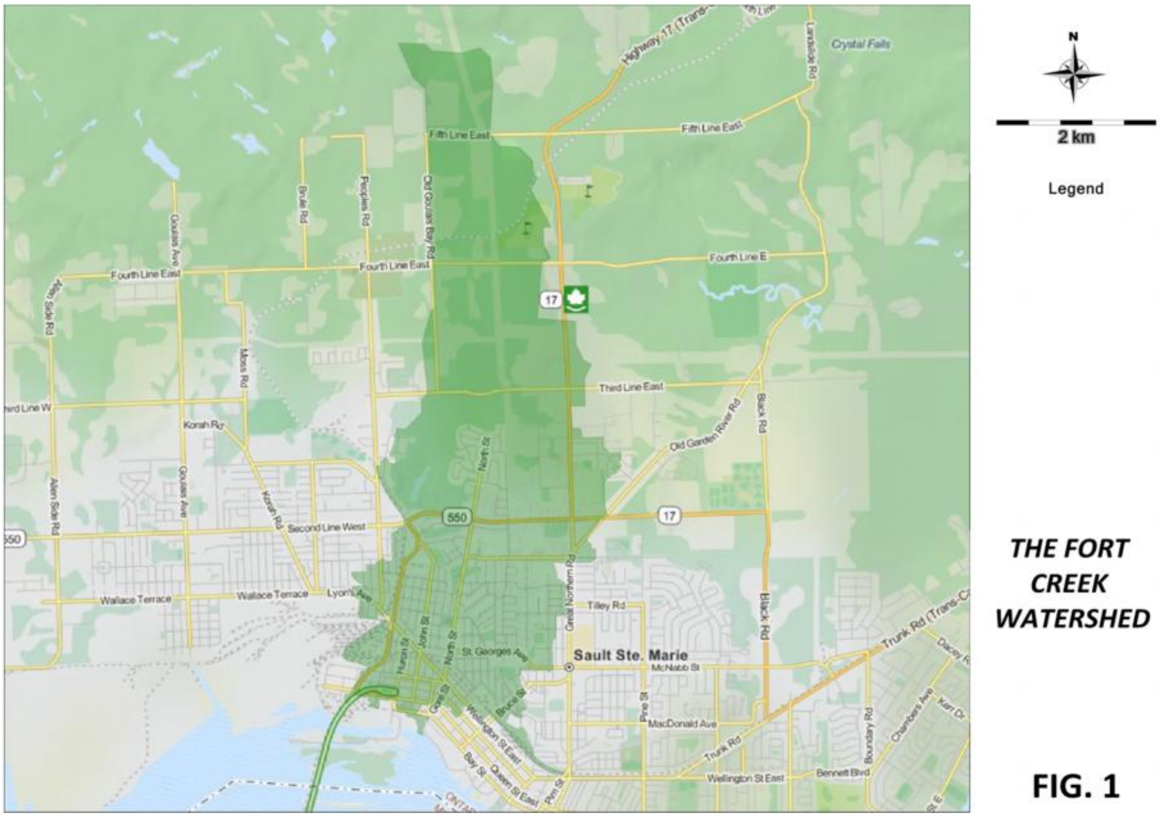
This appraisal has been organized to systematically develop and evaluate the hydrologic and hydraulic characteristics of both the Major and Minor drainage systems to ensure accuracy and to critically examine the assumptions, hydrologic methods and hydraulic evaluations of past reports, all in order to present a comprehensive appraisal of the watershed and its drainage systems.

The general steps undertaken in completing this appraisal were as follows;

- Develop scope of study
- Review of past studies
- Data acquisition
- Estimation and calculation of watershed parameters
- Creation of design storms
- Review of the minor drainage system
- Review of the major drainage system
- Development and cataloging of assumptions
- Hydrologic and hydraulic analysis of the existing minor and major drainage system
- Model verification
- Creation and evaluation of alternative solutions
- Develop recommendations and conclusions

1.3 General Description of the Watershed

The Fort Creek Watershed (shown in Figure 1) encompasses a total catchment area of approximately 1516 hectares and has a total length from its source near Fifth Line to its outlet south of Bay Street West of approximately eight (8) kilometers. In the early 1970's a dam was constructed creating the Fort Creek Reservoir with the purpose of providing protection against flooding to the lower heavily urbanized portion of the watershed. The flood control dam intercepts runoff from an area of approximately 860 hectares. The Fort Creek Dam was designed prior to the advent of the 'Regional Storm' design concept, however post-construction studies have determined that it does serve to mitigate flows resulting from infrequent storms in excess of 1:100 year events, including the Regional Storm albeit in a somewhat uncontrolled fashion. The operation and functionality of the dam is described later in this report.



THE FORT CREEK WATERSHED

FIG. 1

The upper portion of the watershed as far south as Second Line is largely rural in character and includes the steep slopes of the escarpment; however, the area has experienced increased development pressure in the last two decades especially along the Great Northern Road corridor. South of Second Line the topography is generally two tiered with the easterly area being the upper escarpment and the lower relatively flat areas below the escarpment gently sloping toward the south.

This lower portion of the watershed south of Second Line is primarily a built up area and heavy urbanization has occurred south of Conmee Avenue. The Wellington Street corridor from North Street to Carmen's Way is commonly known as Steelton in reference to the time period prior to this areas amalgamation with the City of Sault Ste. Marie in 1918.

The Fort Creek Watershed ultimately outlets to the St. Marys River South of Bay Street West immediately to the west of St. Mary' s River Drive.

1.4 Previous Studies

Proctor and Redfern Ltd. – E.O. 59521 – "Flood Control of Fort Creek", 1960

This report was commissioned by the City of Sault Ste. Marie and prepared in May of 1960. It examined alternatives for the control of flooding of the Fort Creek watershed, namely, increasing waterway capacity or providing a reservoir by means of a dam. Due to economic considerations, the construction of a dam was recommended.

Proctor and Redfern Ltd. – E.O. 6233 – "The Construction of a Dam at Fort Creek", 1960

This report, commissioned by the City of Sault Ste. Marie, was written in April, 1960 and identified the design aspects of the proposed Fort Creek Dam.

Proctor and Redfern Ltd. – E.O. 679 – "Fort Creek Aqueduct Appraisal", 1967

This appraisal compared the costs and other effects of construction of additional aqueduct capacity as an alternative to the construction for the Fort Creek Dam. Written in 1967 and commissioned by the Sault Ste. Marie Region Conservation Authority, it recommended that the Fort Creek Dam be constructed due to its substantially lower costs and many secondary benefits in the area of recreation.

Proctor and Redfern Ltd. – E.O. 70131 – "Fort Creek Channel, Second Line to Aqueduct", 1970

Commissioned by the Sault Ste. Marie Region Conservation Authority in June of 1970, the purpose of this report was to consider the natural open channel between the Fort Creek Dam site and the aqueduct entrance on Hudson Street and recommend flood control measures, maintenance and aesthetic improvements.

M.M. Dillon Ltd. – 7095-01 – "Flood Plain Mapping Report", 1977

Written in November of 1977, the Sault Ste. Marie Region Conservation Authority commissioned this report with the main purpose of updating existing contour mapping of the watersheds within its jurisdictional region and to prepare flood line mapping of same, utilizing the Ontario Government's "Regional Storm" criteria.

The Trow Group,- "Geotechnical Study, City of Sault Ste. Marie, January 1977

This study provides a comprehensive general overview of the geologic formations of the Sault Ste. Marie area. Ground water table elevations, soils stratum and distributions, and engineering properties are provided within the report in addition to common constructability considerations.

Proctor and Redfern Ltd. – E.O. 84521 – "Fort Creek Channel, Second Line to Aqueduct", 1984

This 1984 study examined the overall level of flood protection in the Fort Creek Watershed with the purpose of determining the most cost effective method of accommodating flood flows up to the Regional Storm. The scenarios examined included the costs of maintaining the status quo, the costs of repairs to the existing aqueduct and complete replacement of the aqueduct.

The study recommended that the Fort Creek Aqueduct be maintained in service for the foreseeable future and that the entrance to the aqueduct be examined in detail to determine if improvements can be made and that a flood warning system be considered for the section of watershed downstream of the Fort Creek Dam.

Proctor and Redfern Ltd. – E.O. 87039 – "Fort Creek Flood Reduction Study", 1988

The purpose of this study prepared in 1988 was to examine the Fort Creek channel and culverts between John Street and the entrance to the existing aqueduct opposite Hudson Street and to determine if measures could be provided, economically to reduce the risk of flooding. Additionally, the study included examination of the open channel at Wellington Street and the CP Rail Line. An in depth review of the effectiveness of the Dam and reservoir is contained within including Stage/Storage/Discharge relationships and estimated peak flows at various locations of the Fort Creek reach from the Dam to the St. Marys River.

Conestoga-Rovers and Associates. – 078160-00 – "Fort Creek Aqueduct Hydrologic/Hydraulic Analysis", 2014

Conestoga-Rovers and Associates as a sub-consultant to STEM Engineering were commissioned by the City of Sault Ste. Marie to conduct an assessment of the Fort Creek aqueduct system. The report identified hydraulically deficient sections and evaluated potential future upgrades to alleviate identified areas with deficient hydraulic capacity.

Ontario Ministry of Transportation, Highway Standards Branch, Design and Construction Standards Office. – "The Resilience of Ontario Highway Drainage Infrastructure to Climate Change", 2015

The study undertook an assessment of the resilience of Ontario Ministry of Transportation (MTO) Drainage Infrastructure to Climate Change scenarios using samples of actual highway projects. An investigation into the possible magnitude of changes in rainfall was undertaken using available tools and climate change modelling studies that provide the most current and relevant hydrologic data. The study provides a basis for identifying the degree of effectiveness of the current drainage design standards and procedure for future life cycles of the different drainage infrastructure components; storm sewers, culverts, and bridges. The study also provides some adaption strategies to address climate change impacts on the provincial highway network over the range of the design service life for the different components of the highway infrastructure managed by the MTO.

1.5 Drainage Systems

The City Sault Ste. Marie is served by a dual drainage system consisting of a minor drainage system (including but not limited to; ditches and swales, overland flow routes and, the piped storm sewer system) and a major stormwater drainage system (including but not limited to; overland flow paths, flood control dams and channels and aqueduct systems). The design of stormwater drainage systems generally includes the consideration of drainage for both the minor and major storms. The design of dual stormwater drainage systems reduce the risk proposed or existing structures are damaged by the runoff generated by a major storm event. This requires proper design of streets, curb and gutters, catch basins, pipes, open channels, grading of lots and road profiles, setting of elevations and openings into buildings, foundation drains, roof drains, or other “off-street” connections.

1.5.1 The Minor Drainage System

The minor stormwater drainage system is designed to eliminate or minimize inconveniences or disruption of activity resulting from runoff produced by more frequent, less intense storms. The minor drainage system is sometimes termed the “initial system” and may include features such as curbs and gutters, storm sewer pipes and open drainage channels/roadside ditches.

Current City of Sault Ste. Marie design standards for the minor drainage system is based on a storm frequency of 1 in 10 years (or a storm of such magnitude and intensity with a 10% chance of exceedance in any given year). The storm water piping system (the minor drainage system) falls under the jurisdiction of the City.

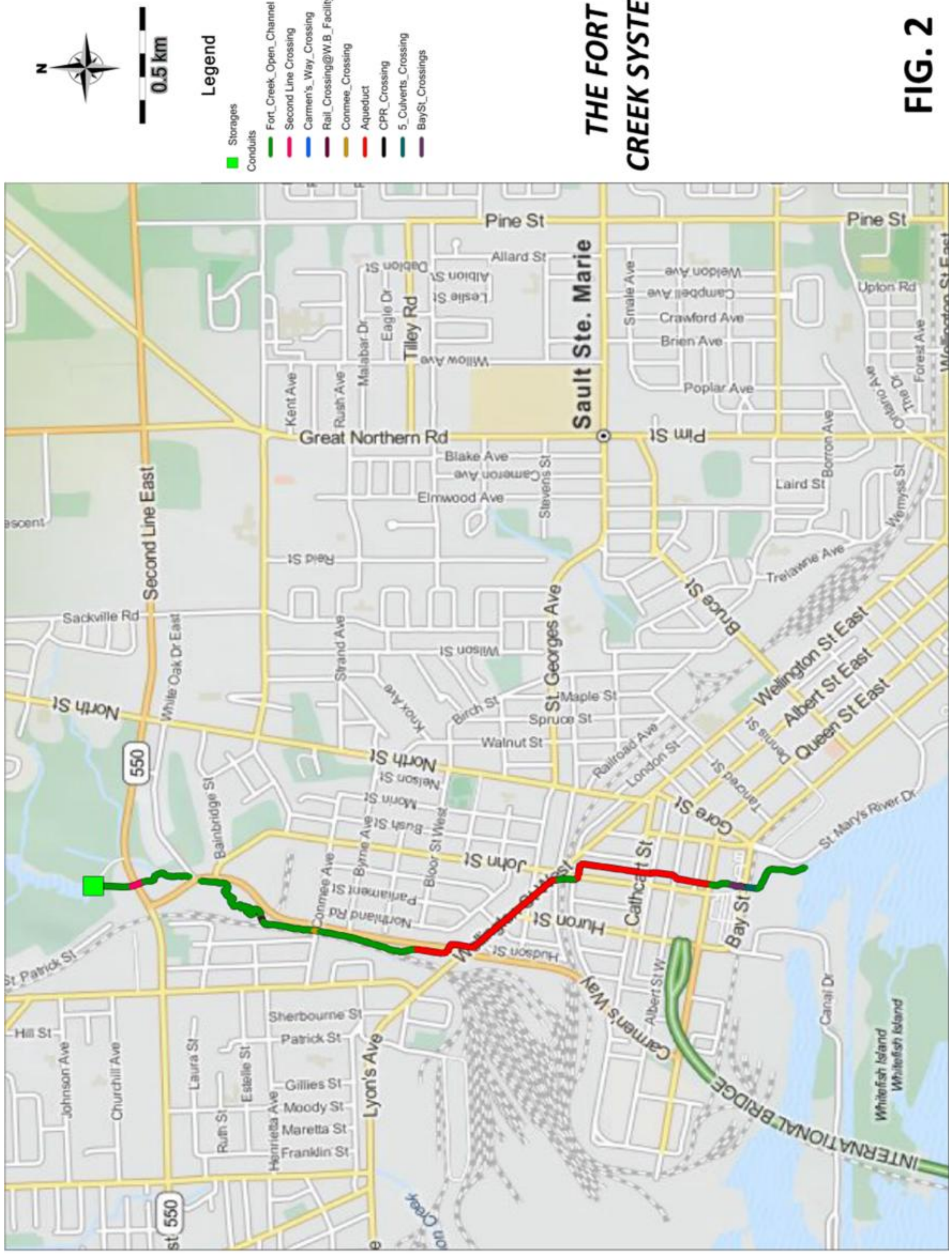
1.5.2 The Major Drainage System

The major stormwater drainage system in general terms is the system which conveys stormwater during a major storm when the capacity of the minor system is exceeded. The major system usually includes features such as streets, curb and gutter systems, swales, and major drainage channels (and aqueducts). The minor stormwater drainage systems may reduce the flow in many parts of the major stormwater drainage system by storing and conveying water underground. The current City of Sault Ste. Marie major drainage system design standard is based on a storm frequency of 1 in 100 years (or a storm of such magnitude and intensity with a 1% chance of exceedance in any given year) and the Regional Storm (Timmins).

Generally, the streams and channels of the major drainage system within the City of Sault Ste. Marie fall under the jurisdiction of the Sault Ste. Marie Region Conservation Authority (SSMRCA) as is the case with the Fort Creek reservoir, dam, and open channels. The aqueduct however is maintained by the City. A detailed description of the system is provided next. All stormwater is generally conveyed southerly and ultimately discharges to the St. Marys River.

1.6 The Fort Creek

The Fort Creek from its furthest upstream reaches to its outlet at the St. Marys River flows through and is comprised of a network of open natural channels, a reservoir and dam, man-made and realigned open channels, culverts and bridge crossings, and large underground box culverts (aqueduct). Figure 2 presents a graphical overview of the Fort Creek system from the Fort Creek Dam to the outfall at the St. Marys River.



THE FORT CREEK SYSTEM

FIG. 2

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The SSMRCA constructed a flood control dam and reservoir immediately upstream of Second Line in the early 1970's. The Fort Creek dam and reservoir captures and controls the discharge of runoff to the lower watershed from an area of approximately 840 hectares. The dam and constructed reservoir discharges flow at an attenuated rate through a series of natural and constructed channels, large diameter circular, elliptical and box culverts prior to entering the underground aqueduct system on Carmen's Way in the vicinity of Bloor Street. The Fort Creek is contained within the concrete aqueduct from the inlet at Carmen's Way to Queen Street West near John Street with the exception of a short length of open channel at the CPR crossing near the tri intersection of John Street, Wellington Street West and St. Andrews Terrace. The lower portion of the Fort Creek is in an open channel south of Queen Street West to the St. Marys River.

The dam and reservoir were designed and constructed prior to the Provincial criteria requiring the use of the "Regional Storm" concept (The Timmins Storm). Past studies and reports examining the operation of the dam and reservoir have concluded the low flow conduit which is controlled by a 3.5ft x 3.5ft (1.067m x 1.067m) sluice gate cannot accommodate the peak flows during the Regional Storm; however, the dam was designed with a working overflow weir at elevation 673.0 feet (205.13m). This closed horizontal hexagonal weir can accommodate flows in excess of the peak flows from the Regional Storm event, albeit in a somewhat uncontrolled nature. Acknowledging this may raise doubts as to the effectiveness of the dam during storms of this magnitude, however we note past hydrologic analyses and the results of our hydrologic analysis have determined that the uncontrolled flows are relatively minor in nature and occur after peak flows within the downstream urban watershed have subsided.

A sluice gate setting of 10% open (0.1067m) or less during a Regional Storm event would result in overtopping of the hexagonal weir. Gate settings greater than 10% would not result in overtopping of the working overflow weir. Yet, overtopping of the hexagon weir with a sluice gate opening of 10% or less has a less significant effect downstream than opening the sluice gate in excess of 50%.

In general, Proctor and Redfern Consulting Engineers concluded that 1:100 year storm flows can be fully retained by the Fort Creek Dam and Reservoir without overtopping the overflow weir at elevation 673.0 feet (205.13 m). Also, peak flows downstream of the dam are not affected by outflow from the dam during 1:100 year storm occurrences. This is due to the fact that downstream peak flows occur at a point in time prior to significant contributions of flow from the dam.

The dam is constructed with an emergency spillway containing five "erodible plugs" starting at elevation 676.0 feet (206.04 m) increasing to elevation 677.5 feet (206.50m) - 1.5 feet (0.46m) below the crest of the dam. The emergency spillway is designed to prevent a catastrophic failure of the dam and is therefore capable of accommodating flows greatly in excess of the Regional Storm.

The Fort Creek dam has served to reduce the frequency of flooding downstream within the urban core of Steelton. Although existing hydrologic design reports, and flood plain mapping reports have varying opinions on the capacity and discharge rates, one fact is consistent – the dam has shown to be highly effective at reducing peak flow rates downstream and lagging the time to peak of this flow. Variations in the estimates of storage capacity and discharge rates can be attributed to numerous factors (primarily the magnitude and duration of the design storm and the pre-event water level within the reservoir).

Through discussion with the SSMRCA, the accumulation of silt and sedimentation within the Fort Creek Reservoir is occurring and the volume of 'dead' storage is decreasing. In one of Proctor and Redfern's previous studies the initial calculation of dead storage volume was 20 ha.m. Until this volume of siltation is realized, it should have little to no effect on the attenuating effects of the dam and therefore has been ignored for the purposes of modeling. It should be clarified that the dead storage is occupied by a permanent pool of water and thus the term 'storage' can lead to confusion as it is preoccupied volume that cannot store additional runoff during storm events.

The Sault Ste. Marie Region Conservation Authority does not have a formal operational plan in place for the operation of the dam. A draft report was provided to Tulloch Engineering by the SSMRCA during the preparation of this study, however it is incomplete and yet to be enacted and approved by the Board of Directors.

The 1988, "Fort Creek Flood Reduction Study" prepared by Proctor and Redfern provided the following operational recommendations, "In the event of a major storm occurrence, the sluice gate at the Fort Creek Dam should remain at approximately 10% open position. If water levels in the reservoir rise in excess of elevation 663 ft. (202.08m) the storm occurrence is in excess of a 1:100 year storm. The 10% open position minimizes downstream effects yet still maintains sufficient discharge rates to keep the reservoir at low level. Immediately following a significant rise in reservoir level, the sluice gate should be opened to 50% in order to restore low reservoir elevation (650ft/198.12m)".

For analysis purposes, we have used this recommendation by Proctor and Redfern and set the sluice gate opening at 10% or 0.107m open for all storm events simulated.

The 1988, "Fort Creek Flood Reduction Study" prepared by Proctor and Redfern provides tabular Storage/Stage Discharge data for various sluice gate settings. These storage discharge relationships were also computed for comparison purposes. Storage/Stage Discharge relationships computed by Tulloch Engineering are based on the City of Sault Ste. Marie GIS topographic information. The relationships correlated well and were incorporated into the model allowing for functionality of the dam and reservoir to be modeled in the analysis.

The Fort Creek dam discharges to a concrete rectangular spillway which transitions to a trapezoidal channel just north of Second Line. Flow is conveyed under Second Line through a double barrel 2.8 m x1.5 m concrete box culvert. On the south side of Second Line, the conveyance system transitions back to a trapezoidal channel. Although the channel appears to be located in its original location between Second Line and the Carmen's Way crossing, improvements to the cross-section have occurred. The watercourse crosses Carmen's Way through a double barrel 2.8m x 1.8m double box culvert reappearing on the west side of Carmen's Way again in a trapezoidal channel. The channel continues southerly paralleling the west side of Carmen's Way with crossings at the Canadian National rail spur servicing the former welded beam factory consisting of a 3.0m x 1.5m corrugated steel arch pipe, and Conmee Avenue consisting of a 1.7m x 1.3m double concrete box culvert. The open channel reaches the inlet to the aqueduct generally opposite the Bloor Street right-of-way on the west side of Carmen's Way. The aqueduct traverses southerly to approximately Bloor Street, paralleling Carmen's Way prior to crossing Carmen's Way to the north east corner of the intersection with Wellington Street West. The aqueduct subsequently crosses to the south side of Wellington Street and immediately turns and proceeds south easterly under the south sidewalk of Wellington Street, to which the roof of the aqueduct forms the sidewalk. The aqueduct proceeds on this path to the tri-intersection of Wellington Street West, John Street, and St. Andrews Terrace. The conveyance system, once reaching this area turns southerly through the Esso gas station, crossing St. Andrews Terrace and discharging to an open channel under the CPR masonry arch crossing and immediately re-entering an underground aqueduct south of the CPR crossing on the north west corner of the intersection of John Street and Edinburgh Street. Once underground, the aqueduct crosses Edinburgh Street, immediately turns 90^o to the east, crosses John Street into a laneway and again turns 90^o and continues southerly paralleling John Street in this laneway, crossing Cathcart Street, Alexandra Street, Albert Street, Central Park Avenue and Queen Street before being discharged to an open rectangular, sheet pile lined open channel on the south side of Queen Street. The channel dissects the Ontario Lottery and Gaming Sault Ste. Marie Casino (OLGSSM) prior to crossing under in quick succession three (3) bridge structures consisting of a pedestrian bridge, an internal access road bridge servicing the OLGSSM and the Bay Street crossing. South of Bay Street, the channel encounters five (5) 2000mm ø CSP parallel culverts allowing the crossing of the CN railway prior to its final trapezoidal open channel 250m leg to the St. Marys River.

The Fort Creek Aqueduct was constructed circa between 1912 and the 1940 with various roadway crossing constructed first, then filled in-between afterwards. The construction of the Fort Creek dam in the early 1970's resulted in reduced peak flows being discharged into the lower urban watershed and therefore, the aqueducts' ability to convey flows from large return period storm events (i.e. the 1:50 year and 1:100 year storms) was substantially increased.

2 HYDROLOGIC ANALYSIS

2.1 General Parameters and Considerations

The analysis of the hydrologic system was completed by using mathematical models as described in Section 3. These models are both empirical and statistical, and founded on known physical laws. The choice of the methods of analysis was based on the purpose for what it is being used. Data on hydrologic variables are fundamental to the analysis and modeling. Presented herein are the various parameters and variables and the methods in which they were derived.

2.1.1 Sub-watersheds and Catchment Areas

Catchment and subcatchment areas were delineated based on City of Sault Ste. Marie Geographic Information System (GIS) information. Due to the functionality and purpose of the Fort Creek reservoir and dam, the catchment area contributing to the reservoir was modeled as a single watershed, whereas the urban area south of the dam was modeled using 698 subcatchment areas.

An initial estimate of the characteristic width is given by the subcatchment area divided by the average maximum overland flow length. The maximum overland flow length is the length of the flow path from the inlet to the furthest drainage point of the subcatchment area.

Subcatchment slopes were derived from topographic information provided by the City of Sault Ste. Marie GIS data. Subcatchment slopes were calculated to best represent the average slope of a representative flow path.

2.1.2 Manning's Roughness Coefficients

Roughness coefficients represent the resistance to flows in channels, pipes and flood plains. It would be impractical in this report to record the detailed methodology and factors that were used in selection of the coefficients used within the analysis. Given the importance of the Manning's n value in open channel flow computations, careful selection was given and the following ranges Manning's Roughness Coefficients were used:

Table 1: Manning's Coefficients

Surface	Min. n	Max. n
Pipe Conduits	0.01	0.024
Aqueducts	0.012	0.015
Open Channels – Below Stage	0.04	0.07
Open Channels – Bank Full	0.07	0.24
Open Channels – Overbank	0.015	0.24
Impervious Areas	0.011	0.016
Pervious Areas	0.15	0.6

2.1.3 St. Marys River Water Levels

Boundary condition water levels at the extreme downstream reach of the watershed at the outlet of the Fort Creek to the St. Marys River can significantly affect the hydraulic capacity of both the aqueduct (major drainage system) and thus by extension, the storm sewer network (minor system).

The 'lower' St. Marys River is referenced as such to describe the river below the rapids, power generating systems and the series of locks connecting the easterly upper St. Marys River - Lake Superior influenced waterway with the western lower St. Marys River – Lakes Huron and Michigan influenced waterway. The discharge of water from the upper St. Marys to the lower St. Marys River is controlled via the two power generating stations and a system of compensating gates. The requirements to maintain a sufficient draft for commercial shipping has a strong controlling influence on minimum water levels in the St. Marys River whereas higher water levels require hydrologic/climatic conditions that would warrant higher discharges through the power generating stations and compensating gates.

As higher water levels have a negative effect on the hydraulic capacity of the aqueduct conveyance system, Tulloch Engineering has selected the maximum average monthly mean water level of 177.96 m as the design water level within the St. Marys River at the outlet of the Fort Creek channel. This elevation presents a conservative, yet realistic water level which could reasonably be observed during seasonal timeframes when substantial storm events statistically occur. This information was obtained from Environmental Canada's Water Office of the Department of Fisheries and Oceans (https://wateroffice.ec.gc.ca) using Station: 02CA005/.

We note, the published water elevation data required conversion to Canada Geodetic Survey (CGS) elevations to relate to the City of Sault Ste. Marie benchmarks to which their infrastructure construction is referenced to.

2.1.4 Soil Types and Infiltration (Curve Numbers – CN)

There are numerous techniques and models available for use in the determination of stormwater runoff. Selection of an appropriate method should be based on an understanding of the principles and assumptions of the method and of the problem under consideration. A commonly used method of estimating losses and determining runoff is the "Hydrologic Soil Complex Method" of the United States Department of Agriculture, Soil Conservation Service (SCS method). The SCS method was selected for use as it has been used extensively for hydrologic analysis in the Sault Ste. Marie area in the past, the method is well documented in these past studies, quality land use and soil type data exists as applicable to the method, and the SCS method is an approved method for analysis in accordance with the City of Sault Ste. Marie Storm Water Management Guidelines, 2015.

The SCS method uses a combination of soil conditions and land use (ground cover) to assign a runoff factor to an area. These runoff factors, called runoff curve numbers (CN), indicate the runoff potential of an area when the soil is not frozen.

In the SCS method, direct runoff is estimated by the use of runoff curve numbers which are related to soils groups and land use. The higher the curve numbers the higher the runoff. Four hydrologic soil groups have been defined:

- (A) Low runoff potential – soils having a low runoff potential due to high infiltration rates (These soils consist primarily of deep, well drained sands and gravels);
- (B) Soils with moderate infiltration rates – soils having moderate a moderately low runoff potential due to moderate infiltration rates (These soils consist primarily of moderately-deep to deep, moderately well- to well drained soils, with moderately fine to moderately coarse textures.);
- (C) Soils with low infiltration rates- soils having a moderately high runoff potential due to slow infiltration rates (These soils consist primarily of soils in which a layer exists near the surface that impedes the downward movement of water or soils with moderate –fine to fine in texture.) and,
- (D) High runoff potential – soils having a high runoff potential due to very slow infiltration rates (These soils consist primarily of clays with high swelling potential, soils with permanently high water tables, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious parent material.

Figure 3 depicts the distribution of soils types within the watershed. Soil types and soil distributions are based on the report prepared by The Trow Group, titled "Geotechnical Study, City of Sault Ste. Marie, January 1977" and upon Tulloch Engineering's extensive data base of geotechnical investigations within the City of Sault Ste. Marie. The watershed contains a variety of soil types consisting of Lacustrine Sand in the northern reaches of the watershed to Lacustrine Clay in the mid-area of the watershed to Glacial Till in the southern portion of the watershed. Pockets of Glacial Till and Alluvium wash are interspersed throughout. The following CN values were assigned for each soil type:

Table 2: CN Values

Landform	Assigned CN Value
Lacustrine Sand (Sp)	30
Glacial Till (Tp)	70
Alluvium (Ar)	77
Lacustrine Clay (Cp)	77

Each subcatchment within the model was assigned the appropriate CN value based on the soil type it is situated within. A composite CN value was calculated for those subcatchment areas that contained two or more soil types.

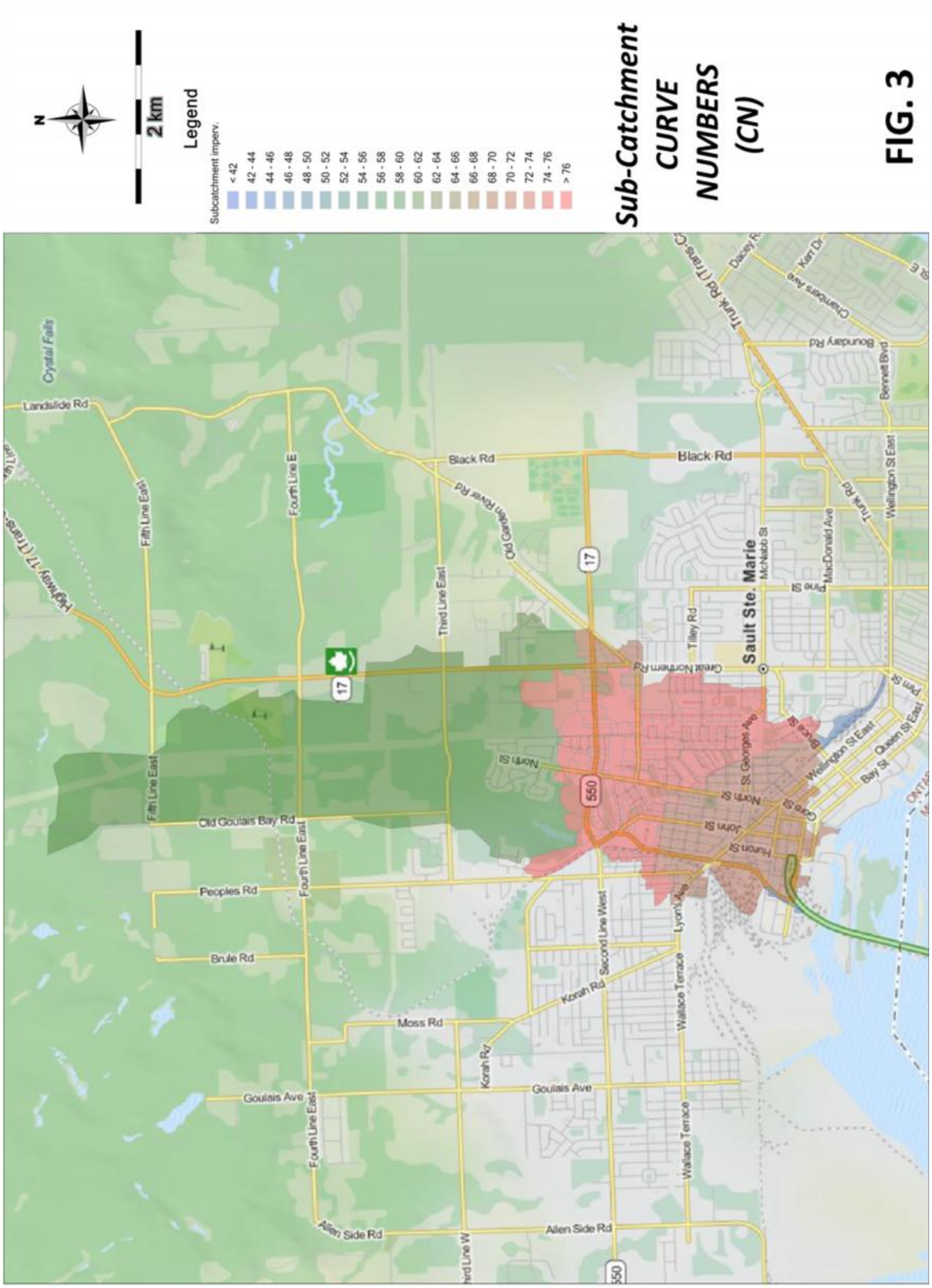

N
2 km
Legend
Subcatchment imperv.
< 42
42 - 44
44 - 46
46 - 48
48 - 50
50 - 52
52 - 54
54 - 56
56 - 58
58 - 60
60 - 62
62 - 64
64 - 66
66 - 68
68 - 70
70 - 72
72 - 74
74 - 76
> 76
Sub-Catchment CURVE NUMBERS (CN)
Sault Ste. Marie
Black Rd
Landslide Rd
Fifth Line East
Fourth Line E
Third Line East
Old Goulais Bay Rd
Peoples Rd
Brule Rd
Moss Rd
Goulais Ave
Allen Side Rd
Korah Rd
Second Line West
Wallace Terrace
Great Northern Rd
North St
John St
Huron St
Lyons Ave
Wellington St East
Queen St East
Bay St
Boundary Rd
Bennett Blvd
Trunk Rd
Tilley Rd
Pine St
MacDonald Ave
McNabb St
St. Georges Ave
Old Garden River Rd
Crystal Falls
17
550

FIG. 3

2.1.5 Antecedent Moisture Conditions (AMC)

The word antecedent simply means "preceding conditions". Antecedent moisture is a term that describes the relative wetness or dryness of a watershed, which changes continuously and can have a very significant effect on the flow responses in these systems during wet weather.

If the watershed is saturated from antecedent rains or is frozen, the runoff potential is higher than indicated by curve numbers derived from average antecedent moisture conditions. Generally, antecedent moisture conditions are reduced to the following three cases:

AMC 1: a condition of watershed soils where the soils are dry.

AMC 2: the average case for floods.

AMC 3: a condition when heavy rainfall occurs during the five days previous to the given storm

The U.S. Department of Agriculture, Natural Resources Conservation Service (NRCS), formerly the Soil Conservation Service (SCS), defines AMC in terms of total rainfall during the 5 days immediately preceding the rainfall event. Dry AMC conditions mean less than 1.4 inches (<35.5mm), average is 1.4 to 2.1 inches (35mm – 53mm), and wet is greater than 2.1 inches (>53mm).

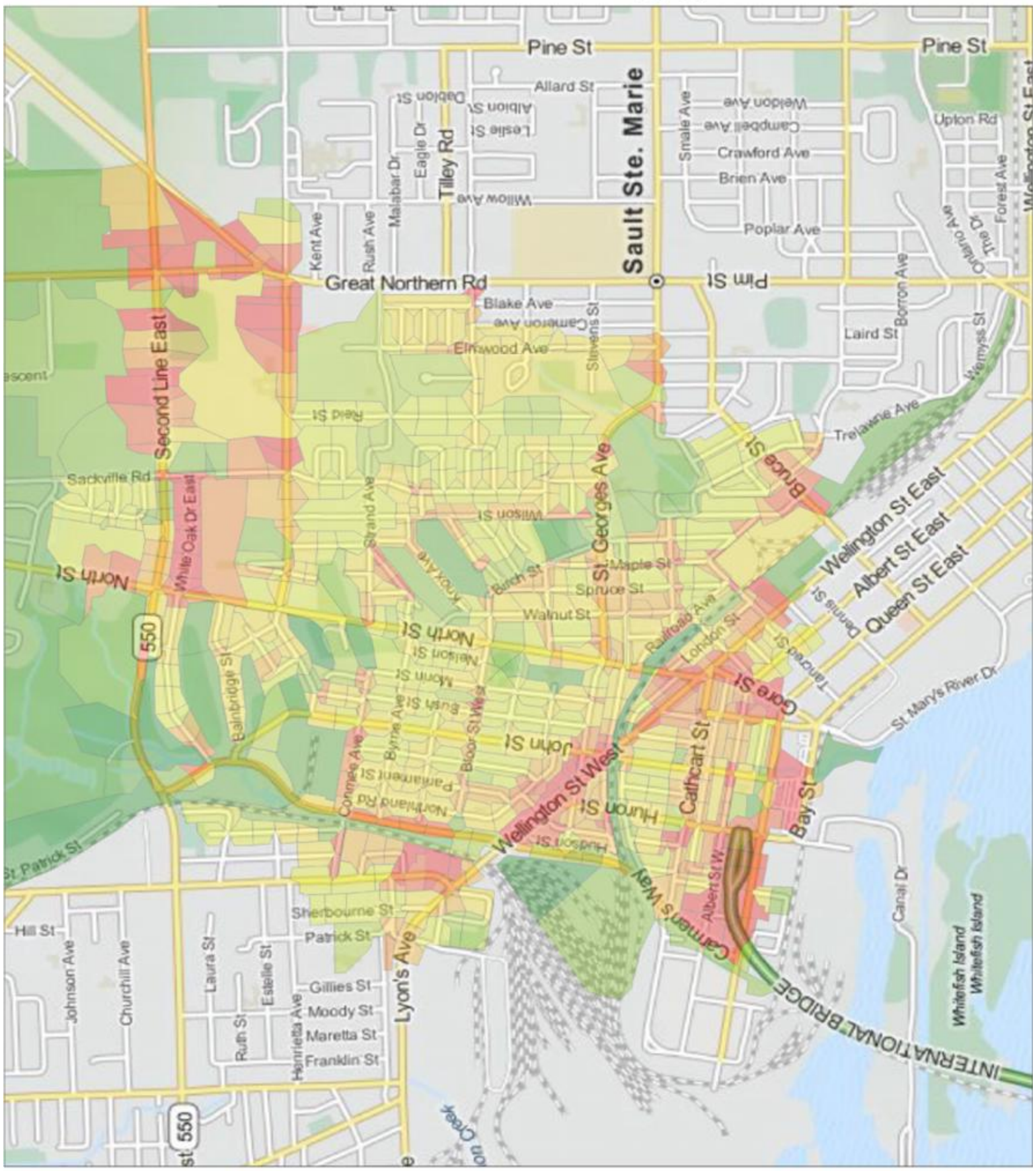
For the purposes of this report, we have assumed the condition of the watershed to be at its average condition. We therefore, must recognize that any of the frequency storms analyzed could produce greater or lesser flows accordingly if the prior condition of the watershed is wetter or dryer than its average condition.

2.1.6 Impervious Levels

Impervious surfaces are mainly man-made structures such as pavements (roads, sidewalks, driveways and parking lots) that are covered by impenetrable materials such as asphalt, concrete, brick, stone and rooftops. Soils compacted by urban development are also highly impervious.

The total coverage by impervious surfaces in an area, (such as a municipality or a watershed) is usually expressed as a percentage of the total land area. The coverage increases with rising urbanization. In rural areas, impervious cover may only be one or two percent. In residential areas, coverage increases from about 10 percent in low-density subdivisions to over 50 percent in multi-family communities.

Each sub catchment area was evaluated for impervious levels by reviewing aerial photography (Google Earth Pro – Image Date 2013) and assigning an impervious percentage. For comparison, the Wellington Street Corridor from Carmen’s Way to John Street is heavily urbanized and nearly completely impervious, whereas the impervious levels of the catchment area draining to the Fort Creek reservoir are very low. The residential area immediately north of Steelton has a higher than typical published imperviousness levels due to the small size of lots and vintage of the development, whereas the Grand Avenue area residential district has imperviousness levels nearer to published typical values. Figure 4 presents a graphical depiction of the distribution of impervious levels by percentage in the watershed.



Sub-Catchment Imperviousness

FIG. 4

2.1.7 Depression Storage

The amount of direct runoff from a storm depends largely on the losses, or the abstractions, caused by infiltration, depression storage and evaporation. These losses depend upon soil type, type of vegetation and amount of impervious cover.

Before overland flow begins or during its early stages, a small portion of the initial rainfall is stored and permanently extracted from surface runoff by interception and surface or depression storage. The interception evaporates and depression storage either evaporates or infiltrates after the rainfall. The interception is subtracted from the beginning of the rainfall, whereas depression storage accumulates only after the rain intensity exceeds the infiltration capacity.

The pervious area depression storage (IA) for each watershed was calculated for each sub-watershed based on its corresponding CN value by the following relations.

$S = \left(\frac{2\ ,4}{C}\right) - 254$ (Metric)

CN ≤70 ; IA = 0.075S

70 < CN ≤ 80 ; IA = 0.10S

80 < CN ≤ 90 ; IA = 0.15S

CN > 90 ; IA = 0.2S

Pervious area depression storage therefore ranged from 7.6 to 28.6 mm.

For impervious area depression storage, a typical published value of 3 mm was used.

2.2 Design Storms and Climate Change

A prerequisite prior to the hydraulic evaluation of both the minor and major drainage systems is the development of runoff hydrographs from catchment areas. Runoff hydrographs were developed for storms with return periods and durations of 1Hr – 1:10 Yr, 1Hr – 1:100 Yr, 24Hr – 1:100Yr and the Regional Storm (Timmins Storm).

A minor storm is defined as a storm typically used for design of the minor storm drainage system, typically a 1 in 10 year return period storm (10% probability of being equaled or exceeded in any given year).

A major storm is defined as a storm used for design of the major storm drainage system. The frequency of such a storm is usually 1 in 100 years (1% probability of being equaled or exceeded in any given year).

2.2.1 Synthetic Design Storms

Synthetic design storm hyetographs are intended to represent some of the long-term statistical properties of recorded rainfall. A number of approaches specify the distribution of rainfall intensity over a specific duration during these design rainfall events. Selected examples of synthetic design storm distributions are provided in Table 3. Return period rainfall amounts for 1 hour, 12 hour and 24 hour events are provided in Table 4, based on the Atmospheric Environmental Service rainfall records from the Sault Ste. Marie Airport.

i. Atmospheric Environment Services (AES) Type 2 distributions. This distribution was developed based on the analysis of one-hour duration rainfall data for different regions of Canada.
ii. SCS Type 2 distributions. Similar to the AES Type 2 distributions, the SCS Type 2 distribution is based on 6, 12, 24 and 48 hour duration rainfall data.
iii. Regulatory Storm events are events that have been selected as the approved standard(s) to be used in particular watershed(s) to define the limits of the flood plain for regulatory purposes. The Timmins Storm, which occurred on August 31, 1961 and into September 1, 1961 is a 12 hour storm with 193 mm of rainfall and was selected to be used for regulatory purposes in North and Central Ontario.

Table 3: Design Storms

RAINFALL DISTRIBUTIONS (PERCENTAGE)														
Type	**Reference**	**Storm Duration**	**INCREMENTS**											
			1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**
Timmins Regional Storm	Ministry of Natural Resources	12 Hour (1 H .Ir)	8	10	6	1	3	10	23	10	12	6	7	4
Return Period Storms	SCS Type 2	24 Hour (2 H .Ir)	2	3	3	4	6	48	16	6	4	3	3	2
	AES Type 2 Northern Ontario	1 Hour (5 Min. Increments)	3	5	7	10	14	10	12	11	9	9	5	5

Table 4: Return Period Design Storm Rainfall Amounts

Return Period (yr)	Recurrence Probability per Year (%)	1 hr Duration Total Rainfall (mm)	12 hr Duration Total Rainfall (mm)	24 hr Duration Total Rainfall (mm)
10	10%	32.9	64.4	74.4
100	1%	49.4	94.3	109.4
Regional Event	-	-	193	-

2.2.2 Climate Change

An increase in extreme weather events may result in more frequent flooding, erosion, shoreline damage, infrastructure failures and decreased water quality due to increased runoff and debris. In accordance with the City of Sault Ste. Marie Stormwater Management Investigative Study – Appendix K, Stormwater Management Guidelines Section 5.1.4, "In the City Sault Ste. Marie, the major storm system has typically been designed to accommodate the runoff produced by a 100 year return period storm event and/or the Regional design storm – The Timmins Storm. Due to uncertainty surrounding the effects of climate change, the City of Sault Ste. Marie will be more frequently updating the rainfall data (intensity-duration-frequency) used to establish municipal design standards". Therefore, the study recommends that the City continue to design the major drainage system based on the 100-year return period storm as well as the Timmins Storm.

The generation of extreme values by simulations is not something that receives any explicit treatment in the formulation of climate models. Furthermore, the performance of climate models is not evaluated on the characteristics of its extreme events, but rather on the basis of aggregated average values. For those two reasons, the analysis of products derived from simulated extremes, such as IDF curves, are likely to be far more uncertain than other climatic indicators based on averaged values. However, in accordance with the terms of reference for the hydrologic analysis of the drainage system (reference RFP document), Tulloch Engineering has chosen to use the IDF curve multipliers in accordance with the report 'The Resilience of Ontario Highway Drainage Infrastructure to Climate Change', as they pertain to the local IDF curve for Sault Ste. Marie. This approach is deemed validated as it is an approach used by the Ministry of Transportation of Ontario (MTO) and provides the most 'local' detailed data as could be found in our limited research.

Generally summarizing the recommendations of the report, rainfall amounts for Sault Ste. Marie (Table 5) for various return periods and durations are increased by the recommended percentages (Table 6) based on climate model predictions to give the recommended future rainfall/IDF curves design values presented in Table 7. It should be noted that the climate change rainfall modeling results discussed in this section and within the referenced study should not be viewed as definitive results of future climate predictions. They are only a presentation of the specific analysis of a limited number of studies and climate change models. Further analysis and research using other model ensembles and climate data trend analysis is needed to be undertaken to develop a better understanding of the magnitude of future changes in precipitation.

A copy of the MTO report can be viewed at:
https://www.library.mto.gov.on.ca/SydneyPLUS/Sydney/Portal/default.aspx?lang=en-US

Table 5: Current SSM Rainfall Amounts

Current Sault Ste. Marie Return Period Rainfall Amounts (mm)									
2010									
Main	**5_min**	**10_min**	**15_min**	**30_min**	**1_hr**	**2_hr**	**6_hr**	**12_hr**	**24_hr**
2_yrs	7.6	10.8	12.8	16	19.8	24.8	35	40.5	46.8
5_yrs	10.1	14.2	17.1	22.6	27.7	34.1	49	54.9	63.5
10_yrs	11.7	16.6	20	27	32.9	40.3	58.3	64.4	74.7
25_yrs	13.7	19.5	23.7	32.6	39.6	48.1	70	76.5	88.7
50_yrs	15.2	21.6	26.4	36.7	44.5	53.8	78.7	85.5	99.1
100_yrs	16.7	23.8	29	40.8	49.4	59.6	87.3	94.3	109.4

Table 6: Climate Change Prediction Adjustment Factors

Climate Change Predictions from the Climate Change Data Portal Sault Ste Marie 90% comparison (Base Case 2007 Extrapolation)									
2065-2095									
Main	**5_min**	**10_min**	**15_min**	**30_min**	**1_hr**	**2_hr**	**6_hr**	**12_hr**	**24_hr**
2_yrs	12%	12%	12%	12%	12%	8%	2%	4%	3%
5_yrs	12%	12%	12%	12%	12%	8%	3%	4%	2%
10_yrs	12%	12%	12%	12%	12%	9%	3%	4%	1%
25_yrs	12%	12%	12%	12%	12%	9%	4%	4%	0%
50_yrs	11%	11%	11%	11%	11%	9%	5%	4%	0%
100_yrs	11%	11%	11%	11%	11%	9%	5%	4%	-1%

Table 7: Adjusted SSM Rainfall Amounts

Adjusted Sault Ste. Marie Return Period Rainfall Amounts (mm) due to Climate change									
Main	**5_min**	**10_min**	**15_min**	**30_min**	**1_hr**	**2_hr**	**6_hr**	**12_hr**	**24_hr**
2_yrs	8.5	12.1	14.3	17.9	22.2	26.8	35.7	42.1	48.2
5_yrs	11.3	15.9	19.2	25.3	31	36.8	50.5	57.1	64.8
10_yrs	13.1	18.6	22.4	30.2	36.8	43.9	60	67	75.4
25_yrs	15.3	21.8	26.5	36.5	44.4	52.4	72.8	79.6	88.7
50_yrs	16.9	24	29.3	40.7	49.4	58.6	82.6	88.9	99.1
100_yrs	18.5	26.4	32.2	45.3	54.8	65	91.7	98.1	108.3

** Obtained from Appendix 'A-4' Climate Change Predictions from Climate Change Data Portal (CCDP) – The Resilience of Ontario Highway Drainage Infrastructure to Climate Change.

Since the intent of the Regional Storm concept is a worst case flood design criteria, it is our opinion that adding further factors of safety is an impractical approach and therefore yields excessive flows which cannot be accommodated in a feasible manner. Therefore, the rainfall intensities and distributions of the Regional Storm event have not been adjusted per the above climate change models.

2.3 Present and Future Land Use

Throughout the modeled scenarios presented later in the report, no increase in impervious percentage or other parameters associated with future development has been accounted for. It has been assumed that the lower watershed south of Second Line will see little in the way of increased imperviousness due to the fact the area is already heavily urbanized. Further, the City of Sault Ste. Marie, through its planning policies has initiated a hard scape softening trend, whereby heavily urbanized areas with large impervious landscapes are encouraged to be softened through green landscape features as roadways are reconstructed and properties redeveloped. This trend could result in, albeit possibly negligible, a lower impervious percentage and lower rate of runoff.

For the portion of the watershed north of Second Line, again no allowance for the potential increase in impervious percentage or other parameters associated with future development has been provided. This is consistent with the City of Sault Ste. Marie Storm Water Management Policy of limiting post development peak runoff flows to predevelopment levels.

The future development of areas draining to the Fort Creek Reservoir has again not been provided with any adjustment to account for future development for two reasons. Firstly, the City of Sault Ste. Marie Stormwater Management Policy would again apply and limit post development flows to predevelopment levels. Secondly, the functionality of the reservoir would apply whereby it would be highly unlikely that any small increase to flow rates entering the reservoir would affect the peak outflow from the dam due to the attenuating effect of the dam and reservoir storage. The functionality and effects of the dam are discussed in Section 1.6.

2.4 Watershed Topography

TULLOCH Engineering has relied on the 1997 topographical information provided by the City of Sault Ste. Marie Geographical Information System (GIS) to generate the watershed and sub-watershed areas, and subsequent results presented later in this report. The information is considered accurate to +/- 0.5m and has the potential to significantly impact the accuracy of the findings.

The Fort Creek catchment area below the Fort Creek dam contains landforms which substantially affect both the minor and major drainage systems. First, the watershed is bisected by an escarpment transversing generally north/south through the lower watershed. Figure 5 depicts the dominant landforms. The escarpment presents a change in elevation from approximately ~220 m to ~190 m. The highest part of the watershed (~245.5 m) is located just west of the intersection of Great Northern Road and Second Line (approximately 2.85 km as the crow flies away from the aqueduct inlet at Carmen's Way opposite Bloor Street). The escarpment is marked by gullies which provide relief used by both the minor system for outlets and also the major system as natural drainage paths to the lower level of the escarpment.

Based on a review of the present day contours and contours presented in past Proctor and Redfern reports, the original Fort Creek Channel alignment prior to urbanization is believed to be located in the area of Northland Road at Conmee Ave and appears to have traversed southerly creeping easterly toward Parliament Street at Bloor Street. The topography would suggest the creek then took a southerly turn crossing Northland Road and St. Georges Avenue to the west of John Street and linking up with the open Channel at the CPR crossing which is believed to be in its original location.

North of the Canadian Pacific Railway Crossing (CPR), the topography causes the overland flow to concentrate at St. Georges Avenue and John Street. Of particular note, the natural fall of the land is away from the Carmen's Way open channel. The Dillon Report of 1977 developed flood plain mapping which would result in the event the Fort Creek open channel along the former Hudson Street alignment (generally follows Carmen's Way in this area) were to overtop, either due to channel capacity or crossing structure(s) restrictions. A copy of this mapping is provided in Figure 6 in Appendix A. Further identified within this figure is that the CPR rail spur has caused an obstruction/barrier to the overland flow route between the John St/St. Georges intersection to John St and Edinburgh St. The result of which causes the overland flow to flood this area, until the minor system can convey this volume to the Wellington Street aqueduct where it empties to the open channel and masonry arch CPR crossing before it re-enters the underground aqueduct – the John St laneway aqueduct.

The topographical obstruction caused by the CPR rail line bisects the entire lower watershed in an east west direction. The overland flows as a result are forced towards the John Street/St. Georges Ave intersection and the masonry arch CPR crossing and open channel at Wellington Street West and John Street. During surcharging of the minor system in the area north of the CPR line, overland flow would be expected to concentrate and flow toward the location of the existing channel as the topographic mapping suggests. West of the existing channel alignment, this overland flow is away from the constructed Fort Creek open channel along the west side of Carmen's Way and thus would not intercept the Wellington Street Aqueduct. Similarly, surface flow from surcharged conditions of the minor system to the east of the suspected old channel alignment would again tend to concentrate and converge at the intersection of St. Georges Avenue and John Street and the masonry arch CPR crossing and open channel at Wellington Street West and John Street. Therefore, the overland major drainage system south of Second Line does not have the opportunity to enter the aqueduct until the minor system has capacity to inlet and convey these flows, and the point of entrance of the majority this volume of water would be the John Street area south of the intersection with St. Georges Avenue.

At the CPR crossing, the existing channel is generally noted in its original location. From this point the topography would suggest an essentially due south direction down John Street, however; as the existing aqueduct is located just east of John Street, it is likely this is the location of the former main channel. A generalized historical map of the area in 1903 was found in the City of Sault Ste. Marie Museum on-line archives, a copy of which is provided as Figure 7 in Appendix A. The historical map depicts the original Fort Creek traversing north to south along Parliament Street.

Work completed (or planned) to the John Street Aqueduct and relief Aqueduct has and/or will raise the grade of John Street between Queen Street and Cathcart Street by varying amounts. This could negatively affect the major drainage system as the topography of the lower watershed south of the CPR rail line would suggest the major drainage system concentrates flows towards John Street with John Street being a major drainage path conveying runoff southerly towards the Queen Street open channel.

The Wellington Street underpass is so named as it is a small section of Wellington Street immediately west of Carmen's Way where the roadway dips under two CPR rail spurs supported by a steel and concrete bridge structure. During construction of Carmen's Way in 2005, an abandoned sanitary sewer was converted to a storm sewer which accepts the drainage within the Wellington Street Underpass roadway depression. This repurposed sewer drains easterly to Carmen's Way and then south along Carmen's Way and more specifically near the old Hudson Street right of way centerline. This drainage characteristic is relevant as this repurposed sewer does not drain to the Fort Creek Aqueduct major system until it outlets just south of the Bay Street culverts in the open channel.

Two scenarios in consideration of the above observation can have a significant impact on design flows within the Aqueduct system. Firstly, it has been observed that debris can build up and impact the inlet capabilities of the Aqueduct on Carmen's Way. There are reported occurrences that the inlet has surcharged and spilled its banks (either due to debris blockage or high flows). The topography of this area directs flow a short distance towards Carmen's Way where it is conveyed southerly to the intersection of Wellington Street. From the intersection, the flow is directed westerly into the vertical roadway sag within the underpass. It can thus be concluded that due to the only available drainage outlet within the underpass being the repurposed sewer, should the inlet of the aqueduct surcharge, this volume of water would be lost from the Aqueduct catchment area and lost from contributing flows to the aqueduct. Secondly, due to the topography of the rail yard and rail line to the west of Carmen's Way and the general direction of surface drainage, any surcharging of the minor system and overland flow to the west of the rail yard and rail line, specifically in the Wellington Street area north of Lyons Ave will runoff into the Wellington Street Underpass and is prevented from flowing easterly and reaching Carmen's Way. As above, this volume of water would be lost from contributing to the Aqueduct.

3 HYDROLOGIC MODELING AND VERIFICATION

3.1 Hydrologic Computer Modeling

Computer models of existing conditions and potential mitigation options were generated using the PCSWMM software program. PCSWMM Version 5.1.910 (SWMM5) was utilized to model both the hydrological runoff and hydraulics of the minor and major systems. PCSWMM provides modeling of flood depths, flows and velocities for both urban and rural applications, including river-based flooding, major/minor (dual drainage) modeling, or simply overland routing of rainfall runoff. Sources of surface flows can be overbank flooding from rivers/streams, flooding from the minor system, and/or distributed rainfall.

Of the various flow and flood routing methods, the Dynamic Wave calculation method was selected. The Dynamic Wave formulation is a widely used approach for the production of the characteristics of flow propagation. While complex, all of the terms of the momentum equation are considered in the model.

PCSWMM is distributed by Computations Hydraulics Inc. (CHI). The model was constructed to analyze both the minor/major drainage (dual drainage) system simultaneously. The model analyses the existing rural and urban drainage systems, where surface flows are routed over the land surface (major system, such as roads, swales, street sags, and storage areas) while part of the runoff simultaneously flows in the underground conveyances (minor system, such as sewer pipes and associated infrastructure). The application used the dynamic wave analysis method allowing it to calculate backwater, reverse pipe flow, and generation of unit hydrographs.

General details of the model include;

Number of subcatchments:	699
Number of nodes:	1696
Number of links:	2677

Analysis Options

Flow Units:	CMS
Infiltration Method:	CURVE_NUMBER
Flow Routing Method:	DYNAMIC _WAVE
Antecedent Dry Days:	0.0
Wet Time Step:	00:01:00
Dry Time Step:	00:01:00
Routing Time Step:	0.50 sec
Head Tolerance	0.004921 m

Process Models

Rainfall/Runoff:	YES
RDII:	NO
Snowmelt:	NO
Groundwater:	NO
Flow Routing:	YES
Ponding Allowed:	NO
Water Quality:	NO

The model achieved a Runoff Quantity Continuity Error of -0.044% and a Flow Routing Continuity Error of -0.095% well within acceptable limits of continuity.

3.2 Assumptions

The following assumptions were made in addition to the aforementioned parameters discussed in the previous sections;

i. The maximum monthly mean level of the St. Marys River downstream of the Rapids is 177.96 m (Canadian Geodetic Datum). This information was obtained from Environment Canada's Water Office of the Department of Fisheries and Oceans (https://wateroffice.ec.gc.ca) using Station: 02CA005. The maximum monthly mean elevation of the St. Marys River was used in all scenarios with the exception of the existing conditions model.

ii. Sewer data provided by the City of Sault Ste. Marie was assumed to be accurate with respect to pipe sizing, lengths, inverts and slopes and topographic elevations were assumed accurate, with the exception of the following;
   - Missing Inverts were calculated using the closest upstream conduit slope,
   - Missing storm sewer characteristics were assumed to be the same as the nearest upstream sewer,
   - Pipes, inlets and outlets were assumed to be free of debris and obstructions.
   - The roof of the aqueduct was modeled to be the lowest elevation of the roof beams.

iii. The Fort Creek Aqueduct between John Street and Cathcart Street was modeled with uniform cross sections. Cross sections were increased (or decreased) as deemed appropriate due to its gradually changing cross section.

iv. A coefficient of 0.5 was used for all entrance and exit losses.

v. A 450 mm dia. Storm sewer was added to drain the Wellington Street Underpass to the appropriate outfall location. This was assumed in order to maintain and gauge model continuity and to improve the water quantity mass balance.

vi. The five (5) Bay Street culverts and the channel south of Bay Street discharging to the St. Marys River have been modeled partially filled with sediment. As noted later in the report, the recommendation to remove the sediment and accordingly perform regular maintenance is suggested; however, it is inevitable that due to the level of the St. Marys River and the invert of the channel that sediment buildup will occur and it is unknown and cannot be predicted when maintenance activities will or can take place. For this reason, we have taken the approach to assume a level of sediment within the channel and culverts and hydraulically analyze and design for this condition.

vii. Sanitary obstructions within the existing aqueduct were modeled as weirs as they protrude into the cross section.

viii. There are three (3) water crossings in close proximity to each other near Bay Street, namely; Algoma Central Railroad (ACR) and service road, the Bay Street Bridge and the private bridge between the OLG Casino center and parking lot. The five culverts allowing crossing of the ACR and service road were used in the model as this element was the most flow restrictive. Further, a sheet pile wall extends over the length on either side of the creek from Queen Street to the southern edge of the Bay Street Bridge (forms part of the abutments) allowing these upstream crossings to be modeled as an open channel.

ix. Climate change prediction multipliers were not used in the analysis of existing conditions.

x. Climate change prediction multipliers were used for return period storm events only, in the analysis of alternative models.

xi. The major drainage system was modeled using three (3) typical transect cross sections; 1) a typical 10 m back to back of curb distance asphalt road cross section within a 20 m roadway allowance comprised of grassed overbanks, 2) an overland asphalt parking lot cross section with curbs, and 3) an overland grass cross section.

xii. The model incorporated a dual drainage system analysis. The minor system is attached to the major system. It was assumed that there was no inlet or other restrictions into the minor system. The catchment area drains to the major system node where it is allowed to enter the minor system. If the minor system is surcharged to the rim elevation then the runoff is conveyed overland through one of the three (3) typical major drainage system cross sections re-entering the minor system on its downstream path where/if capacity exists.

3.3 Summary of Findings - Existing Conditions

The findings presented are not intended to provide a complete listing of all flooded or surcharged areas, and is intended to present problematic areas within the drainage system and within the scope of this study only.

3.3.1 Observations due to a 1 Hour 1:10 Year Return Period Storm Event

- The hydraulic grade line of the minor drainage system can be expected to come to surface and in some cases localized flooding can be expected. Figure 8 presents the expected locations in graphical form.
- The Major drainage system is directed to John Street and impeded by the CPR Railway Crossing. Localized flooding in this area can be expected. The minor system on St. Georges Ave. from Wellington Street to Bush Street appears to be the restriction causing surcharging of the hydraulic grade line to the surface in this area.
- Flooding of the Wellington Street Underpass can be expected.
- The minor drainage system surcharges on Albert Street west of John Street adjacent to the former Etienne Brule Public School. The major overland drainage path for this area is through the school property on the west side of the building to Queen Street at the SSMOLG Casino entrance. Localized ponding of water in this area can be expected.

3.3.2 Observations due to the 1hr 1:100 Year Storm Event

- The elevation within the Fort Creek Reservoir reaches 200.05m (656.33ft) subjected to a 1:100 year 1 hour storm event. Peak outflow from the dam at a 10% sluice gate opening is 0.419 m^3/s.
- The Fort Creek system from the Dam to the outfall to the St. Marys River does not surcharge to ground level.
- Pipe surcharging occurs in the laneway paralleling John Street just south of the crossing of John Street to Cathcart Street and between Albert Street and Queen Street.
- Localized Flooding can be expected on John Street with the intersection of St. Georges Avenue south to the CPR Crossing and on St. Andrews Terrace east of John Street.
- The intersection of Charles Street and Northland Road can be expected to see localized flooding above 0.2m depth as measured on the roadway.
- Severe flooding of the Wellington Street Underpass can be expected.
- The hydraulic grade line rises above pipe obvert in the 4 m by 1 m concrete box crossing Carmen's Way just north of the intersection of Wellington Street. The hydraulic grade line does not reach the surface.

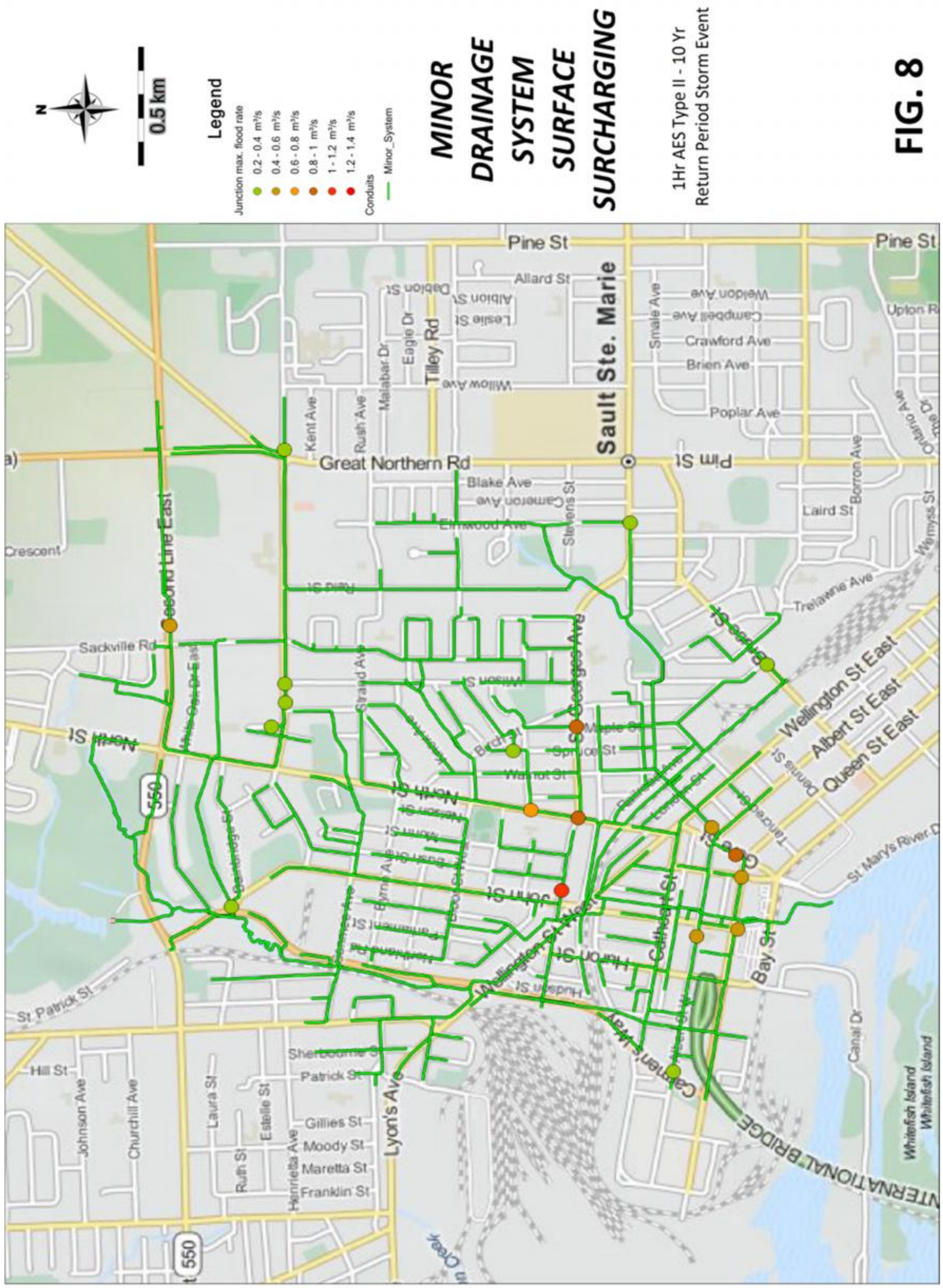

0.5 km
Legend
Junction max. flood rate
0.2 - 0.4 m³/s
0.4 - 0.6 m³/s
0.6 - 0.8 m³/s
0.8 - 1 m³/s
1 - 1.2 m³/s
1.2 - 1.4 m³/s
Conduits
Minor_System
MINOR
DRAINAGE
SYSTEM
SURFACE
SURCHARGING
1Hr AES Type II - 10 Yr
Return Period Storm Event
Sault Ste. Marie
Pine St
Great Northern Rd
Second Line East
North St
Wellington St East
Albert St East
Queen St East
Bay St
John St
Pim St
Lyon's Ave
St Patrick St
INTERNATIONAL BRIDGE
Whitefish Island

FIG. 8

3.3.3 Observations due to the 24hr 1:100 Year Storm Event

- The elevation within the Fort Creek Reservoir reaches 201.46 (660.96ft) subjected to a 1:100 year 24 hour storm event. Peak outflow from the dam at a 10% sluice gate opening is 0.556m³/s.
- The Fort Creek System surcharges to surface or spills its channel banks in two (2) locations during the 24 hour rainfall event; 1) Aqueduct Inlet opposite Bloor Street on Carmen's Way, and 2) within the laneway paralleling John Street just north of Cathcart Street. Localized flooding can be expected in these areas.
- Pipe surcharging occurs in the laneway paralleling John Street between Alexandra Street and Queen Street. Localized flooding can be expected in this area.
- The Conmee Avenue Crossing surcharges above the pipe obvert.
- The rail spur crossing Carmen's Way into the former Algoma Steel Welded Beam facility surcharges above the pipe obvert.
- Severe flooding of the Wellington Street Underpass can be expected.
- The intersection of Northland Road and Kehoe Ave can be expected to see localized flooding above 0.2m depth as measured on the roadway.
- Ponding up to depths of 0.33m could occur immediately south of Queen Street within the Parking Lot and throughway of the OLGSSM Casino.
- The Major drainage system is directed to John Street and impeded by the CPR Railway Crossing. Surface water depths in this area are estimated to reach 0.32m on John Street with greater depths of up to 0.63m on St. Andrews Terrace east of John Street.
- The intersection of Charles Street and Northland Road can be expected to see localized flooding above 0.25m depth as measured on the roadway.
- The intersection of Northland Road and Kehoe Ave can be expected to see localized flooding above 0.2m depth as measured on the roadway.

3.3.4 Observations due to the Regional Timmins Storm Event

- The elevation within the Fort Creek Reservoir reaches 205.50m (674.21ft) with a maximum outflow of 10.993 m³/s when subjected to the Regional Storm event. An elevation of 205.13m (673ft) is sufficient to overtop the hexagonal weir, but does not overtop the dam's erodible plugs. This correlates well to the Proctor and Redfern conclusions of obtaining a water level within the reservoir of 205.26m (673.43ft) with a 10% sluice gate opening setting.
- The Fort Creek System surcharges to surface or spills its channel banks in three (3) locations during the regional rainfall event; 1) rail spur crossing Carmen's Way into the former Algoma Steel Welded Beam facility, 2) Aqueduct Inlet opposite Bloor Street on Carmen's Way, and 3) within the laneway paralleling John Street just north of Cathcart Street. Localized flooding can be expected in these areas.
- The Major drainage system is directed to John Street and impeded by the CPR Railway Crossing. Surface water depths in this area are estimated to reach 0.26m on John Street with greater depths of up to 0.58m on St. Andrews Terrace east of John Street.
- Ponding up to depths of 0.27m could occur immediately south of Queen Street within the Parking Lot and throughway of the OLGSSM Casino.
- The intersection of Charles Street and Northland Road can be expected to see localized flooding above 0.2m depth as measured on the roadway.
- Severe flooding of the Wellington Street Underpass can be expected.
- The intersection of Northland Road and Kehoe Ave can be expected to see localized flooding above 0.2m depth as measured on the roadway.

3.3.5 General Observations

The Wellington Street Aqueduct inlet is a 2.75 m wide by 2.02 m high CSP with an inlet capacity of approximately 7.5 m³/s prior to surcharging of the pipe. In accordance with the latest structural condition report prepared by STEM Engineering, the existing inlet is in need of structural repairs. The maximum surcharge height prior to channel overtopping is 0.34 m above pipe obvert, corresponding to a surcharged inlet capacity of approximately 8.1 m^3/s. Problematic debris buildup compounds this inlet restriction, and overtopping of the banks has been reported to frequently occur. The overland flow route is south-bound along Carmen's Way to the intersection with Wellington Street where the flow is directed westerly due to the grading of the roadway towards the Wellington Street underpass.

The Wellington Street underpass is a man-made depression as a result of Wellington Street passing under a multi-rail spur line. The depth of the Wellington Street underpass is approximately 4 m from the lowest elevation point in the vertical sag curve to the lowest point of vertical relief or roadway crest of Wellington Street to the east of the underpass. The Wellington Street underpass is drained via gravity sewer (formerly a sanitary sewer which was converted during the original construction of Carmen's Way) which conveys the storm water easterly to Carmen's Way, then southerly to Queen Street, where eventually it joins with the storm sewer system on Queen Street. This Queen Street storm system discharges to the Fort Creek open channel south of the five (5) parallel culverts south of Bay Street. Allowing surcharging of the aqueduct inlet and allowing the flow to flood the underpass is unacceptable. The assessment of the consequences of this action are beyond the scope of this report, however the alternatives assessed herein do attempt to mitigate this problem to the limited extent of improving the inlet capacity of the aqueduct. As a result, upgrading of the inlet is required structurally, from a debris management and maintenance perspective, and to adequately accept the Fort Creek open channel flow. The drainage patterns to the west of the railway tracks would remain unaltered and would thus still flow to the Underpass area.

Within the lower watershed, both north and south of the CPR Railway crossing (John Street and Wellington Street), the major overland flows tend to converge along the path of the original Fort Creek and collect in the area of St. Georges Avenue, John Street and St. Andrews Terrace. See Figure 9.

The major drainage system along Second Line conveys a significant volume of water from east to west, reaching depths that indicate localized flooding could be occurring from approximately 465 Second Line East to generally the rail crossing of Second Line just to the west of the Fort Creek Conservation Area.

Sanitary sewer restrictions within the aqueduct, siltation in the open channel south of Bay Street and the sediment build-up within the five (5) parallel culverts south of Queen are major contributors to increases in the hydraulic grade line.

3.4 Comparison of Results with Past Reports

Table 8 provided in Appendix B presents computed flows and reservoir characteristics as compared to past reports. In comparison to past reports, it is noted that the results obtained for the various storm events analyzed for the existing conditions model yielded the lowest flows of all the past reports. A number of generalized factors could explain the reason(s) for this;

1) Changes in the infrastructure over the years could have an effect on the runoff hydrographs and flow rates within the minor system and overland flow routes.
2) Advancement in hydrological analysis theory coupled with the advancement in technology and computational power of computers to perform more complex analysis of watersheds and hydraulic analysis of drainage systems. i.e. HYMO vs. SWMM
3) Method of calculating rainfall losses i.e. SCS/Green-Ampt/Horton.
4) Differing assumptions made in each model.

More specifically with respect to Item No. 2, the 1984 and 1988 Proctor and Redfern, and 1977 Dillon reports idealized the runoff and did not separate the major system and the minor system flows. The reports used early versions of the HYMO model which generalized the watershed into six (6) subwatersheds and routed flows in a cascading effect into each other. **HYMO** (HYdrologic MOdel) developed in 1972 by the United States Soil Conservation Service (SCS) is now rarely used in its original form. **OTTHYMO** (OTTawa HYdrological MOdel) exists today in many versions, however in the 1980's two versions (1984 and 1989) were commonly used. The latter being the most popular in Canada for a number of years thereafter, as engineers appreciated its capacity to simulate drainage of rural watersheds. Current versions can also simulate major and minor urban drainage networks as well as simple flood routing in sewer pipes and pipe networks.

This problem-oriented computer language provides commands, commonly used in hydrology, to transform rainfall into runoff hydrographs and to route these hydrographs through streams and valleys or reservoirs. Routing through pipe networks was not an option in the original versions. While routing of streams etc. does occur, the lagging and storage effect due to flow restrictions of crossings and the dual drainage system may not be adequately captured. We also point out, these reports were prepared when computers were generally first gaining popularity and sufficient computing power to analyze complex hydrological models had likely not yet been employed.

HYMO models are sensitive to percentage imperviousness and directly connected imperviousness. Matching urban hydrographs (high intensities and imperviousness) with SWMM models is generally achievable but matching runoff from low imperviousness areas is where differences between the models are amplified. The urban hydrology components in HYMO models tend to give similar results to those from SWMM, however; the comparison of rural catchments using HYMO has always been in question as the original HYMO manual equations only reflect southern US watersheds.

There is a second and probably more important difference between the two methods which concerns the modeling of rainfall losses.

The SWMM Runoff block uses a surface water budget approach in which the idealized plane surface receives rainfall as input and from which the output is a combination of infiltration and runoff. The (OTT)HYMO model uses the concept of effective rainfall which is calculated separately before the convolution process. Using the notion of effective rainfall, no infiltration can occur after cessation of rainfall, whereas with the surface water budget, infiltration can continue after the end of incident rain as long as there is a finite depth of flow on the surface. If this is converted to an equivalent effective rainfall hyetograph, it is not uncommon to get what appears to be "negative" effective rainfall intensity towards the end of the storm when intensity is less than the infiltration capacity and the surface depth is finite.

This can be particularly evident if a synthetic storm like a SCS hyetograph is used with an infiltration model such as Horton or Green & Ampt. This can cause significant differences in both the peak runoff and runoff volume as computed by the two methods. The observation that HYMO results and SWMM results do not match should not be a surprise.

Tulloch Engineering is therefore of the opinion, the broad brush watershed HYMO analysis used in 1988 and previous years reports, while widely used and accepted at the time, were simplistic models that did not accurately capture the lagging and storage effects of the watersheds drainage systems and resulted in overestimation of peak flows. Further, the one dimensional models and the development of the subwatersheds did not account for the topographical innuendos, specifically; that major drainage system flows away from the Fort Creek Channel and Wellington Street Aqueduct only to be concentrated at John Street and St. Georges Avenue, and the damming effect of the CPR railway.

Our review of the most recent report prepared by Conestoga Rovers and Associates (CRA) (2014) noted that an assumption was made similar to the 1977 M.M. Dillon Ltd. report which was adopted into the 1984 Proctor and Redfern Ltd. report. This assumption allotted outflows (8.5 m^3/s and 15.57 m^3/s)[1] from the Fort Creek Dam which required that the reservoir was partially full immediately before occurrence of the Regional Storm or major rainfall event. Section 1.6 above describes the functionality and operation of the Dam and Reservoir in detail. Proctor and Redfern Ltd. in its 1988 report subsequently discussed and dismissed this assumption as being overly conservative. If this assumption were to be accepted, then one of three possible scenarios must be assumed to have occurred immediately prior to the advent of a Regional Storm.

1 Table 4 - Proctor and Redfern Ltd., E.O. 84521 - November 1984

i. The reservoir level must have been purposely raised to a high level by closing the low flow conduit sluice gate;
ii. A major storm must have occurred immediately prior to the Regional Storm;
iii. The low flow conduit inlet must have been plugged for a long period of time.

We agree with the argument set forth in Proctor and Redfern's 1988 report and therefore again hold the following statement in accord, "Since the intent of the Regional Storm concept is a worst case flood design criteria, it is our considered opinion that adding further factors of safety is an impractical approach and yields excessive flows which cannot be accommodated in a feasible manner." Thus, our hydrologic model and the initial conditions utilized for analysis of the minor and major drainage systems assumes that the reservoir is not partially full and that its elevation is at normal level 198.12m (650ft) at the advent of the Regional Storm or other major storm event.

The CRA model also appears to have used unrealistically low imperviousness levels for sub catchment areas. The low imperviousness levels would in effect somewhat counteract the reservoir outflow assumptions.

3.5 Model Verification and Model Calibration

Tulloch Engineering was provided with rainfall data from three (3) rain gauges and six (6) flow gauges identified and located as follows;

i. PCM Unit 1 – (Rain Gauge) 99 Foster Drive (Civic Center)
ii. PCM Unit 2 – (Flow Gauge) Main Aqueduct on Queen Street
iii. PCM Unit 3 – (Flow Gauge) 1500mm Concrete Pipe at Wellington and John St. (just east of Wardlaw Fuel on Wellington St.)
iv. PCM Unit 4 – (Flow Gauge) 1200mm Concrete Pipe at St. Andrew's Terrace east of John St. (east of HR Lash)
v. PCM Unit 5 – (Rain Gauge) 71 White Oak Drive
vi. Qeye Unit 1 – (Flow Gauge) 1200 mm Concrete Pipe on North Street, south of White Oak Dr. and north of Bainbridge St.
vii. Qeye Unit 2 – (Flow Gauge) Main Aqueduct east of John St.
viii. Qeye Unit 3 – (Flow Gauge) Main Aqueduct, Wellington St. west of John St at Paul's Bakery.

Prior to utilizing the measured flow and rainfall data, Tulloch Engineering performed a verification of the data and determined much of the data was unreliable due to data gaps, missing parameters within whole data sets and generally data which is highly variable with suspected errors in the recorded values.

The measured rainfall data for August 16th, 2012 corresponding to the largest rainfall event of the season was found to be complete and was generally consistent between rain gauges PCM Unit 1 and PCM Unit 5. The data from PCM Unit 5 was selected for hydrologic analysis due to its central position within the watershed and hydraulically routed through the existing conditions model. Recorded flow data as provided to Tulloch Engineering by the City of Sault Ste. Marie was compared to the prediction model. Recorded flow data for Qeye Units 2 and 3 appear whole and reasonable for the storm event.

Predicted flow rates from these two locations were compared to the measured flow rates; 1) Qeye Unit 2 – Main Aqueduct south of the CPR crossing and east of John Street: See Figure 10, and 2) Qeye Unit 3 – Main Aqueduct, Wellington Street, west of John Street in front of Paul's Bakery: See Figure 11.

After analysis of the attempted calibration results, Tulloch Engineering is of the opinion the accuracy of the model could not be verified to an acceptable degree of error by comparing peak flows generated by the model to measured peak runoff flows without reducing impervious levels and CN numbers to unrealistic levels. The uncalibrated theoretical model which utilizes available data over-estimates observed flows by as much as 2 – 2.5 times. In order to calibrate the model to observed data, significant reductions in the imperviousness percentage of sub-catchments and/or a significant reduction to the sub-catchment curve numbers would be required. These reductions would result in highly unrealistic low values.

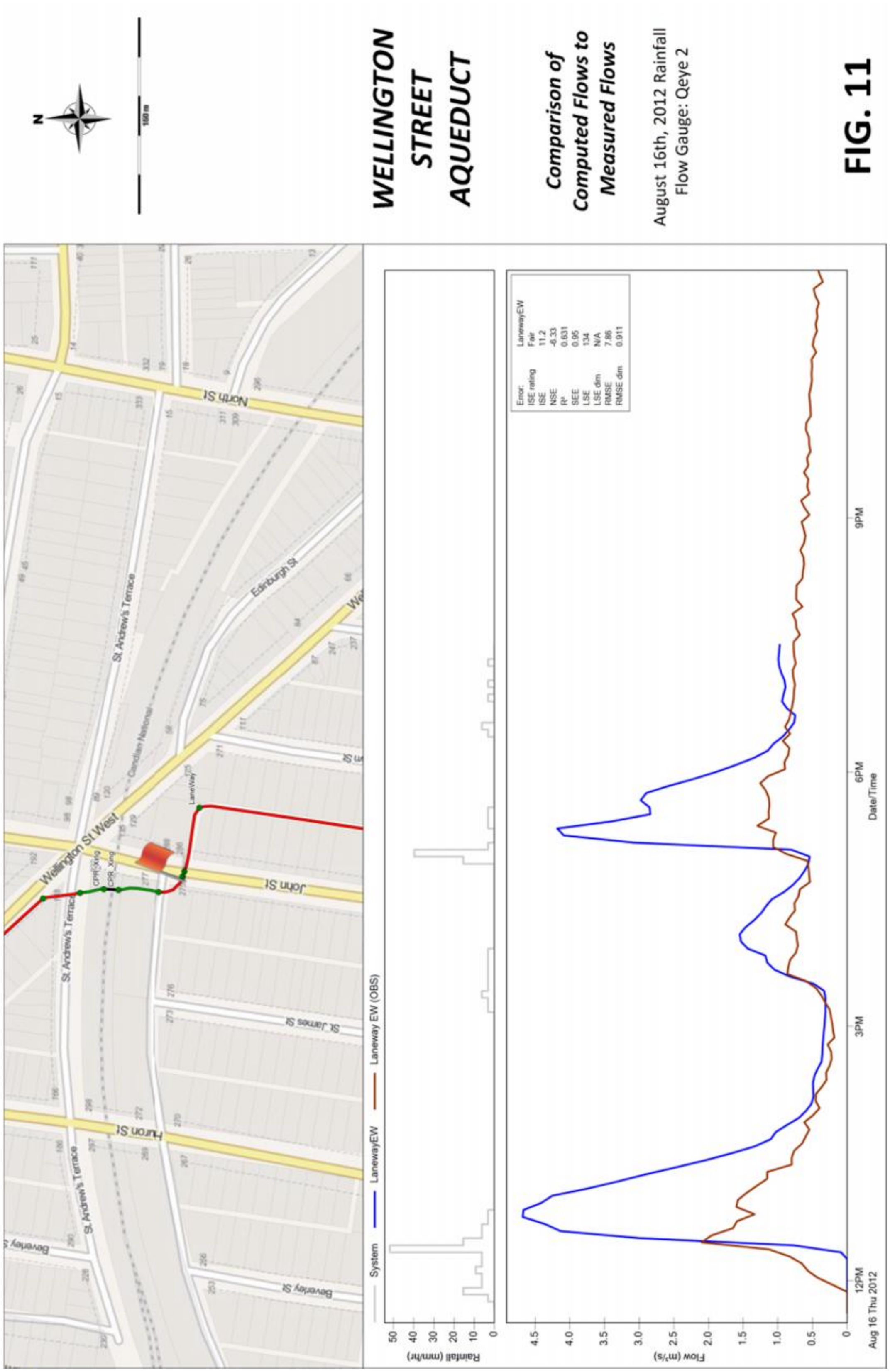
Tulloch Engineering is therefore of the opinion, the theoretical result obtained is both reasonable and acceptably conservative for use in the design and analysis of alternative solutions to the problem defined in Section 1.1. However, dependent on the timing of the project and should sufficient time exist, we would also recommend the City of Sault Ste. Marie continue or recommence a flow monitoring regime such that we may further attempt to calibrate the model and collaborate our results to ensure an acceptably conservative design.

WELLINGTON STREET AQUEDUCT

Comparison of Computed Flows to Measured Flows

August 16th, 2012 Rainfall
Flow Gauge: Qeye 3

FIG. 10


WELLINGTON STREET AQUEDUCT
Comparison of Computed Flows to Measured Flows
August 16th, 2012 Rainfall
Flow Gauge: Qeye 2
FIG. 11
Wellington St West
John St
North St
Huron St
St. James St
St Andrew's Terrace
Edinburgh St
Beverley St
System
LanewayEW
Laneway EW (OBS)
Rainfall (mm/hr)
Flow (m³/s)
Date/Time
12PM
Aug 16 Thu 2012
3PM
6PM
9PM
Error: LanewayEW
ISE rating Fair
ISE 11.2
NSE -6.33
R² 0.631
SEE 0.95
LSE 134
LSE dim N/A
RMSE 7.86
RMSE dim 0.911

4 ALTERNATIVE SOLUTIONS TO THE PROBLEM

The EA parent study to this hydraulic assessment evaluates alternative solutions to the concerns identified in the Problem Statement for the identified Steelton study area.

Hydraulically evaluating all of the alternatives identified in the EA for the study area is impractical, thus only the preferred alternative which has been identified as feasible within the EA parent study is herein hydraulically modeled and evaluated. Outside of the study area, alternative solutions exist to various restrictions to flow elsewhere in the Fort Creek system, which, when corrected will also impact the Steelton area. Therefore, hydraulic alternatives inside the study area and elsewhere are thus considered below.

4.1 Alternative No. 1 – Do Nothing

In accordance with Section 3, the existing conditions model is considered as the 'Do Nothing' alternative. As detailed in Section 3.3, the existing system is hydraulically inadequate to convey the Timmins Regional Storm Event, nor the SCS Type 2 - 24 hour 100 year storm event. The hydraulic grade line of the aqueduct system subjected to an AES Type 2 - 1 hour, 100 year storm event was found not to surcharge to surface, however; flooding in the areas of John Street, St. Georges Avenue and St. Andrews Terrace can be expected. Additionally, the Do Nothing scenario does not address the structural deficiencies or achieve a solution to the problem statement in accordance with the parent Municipal Class Environmental Assessment.

4.1.1 Alternative 1A – Modified Base Line Conditions

Alternative 1A builds on the analysis of the existing conditions model by establishing base design assumptions. The elevation of the St. Marys River is set at 177.34 m Geodetic Survey of Canada which represents the maximum average monthly mean elevation of the lower St. Marys River.

In accordance with Section 2.2, climate change adjustments have been incorporated into return period rainfall events.

Additionally, it is reasonable to assume any reconstruction of the Aqueduct south of the CPR would address the flow restrictions caused by the sanitary sewer crossings which are modeled as weirs in the existing conditions scenario. Therefore, for this scenario and all subsequent scenarios, the sanitary restrictions have been removed.

The analysis of Alternative 1A of the aqueduct system resulted in various hydraulically inadequate sections when subjected to return period rainfall events exceeding and equal to 100 years. As expected, the Timmins Regional storm event produced the most significant aqueduct surcharging and flooding followed by the 24 hour duration SCS Type 2 distribution – 100 year return period rainfall event and the 1 hour duration AES Type 2 distribution 100 year return period rainfall. While the majority of surcharged conduits and flooding occurred in the downstream reaches of the aqueduct in the Central Creek Park, Albert Street, Cathcart Street and laneway east of John Street similar to the 'existing conditions' model, a moderate improvement to the hydraulic gradeline attributable to the removal of the sanitary obstructions is noted, albeit the assumed increase in the level of the St. Marys River somewhat reduces the noted improvements.

4.1.2 Alternative 1B – Modified Base Line Conditions incorporating 2013, 2015 and 2016 Aqueduct Improvements.

Alternative 1B incorporates work completed and currently being planned to the downstream extents of the aqueduct. Work was completed in 2013 and 2015, and work is planned for the upcoming 2016 construction season. Descriptions of the work incorporated into Alternative 1B are as follows:

Description of Contract 2013-4E work:

In 2013 the City of Sault Ste. Marie and their consultant, STEM Engineering tendered for and constructed the replacement of the Aqueduct from its outlet on the south side of Queen Street, the Queen Street crossing, the construction of the main aqueduct northerly through Central park, and the laneway between Central Park Avenue and Albert Street terminating the project extents on the south side of Albert Street. As part of the project was the construction of a 'relief' aqueduct barrel from the outlet to the open channel on the south side of Queen Street, crossing Queen Street before undergoing a change in direction westerly towards John Street within the southerly portion of Central Park.

Description of Contract 2015-4E work:

Contract 2015-4E undertaken in 2015 by the City and STEM Engineering extended the 'relief' aqueduct to John Street and north on John Street to Albert Street. Detailed design drawings, and as built invert data were reviewed and input into the model.

Description of Contract 2016-5E work:

Planned for the 2016 construction season, the City and STEM Engineering intends to extend the relief aqueduct along John Street from Albert Street to Cathcart Street. Included in this contract is the reconstruction of the main aqueduct crossing the Albert Street right-of-way.

A key observation resulting from the hydraulic analysis of Alternative 1B indicates that the existing aqueduct does not surcharge to ground level when subject to the Timmins regional storm event. However, the model indicates that the hydraulic grade line surcharges above the aqueduct pipe obvert from its outlet south of Queen Street northerly to the Albert Street crossing. Although initially it appears that the aqueduct is of sufficient hydraulic capacity, as previously noted the aqueduct inlet on Carmen's Way adjacent to Bloor Street has insufficient inlet capacity. Approximately 2.5 m^3/s does not enter the underground aqueduct system and instead spills onto Carmen's Way and flows southerly overland and is eventually captured within the Wellington Street overpass depression. This additional flow when captured would negatively affect the hydraulic grade line downstream. Also of note, the railway crossing at the former Welded Beam Facility crossing Carmens Way surcharges and spills its banks and thus transfers approximately 2 m^3/s to the roadway overland major drainage route.

Flooding in the lower end of John Street in the area of St. Georges Avenue and St. Andrews Terrace is expected. As anticipated, the model indicates significant contributions of overland flow from both the John Street and St. Georges Avenue overland flow routes concentrating in the area noted above. While the model does not take into account inlet capacity of the minor drainage system, it is expected the inlet capacity of the catchbasins in this area would be overwhelmed in addition to the model findings of a flooded storm sewer system.

4.2 Alternative 2 – Improvements to the Wellington Street Aqueduct

Alternative 2 focuses on the replacement of the Wellington Street Aqueduct. This was prioritized in the parent EA document due to its failing structural condition. Inlet capacity deficiencies and measures to intercept and inlet overland flows into the underground system along John Street between St. Georges Ave. and St. Andrews Terrace are considered.

4.2.1 Alternative 2A – Improvements to the Aqueduct Inlet on Carmen's Way

The hydraulic analysis of the existing conditions confirms reports that the inlet to the aqueduct is subject to surcharging onto Carmen's Way. Additionally, as reported by the Sault Ste. Marie Region Conservation Authority (SSMRCA) and the City of Sault Ste. Marie Public Works and Transportation department, the inlet of the aqueduct is prone to clogging due to debris build-up washed down from the open channel. Further, the inlet and corresponding culverts up to the 4m wide x 1m high concrete box culvert installed with the construction and crossing of Carmen's Way in 2005 have areas of structural deficiencies and are in need of replacement due to their poor condition.

Alternative 2A enlarges the entrance to the aqueduct and the detailed design should include a debris rack and method of maintenance access. It should be noted, a box culvert with a larger height than required is preferred due to the potential of debris build-up. This would accommodate debris build-up while still providing adequate hydraulic inlet capacity. Alternative 2A incorporates a 4m wide by 2m high concrete box culvert at the Aqueduct inlet to the start of the 4m x 1m box culvert crossing Carmen's Way.

When the model is run subjected to the Timmins Regional Storm, peak flows entering the aqueduct at its inlet adjacent to Bloor Street are approximately 10.5 m^3/s. The hydraulic grade line within the concrete box culvert crossing Carmen's Way was found to surcharge above the obvert in the upstream half of the pipe, however this does not reach the surface and is not expected to cause surface flooding.

Similar to the results of Alternatives 1a and 1b, flooding is occurring along John Street between St. Georges Avenue and the CPR railway. The main aqueduct south of the CPR railway can be expected to surcharge above obvert from Alexandra Street south to the outlet to the open channel south of Queen Street. The computed hydraulic grade line does not reach ground surface.

Although the results appear promising, flooding in the area of John Street between St. Georges Ave. and St. Andrews Terrace remains to be addressed and is expected to contribute considerable flows to the aqueduct south of the CPR railway crossing and cause negative impacts.

4.2.2 Alternative 2B – Increasing Inlet Capacity and Reducing Flooding on John St., St. Georges Ave. and St. Andrews Terr.

Alternative 2B incorporates all previous scenarios and address the major overland drainage system and flooding issue on John Street south of St. Georges Avenue and on St. Andrews Terrace east of John Street.

As described previously in this report, the topography of the watershed causes overland flow to concentrate in this area with the CPR railway acting as a barrier. The controlling ground elevation is located on the south west corner of John Street and St. Andrews Terrace at the edge of the open channel and outlet of the Wellington Street aqueduct.

This scenario proposes to increase the inlet capacity in this area and capture the major drainage system flows, resulting in the requirement to increase the capacity of the storm sewer from John Street westerly on St. Georges Avenue and discharging into the aqueduct on Wellington Street.

Similar to Alternative 2A the hydraulic grade line within the concrete box culvert crossing Carmen's Way was found to surcharge above the obvert but not reaching the surface in the upstream half of the pipe, however; the remainder of the Wellington Street Aqueduct was found to be hydraulically adequate. Flows contributing to the aqueduct from both the John Street and the St. Georges Avenue storm sewers peak at 5.04 m^3/s. Comparing peak flow rates between Alternatives 2A and 2B show a 2.0 m^3/s (15.98 m^3/s to 17.99 m^3/s) increase in the peak flow rate at the CPR crossing as a result of capturing and providing conveyance capacity to the major overland flow contributed from John Street and St. Georges Avenue.

Downstream, the main aqueduct south of the CPR rail way surcharges from approximately midway between Cathcart Street and Edinburgh Street southerly to the outlet to the open channel south of Queen Street. Although the model indicates the hydraulic grade line does not reach the surface, there are areas where it is very near surface and any deviations and/or the accuracy of the ground elevations could result in the hydraulic grade line above ground level and flooding.

4.3 Alternative 3 – Extension of the Relief Aqueduct and Open Channel Improvements

The purpose of Alternative 3 is to evaluate the effectiveness of extending the relief aqueduct northerly on John Street ultimately connecting with the main aqueduct near the CPR crossing and to evaluate the effectiveness of improving the hydraulic characteristics of the open channel south of Bay Street.

4.3.1 Alternative 3A – Extension (Completion of) the Relief Aqueduct

Alternative 3A was primarily developed to extend the relief aqueduct on John Street from where Contract 2016-5E terminates northerly to the CPR crossing. A similar cross-section corresponding to the northwest extent of 2016-5E contract work was assumed.

Additionally, the main aqueduct was extended using a 4.878 m high by 1.675 m wide box culvert northerly along its existing route to tie into and join with the relief aqueduct at the CPR crossing.

The hydraulic analysis of the system subjected to the Timmins Regional storm continues to result is areas of hydraulic inadequacy within the lower reaches of the aqueduct south of the CPR railway, specifically from Alexandra Street south within the main aqueduct, and from the approximate midpoint between the CPR railway and Cathcart Street on John Street for the relief aqueduct. The hydraulic grade line within these areas surcharges above the obvert of the aqueduct. The hydraulic grade line nearly reaches the surface, specifically between Albert Street and Alexandria Street within the relief aqueduct in addition to a number of other areas in both the relief aqueduct and main aqueduct.

The Wellington Street aqueduct is noted to have sufficient hydraulic capacity with the exception of the crossing of Carmen's Way, however; the hydraulic grade line remains below surface. Of additional importance, the rail crossing of Carmen's Way into the former welded beam facility continues to surcharge and result in surface flooding and to reduce flows in the open channel by approximately 2.1 m^3/s. The surcharging occurs as a direct result of flows resulting from the overtopping of the hexagonal weir at the dam, and occurs after downstream peak flows have subsided. Therefore, should this crossing be improved, it would not result in detrimental impacts to the downstream aqueducts.

4.3.2 Alternative 3B – Open Channel Improvements South of Bay Street

Alternative 3B includes improvements to the open channel south of Bay Street to the outlet to the St. Marys River, in addition to the elements comprising Alternative 3A and those previous. Accumulated sediment within the five (5) parallel CSP culverts south of Bay Street is removed (0.15 m remaining), in addition to sedimentation within the downstream reach matching inverts at the outlet of the CSP's and the confluence with the river.

Significant improvements to the hydraulic grade line within both the main aqueduct and relief aqueduct results from the above improvements. Within the main aqueduct, surcharging is limited to the vicinity of the outlet to the open channel south of Queen Street and is very minor. Within the relief aqueduct up John Street, moderate surcharging occurs from Alexandra Street south to the outlet to the open channel. The hydraulic grade line stays below ground level. The crossing of Carmen's Way continues to surcharge, however does not reach the surface and the rail crossing at the welded beam facility continues to restrict flows after downstream peak flows have subsided as per Alternative 3A.

4.4 Alternative 4 – The Preferred Alternative

Building upon the previous alternatives of; improving the inlet to the aqueduct on Carmen's Way (Alternative 2A and on); increasing capacity on St. George's Avenue from John Street to Wellington Street with increases to the inlet capacity to capture the major overland drainage route flows (scenario 2B and on), and; incorporating past construction projects, current construction projects and the future completion of the aqueduct and by-pass aqueduct work south of the CPR crossing (Alternative 3A and 3B) this scenario (Scenario 4) aims to optimize the hydraulic design of the aqueduct system and open channel to the outlet at the St. Marys River.

In accordance with the parent Environmental Assessment, the preferred alternative consists of the following:

i. Incorporation of Alternatives 1A, 1B, 2A, 2B, 3A, 3B
ii. Optimization of Aqueduct sizing from its inlet at Carmen's Way to the north extents of Contracts 2015-4E and 2016-5E
iii. Improve and enlarge the channel south of Bay Street in addition to removing sediment from the five (5) CSP culverts south of Bay Street and open channel between Queen Street and Bay Street.

Due to the history of debris build-up at the entrance to the aqueduct on Carmen' Way, a slightly larger opening, especially with an allowance for potential higher water elevations caused by debris build-up at lower elevations, a 2m high by 4m wide box culvert with debris 'rack' is proposed for the inlet to the crossing of Carmen's Way and Wellington Street. Once across to the south side of Wellington, it is proposed to transition from a 4m wide by 1m high box culvert to a single 1.2m high x 3.6m wide box culvert. This sizing was selected to allow the matching of the existing invert and more importantly allow for the construction of curb and sidewalk on top and independent of the box culvert with out raising the final grade.

This cross-section continues to the intersection with St. George's Avenue where the proposed section increases to 1.5m high by 4m wide. The cross-section again requires increasing where it accepts the 1.5m high by 2.4m wide box culvert proposed to improve and accept the major overland drainage system from John Street. The proposed cross-section increases to a twin 1.5m x 3 m double box culvert which crosses Wardlaw Fuels and St. Andrews Terrace and outlets to the open channel adjacent to the CPR crossing and John Street. Figure 18 depicts the general arrangement of the preliminary preferred alternative.

Due to the high potential of constructability issues of constructing a conveyance system through/along the laneway south of the CPR crossing to Queen Street parallel to John Street, Tulloch Engineering is of the opinion that extending the John Street by-pass aqueduct, using the same cross-sectional dimensions as the termination point in Contract 2016-5E, northerly to the John Street crossing/CPR open channel is preferred in order to reduce the cross-sectional area required in the laneway aqueduct to the greatest extent practical. As such, along John Street, the box culvert was assumed to be 1.5m high by 3m wide.

Within the laneway paralleling John Street south of the CPR crossing to Albert Street, box culvert sizing was selected (varies from 1.5m to 1.8m high by 3m wide) such that the selected sizing would fit within the inside dimensions of the existing concrete aqueduct. This concept presents significant reductions in constructability concerns and significant potential cost savings.

Due to the back water effect and reduced flow velocities caused by the elevation of the St. Marys River, substantial sedimentation has occurred in the open channel south of Queen Street to the St. Marys River. Alternative 4 proposes to remove the accumulated sediment (up to 0.4m) and remove the accumulated sediment within the five (5) parallel culverts immediately south of Bay Street. Tulloch Engineering recognizes these culverts are not owned by the City but by the CN Railway, however; some attempt should be made to complete this work. South of these culverts, it is proposed to improve the open channel to the outlet of the St. Marys River by widening the base of the existing trapezoidal channel from 6m to 8m, reinstating minimum 4:1 side slopes, providing erosion protection and maintenance access.

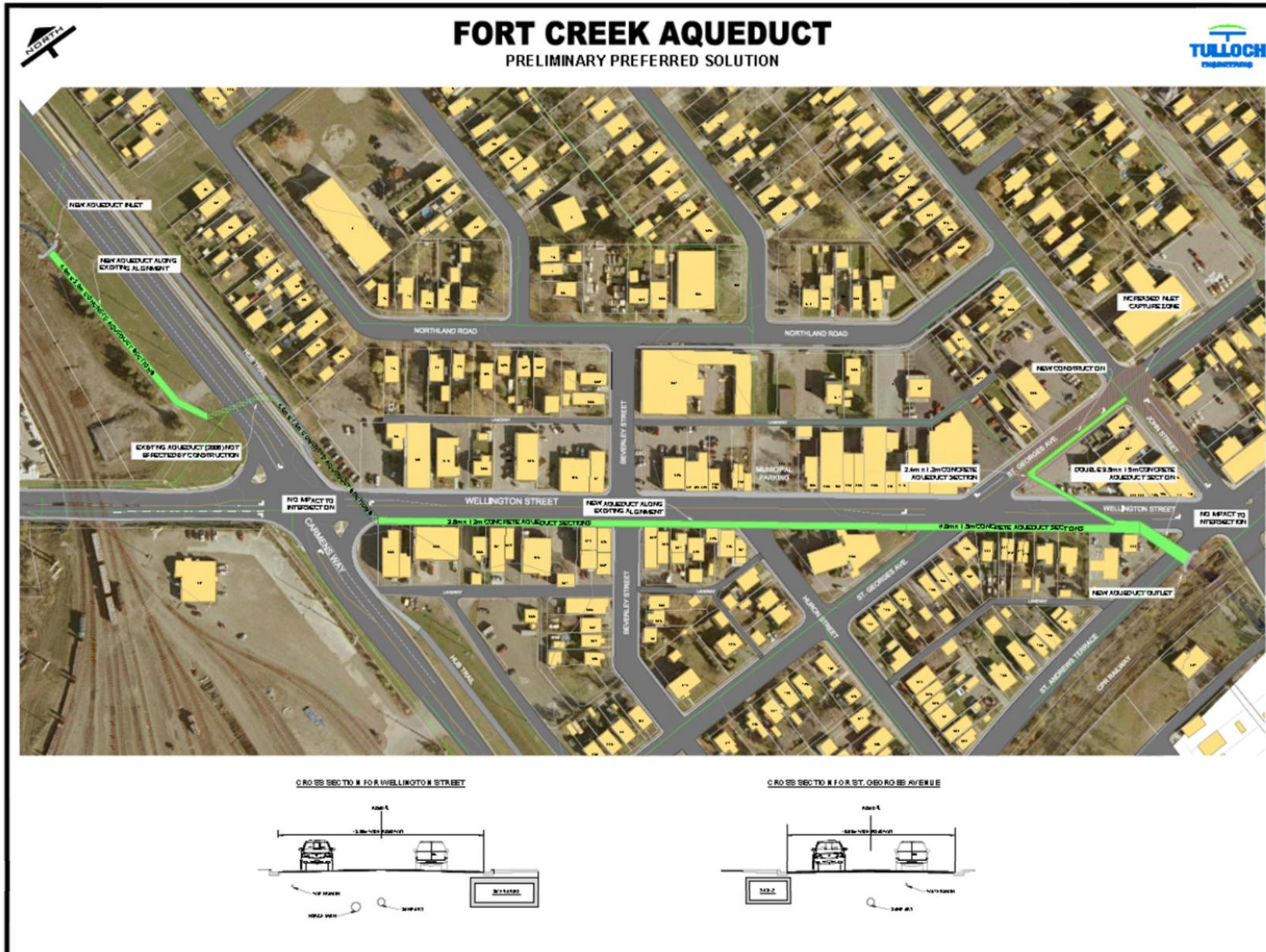
The hydraulic analysis of the Wellington Street aqueduct system as described for Alternative 4 subjected to the Return Period Event Storms and the Timmins Regional Storm results in no surcharging of the obverts of the aqueduct box culverts. The hydraulic grade line remains within the barrel for the extents of the underground system with the exception of the existing 4m by 1m concrete box culvert crossing Carmen's Way and Wellington Street which was installed in 2005/2006 and does not reach the surface.

The rail crossing of Carmen's Way into the former Welded Beam Facility continues to surcharge. Surface flooding is expected which results in a reduction of flows in the open channel by approximately 2.1 m^3/s. The surcharging occurs as a direct result of flows from the overtopping of the hexagonal weir at the Fort Creek dam, and occurs after downstream peak flows have subsided. Therefore, should this crossing be improved, it would not result in detrimental impacts to the downstream aqueducts.

The hydraulic analysis of the lower system incorporating the above scenarios and previous alternatives, yielded positive results subjected to various design storm events. The aqueduct within the laneway does not surcharge its obvert between the CPR crossing and the north limits of Contract 2016-5E (Cathcart Street). Flow rates within this aqueduct ranged from 9.385 m^3/s at the north end to 9.587 m^3/s at the limits with Contract 2016-5E. The main aqueduct within the limits of Contract 2013-4E and 2016-5E (Cathcart Street to the Queen Street open channel) also did not present surcharging with exception of the last section between Central Park Avenue and the outlet to the open channel on the south side of Queen Street. However, the surcharging is minor in nature and the hydraulic grade line is not expected to reach the surface.

The aqueduct proposed within the John Street right-of-way utilizing the cross-section as dictated by Contract 2016-5E can be expected to surcharge the obvert of the barrel for the majority of the way down John Street, however; the surcharging is minor and the hydraulic grade line is expected to remain well below the ground surface with the exception of a localized depressed area between Alexandria Street and Albert Street.

FIGURE No. 18: Preliminary Preferred Alternative



These positive results are dependent on the removal of accumulated sediment within the open channel south of Queen Street and the five (5) CSP culverts immediately south of the Bay Street bridge. Further increasing the conveyance capacity of the open channel south of Bay Street to the St. Marys River is critical to ensuring the hydraulic grade line is lowered as much as practical from the St. Marys River to the exit of the aqueduct system at Queen Street.

5 CONCLUSIONS

The completed analyses have resulted in peak flows less than those reported in previous reports. Previous reports utilized commonly accepted techniques and analysis methods, including computer programs, available at the time. The results of the broad brush watershed analysis techniques and hydrologic models employed by Proctor and Redfern Ltd. and M.M. Dillon Ltd. from 1963 to 1988 not only vary from current (CRA 2014 and Tulloch 2016) study results but also vary significantly amongst themselves. Tulloch Engineering is of the opinion, the broad brush watershed analysis used in 1988 and previous years reports, while widely used and accepted at the time, were simplistic models that did not accurately capture the lagging and storage effects of the watershed's drainage system and resulted in overestimation of peak flows. Further, the one dimensional 'cascading' models and the development of the subwatersheds did not account for the topographical characteristics of the watershed.

Our review of the most recent report prepared by Conestoga Rovers and Associates (CRA) (2014) noted that a questionable assumption was made similar to the 1977 M.M. Dillon Ltd. report which was adopted into the 1984 Proctor and Redfern Ltd. report. This assumption allotted outflows (8.5 m^3/s for a 100 year return period storm event and 15.57 m^3/s for the Regional storm event) from the Fort Creek Dam which appears to be a direct reference to the 1984 Proctor and Redfern Ltd. report. Proctor and Redfern Ltd. in its 1988 report subsequently discussed and dismissed this assumption as being overly conservative and instead computed stage/discharge outflows from the dam and reservoir in its analysis. Additionally, the differing results concluded herein as compared to the results from CRA's 2014 report can be partially attributable to the simultaneous analysis (or lack thereof) of both the minor and major drainage systems. The CRA report considered only minor 'pipe' drainage system flows, whereas this study examines both the minor system pipe flows and the major, overland drainage flow system.

After analysis of the attempted calibration results, Tulloch Engineering is of the opinion the accuracy of the model could not be verified to an acceptable degree of error by comparing peak flows generated by the model to measured peak runoff flows without reducing impervious levels and runoff curve numbers (CN) to unrealistic low levels. The theoretical model which utilizes available data over-estimates observed flows by as much as 2 – 2.5 times, leading to a greater confidence that the results are conservative. Tulloch Engineering is therefore of the opinion the theoretical results are both reasonable and acceptably conservative for use in the design and analysis of alternative solutions to the problem defined in Section 1.1. Accordingly, we conclude the following;

- The existing Aqueduct is hydraulically ***adequate*** to convey the 1Hr – 1:100Yr rainfall event without surcharging above ground level.
- The existing Aqueduct is hydraulically ***inadequate*** to convey the 24Hr – 1:100Yr rainfall event without surcharging above ground level.
- The existing Aqueduct is hydraulically ***inadequate*** to convey the Timmins Regional Storm event without surcharging above ground level.

The major overland drainage system north of the CPR Line to Conmee Ave does not correspond with the alignment of the Aqueduct. Conveyance restrictions within the storm sewer network coupled with the overland flow route(s) prevents the major drainage system from getting to the Wellington Street Aqueduct in this area. Further, the flow concentrates and flooding can result near the intersection of John Street and St. Georges Avenue and the tri-intersection of Wellington Street, John Street and St. Andrews Terrace due to the grade rise of the CPR Line. Increasing inlet capacity and conveyance to accommodate the overland flow concentration at this point and discharging to the aqueduct should alleviate this flooding issue.

The implementation of the Preferred Alternative or equivalent provides a practical, cost effective and efficient solution to the problem defined in the parent Environmental Assessment document and as presented in Section 1.1.

6 RECOMMENDATIONS

In accordance with the above observations, technical analysis and implementation of the Preferred Alternative, the following additional recommendations are provided. Various parties may be responsible for these recommendations and the effectiveness of these recommendations is largely dependent on the rainfall event that would be experienced. Additionally, these recommendations should be completed in accordance with detailed designs, which should be completed by a professional engineer experienced in hydraulic and hydrologic design.

i. Develop and adopt a detailed operational plan for the Fort Creek Dam and Reservoir including but not limited to operational guidelines to control the storage and discharge of flow from the reservoir.
   - Proctor and Redfern in its 1988 report make the following recommendation, "In the event of a major storm occurrence, the sluice gate at the Fort Creek Dam should remain at approximately 10% open position. If water levels in the reservoir rise in excess of elevation 663 ft. the storm occurrence is in excess of a 1:100 year storm. The 10% open position minimizes downstream effects yet still maintains sufficient discharge rates to keep the reservoir at low level. Immediately following a significant rise in reservoir level, the sluice gate should be opened to 50% in order to restore low reservoir elevation."

ii. Increase conveyance capacity under the CN Rail spur servicing the former Algoma Steel Welded Beam facility.

iii. Should schedule and budget permit, continue and/or reinstate flow monitoring within the Fort Creek Aqueduct to confirm design flows.

iv. Review the major overland drainage system south of the CPR rail line and along John Street to ensure the major system is not negatively affected by changes to road grade due to recent construction projects.

v. The accumulation of silt and sedimentation within the Fort Creek Reservoir is occurring, resulting in a decrease of the volume of 'dead' storage. An initial calculation of dead storage volume was 20 ha.m by Proctor and Redfern. Until this volume of siltation is realized, it should have little to no effect on the attenuating effects of the dam, however, it has also been recognized in previous studies that significant siltation has and is continuing to occur. We therefore recommend a siltation study be undertaken to determine the remaining volume of dead storage within the reservoir and initiate a dredging and sedimentation removal program if recommended.

vi. Undertake a drainage study examining in detail the causes and potential solutions to alleviate flooding of the Wellington Street Underpass.

vii. Implement the Preferred Alternative (Alternative 4) presented herein.

7 STATEMENT OF LIMITATIONS AND QUALIFICATIONS

The attached Report (the "Report") has been prepared by TULLOCH Engineering Inc. (the "Consultant") for the benefit of the client (the "Client") in accordance with the agreement between Consultant 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 the Consultant's professional judgment in light of the Limitations and industry standards for the preparation of similar reports;
- may be based on information provided to Consultant 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.

The Consultant 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. The Consultant 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.

The Consultant agrees that the Report represents its professional judgment as described above and that the Information has been prepared for the specific purpose and use described in the Report and the Agreement, but the Consultant 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 the Consultant represents the Consultant's professional judgment in light of its experience and the knowledge and information available to it at the time of preparation. Since the Consultant has no control over market or economic conditions, prices for construction labour, equipment or materials or bidding procedures, The Consultant, 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.

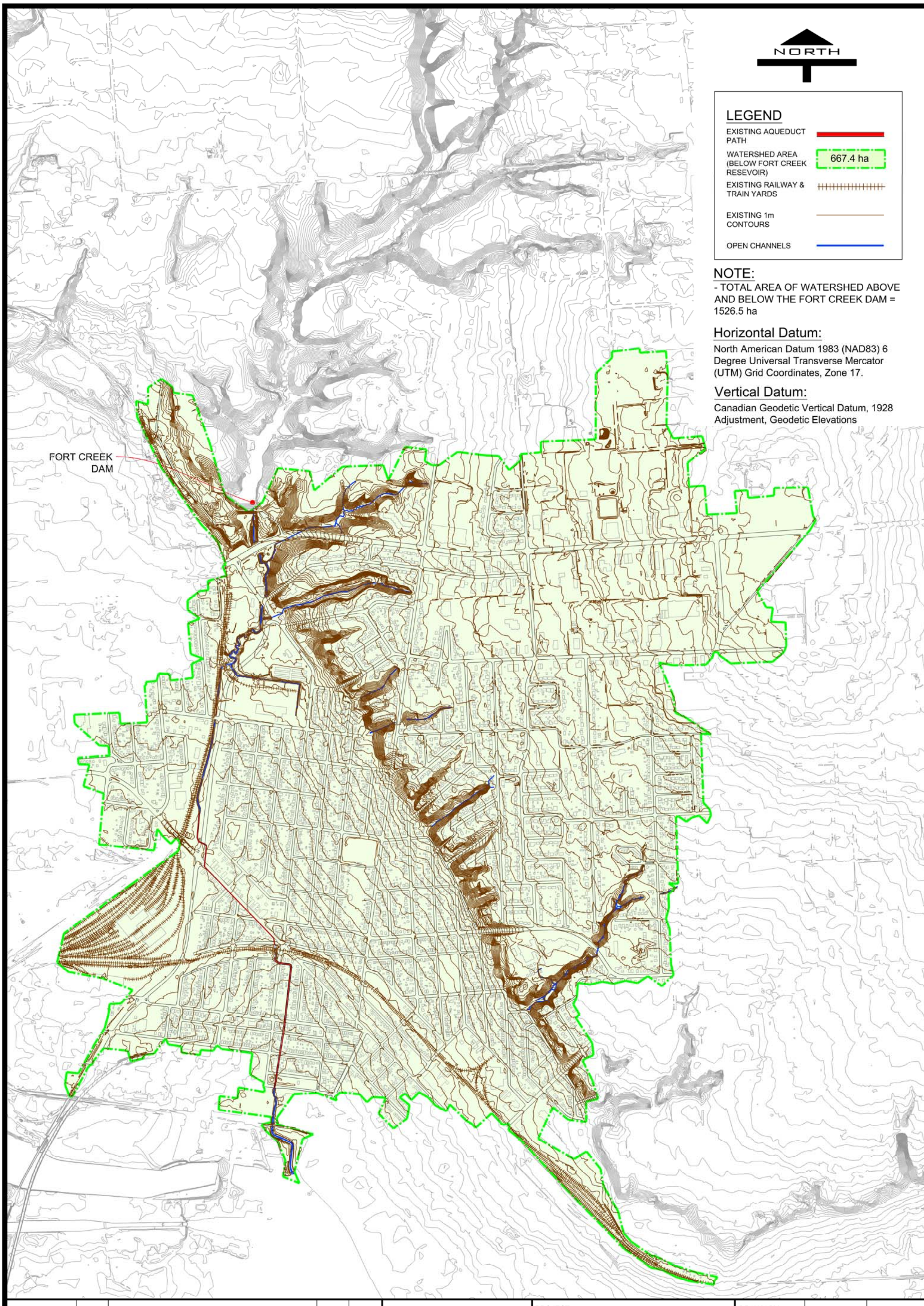
Except (1) as agreed to in writing by the Consultant and Client; (2) as required by-law; or (3) to the extent used by governmental reviewing agencies for the purpose of obtaining permits or approvals, the Report and the Information may be used and relied upon only by Client.

The Consultant 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 Consultant 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.

APPENDIX A

TULLOCH
ENGINEERING



DATE	REV.	REVISION	BY	APPD
2016/01/12	0	ISSUED FOR REPORT	AGH	JVM

TULLOCH ENGINEERING

PROJECT:
FORT CREEK AQUEDUCT
WELLINGTON ST, CARMENS WAY TO JOHN ST.

DRAWING:
LOWER WATERSHED - TOPOGRAPHIC

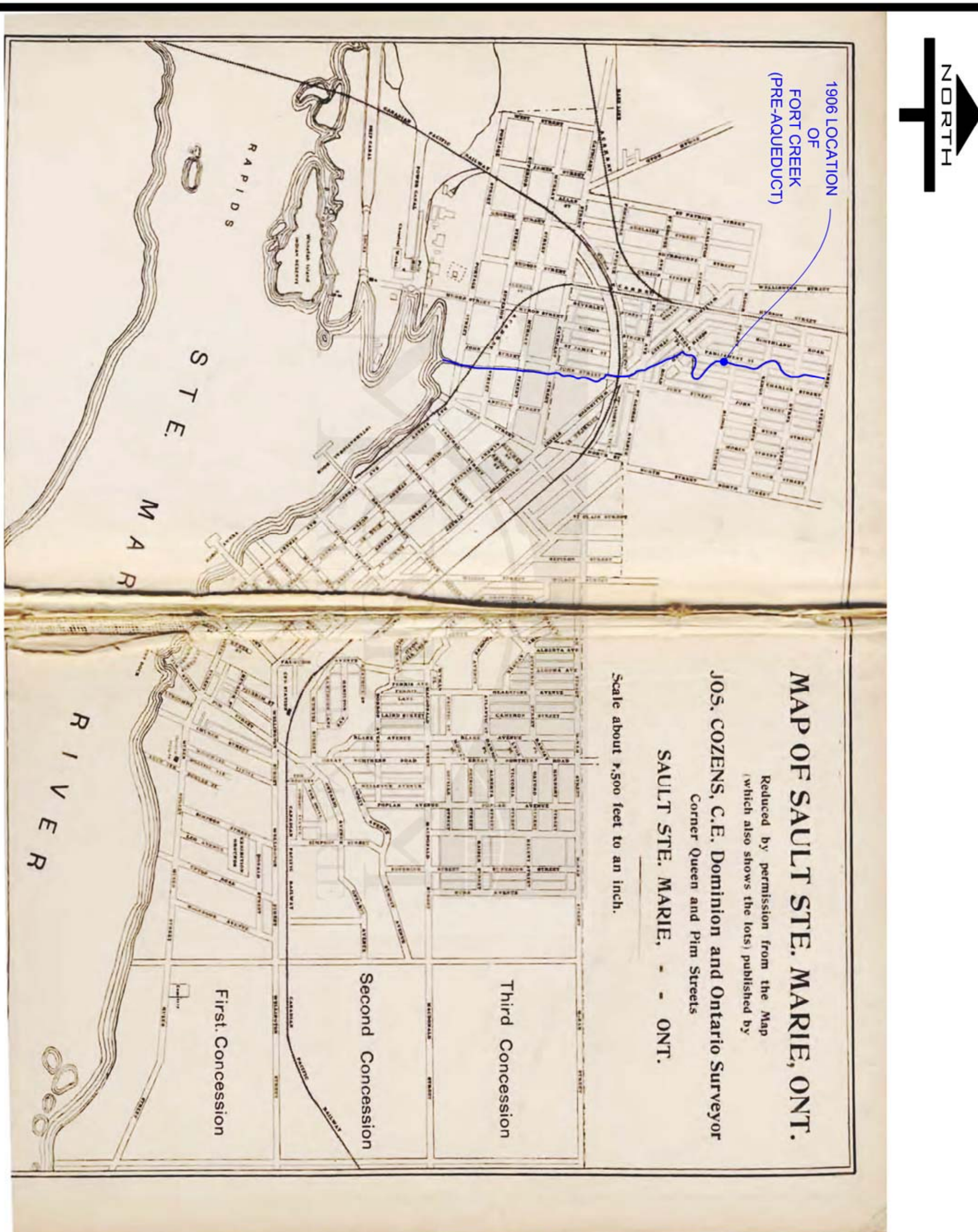
DRAWN BY: KN/AGH
SCALE: 1:25000
DATE: 02/09/2016
DESIGNED BY: JVM
PROJECT No. : 15-1192
CHECKED BY: JVM
APPROVED BY: JVM

FIG. 5

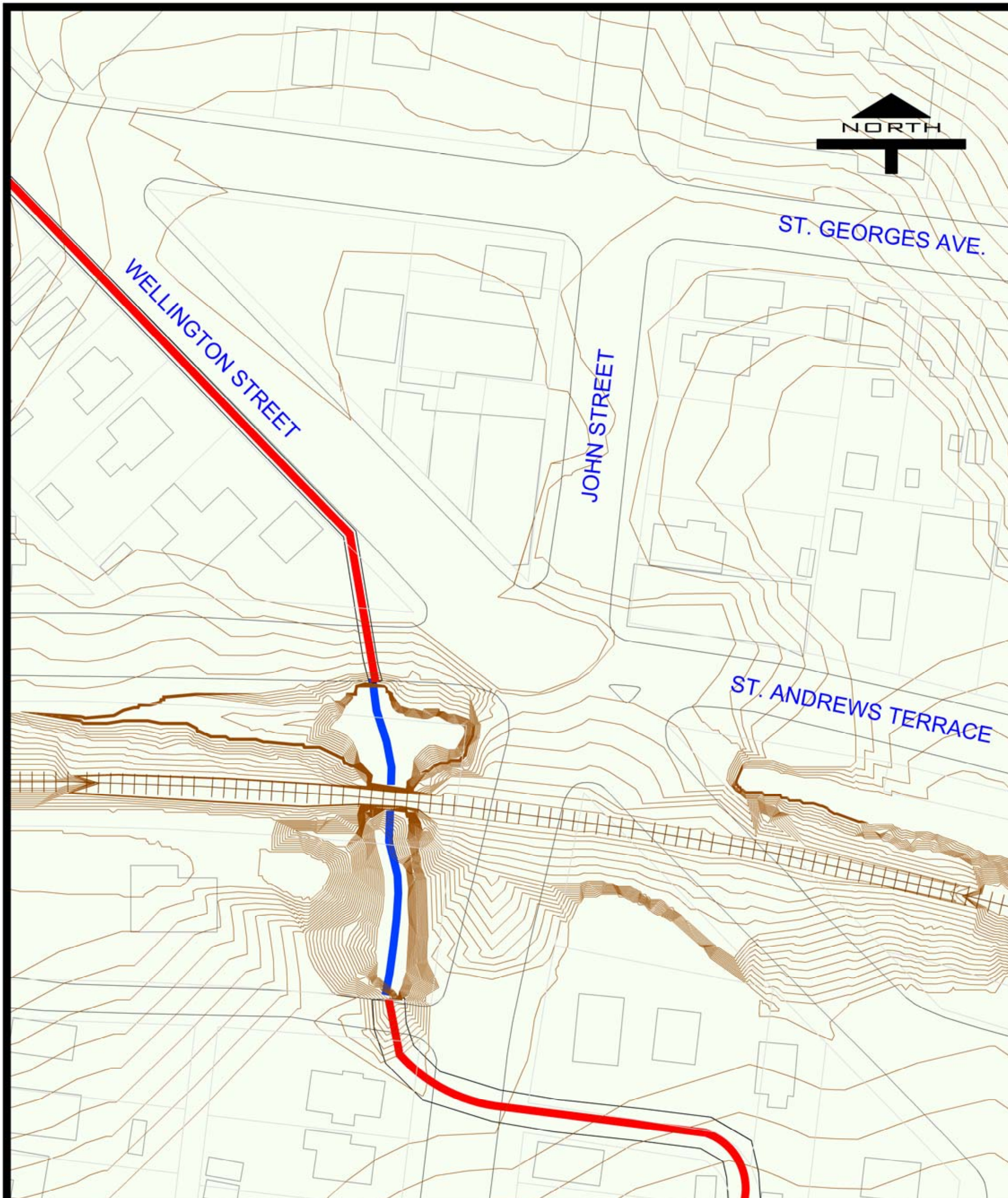
DATE	REV.	REVISION	BY	APPD
2016/01/12	0	ISSUED FOR REPORT	AGH	JVM

TULLOCH ENGINEERING

PROJECT: **FORT CREEK AQUEDUCT** WELLINGTON ST, CARMENS WAY TO JOHN ST.	DRAWN BY: KN/AGH	SCALE: N.T.S. / DATE: 03/01/2016
	DESIGNED BY: JVM	PROJECT No.: 15-1192
DRAWING: FLOODLINES FOR REGIONAL 100YR & 50 YR	CHECKED BY: JVM	**FIG. 6**
	APPROVED BY: JVM	



TULLOCH ENGINEERING	PROJECT: **FORT CREEK AQUEDUCT** WELLINGTON ST, CARMENS WAY TO JOHN ST.	DRAWN BY: AGH	SCALE: N.T.S.	DATE: 03/02/2016
		DESIGNED BY: PM	PROJECT No.: 15-1192-01	
	DRAWING: 1906 LOCATION OF FORT CREEK	CHECKED BY: JVM	**FIG. 7**	
		APPROVED BY: JVM		



TULLOCH ENGINEERING

PROJECT: **FORT CREEK AQUEDUCT** WELLINGTON ST, CARMENS WAY TO JOHN ST.	DRAWN BY: AGH	SCALE: 1:1000	DATE: 03/02/2016
	DESIGNED BY: JVM	PROJECT No. : 15-1192-01	
DRAWING: TOPOGRAPHY OF THE WELLINGTON ST. / JOHN ST. / ST ANDREWS TERR. INTERSECTION	CHECKED BY: JVM	**FIG. 9**	
	APPROVED BY: JVM		

APPENDIX B

TULLOCH
ENGINEERING

Table 8: Flow Comparisons of Past Reports

Author (year)	Report Title	Resevoir Drainage Area (ha)	Storm Event Details	Resevoir Peak Inflow (m3/s)	Peak Resevoir Outflows (m3/s)		Peak Flow Rates by Location (m3/s)					Notes
					Gate Condition	Outflow	Conmee	Bloor	CPR	Queen St Outlet	St Mary's River Channel	
P&R (1963)		769										
P&R (1967)	Fort Creek Aqueduct Appraisal	769	100 yr 6hr		Unknown Gate Condition			14.7	25.48		30.02	See Note 1.
P&R (1970)	Fort Creek Channel, Aqueduct to Second Line	769	100 yr 6hr		Unknown Gate Condition	0.71		11.46				See Note 1.
Dillon (1977)	Flood Plain Mapping Report		Regional (12 hr)	38.79	Unknown Gate Condition	12.74	13.59	17.84	27.04	34.8	39.08	See Note 2 and 5.
P&R (1984)	Fort Creek Watershed Appraisal	750	100yr 24hr	31.15	Unknown Gate Condition	8.50	11.21	16.88	23.87		35.96	See Note 1, 2 and 5.
			Regional (12 hr)	42.96	Unknown Gate Condition	15.57	24.44	29.73	38.43		52.96	See Note 2 and 5.
P&R (1988)	Fort Creek Flood Reduction Study	750	100yr 24hr	31.12	Gate 0% Open	0.00	10.48	11.30	27.21	29.08	31.97	See Note 1.
					Gate 10% Open	0.59	10.48	11.30	27.21	29.08	31.97	See Note 1.
					Gate 50% Open	2.63	10.48	11.30	27.21	29.08	31.97	See Note 1.
					Gate 100% Open	4.53	10.48	11.30	27.21	29.08	31.97	See Note 1 and 5.
			Regional (12 hr)	42.96	Gate 0% Open	3.99	13.39	14.44	34.97	37.75	41.48	
					Gate 10% Open	3.00	13.71	14.70	35.40	38.06	41.82	
					Gate 50% Open	3.85	15.09	15.60	36.90	39.13	42.98	
					Gate 100% Open	7.28	16.54	16.57	38.43	40.24	44.20	See Note 5.
CRA (2014)	Fort Creek Aqueduct Hydrologic/Hydraulic Analysis (Existing Conditions-STEM 2)	N/A	100 yr 1 hr AESII	N/A	Unknown Gate Condition	8.5	12.678	10.34	14.59	16.77	21.32	See Note 2 and 3.
			100 yr 24hr AESII	N/A	Unknown Gate Condition	8.5	14.41	11.27	16.782	22.95	24.07	See Note 2 and 3.
			Regional (12 hr)	N/A	Unknown Gate Condition	15.57	17.965	16.34	19.18	25.31	38.49	See Note 2 and 3.
Tulloch (2016)	Existing Conditions Model	859.1	100 yr 1 hr AES Type 2	64.55	Gate 10% Open	0.469	6.351	6.903	12.683	16.578	17.276	
			100 yr 24hr SCS Type 2	49.93	Gate 10% Open	0.555	7.918	9.521	14.37	17.954	18.691	
			Regional (12 hr)	32.52	Gate 10% Open	11.04	10.494	10.535	14.298	17.899	18.657	See Note 4 and 5.
Tulloch (2016)	Alternative 4 - Preferred Solution	859.1	100 yr 1 hr AES Type 2	71.85	Gate 10% Open	0.486	8.061	8.171	16.782	22.877	23.476	Note: Climate Change Multipliers apply
			100 yr 24hr SCS Type 2	49.93	Gate 10% Open	0.555	8.282	9.65	18.638	24.68	25.359	
			Regional (12 hr)	32.52	Gate 10% Open	11.04	10.504	10.533	18.109	24.01	25.009	See Note 4 and 5.

Note 1: The rainfall distribution type is unknown, however it is suspected it is an early form of the SCS distribution which was developed for the southern United Sates.

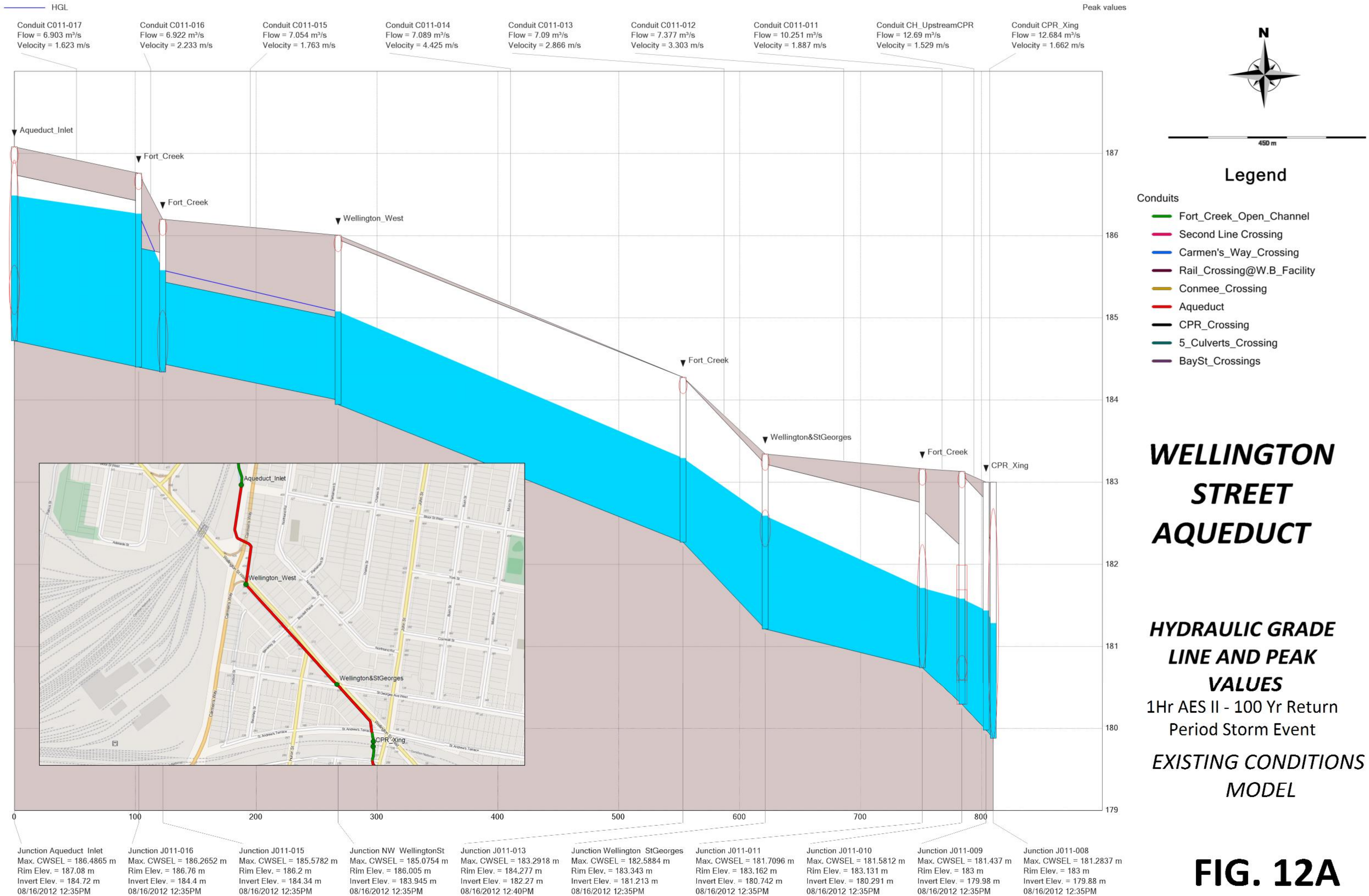
Note 2: These reports used the assumption the Fort Creek Dam was partially full prior to the advent of the Regional Storm event

Note 3: The hydraulic analysis was limited to the minor system only. To prevent losses from surcharged pipes and manholes, a 'allow ponding' option was permitted resulting the surcharged water to pond within a defined area, which in turn could result in excessive head effects (pressure pipe) conditions.

Note 4: Surcharging and overtopping of the banks occurs at the railway crossing Carmen's Way servicing the former Welded Beam facility. Approximately 2 m^3/s is lost from the channel and is routed overland in the major drainage system

Note 5: Overtopping of the hexagon weir occurs at the Dam

APPENDIX C



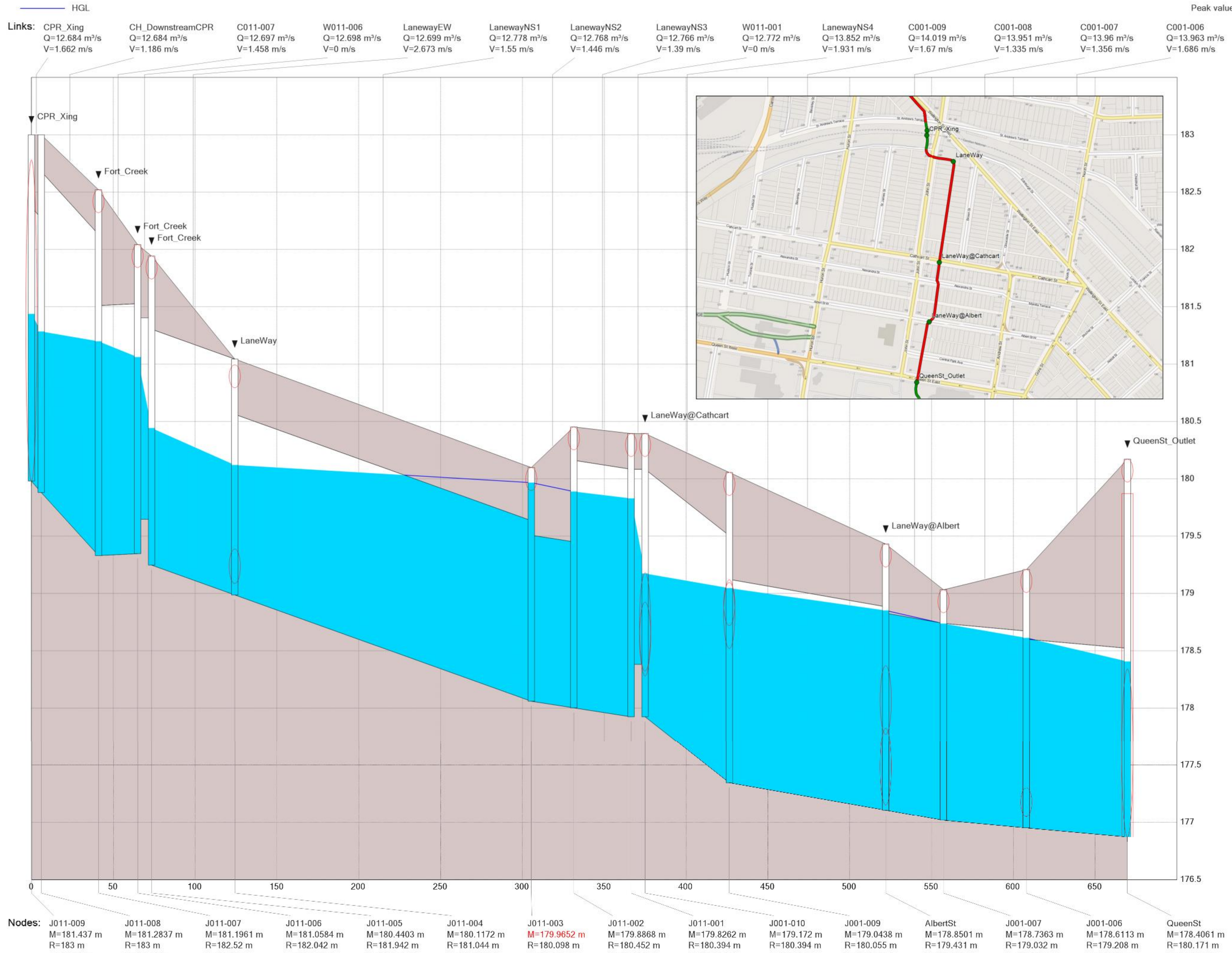
WELLINGTON STREET AQUEDUCT

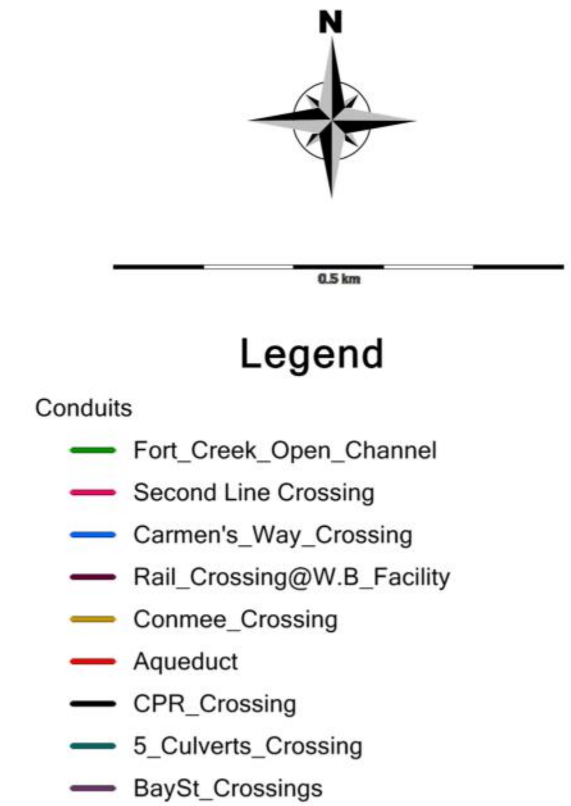
HYDRAULIC GRADE LINE AND PEAK VALUES

1Hr AES II - 100 Yr Return Period Storm Event

EXISTING CONDITIONS MODEL

FIG. 12A





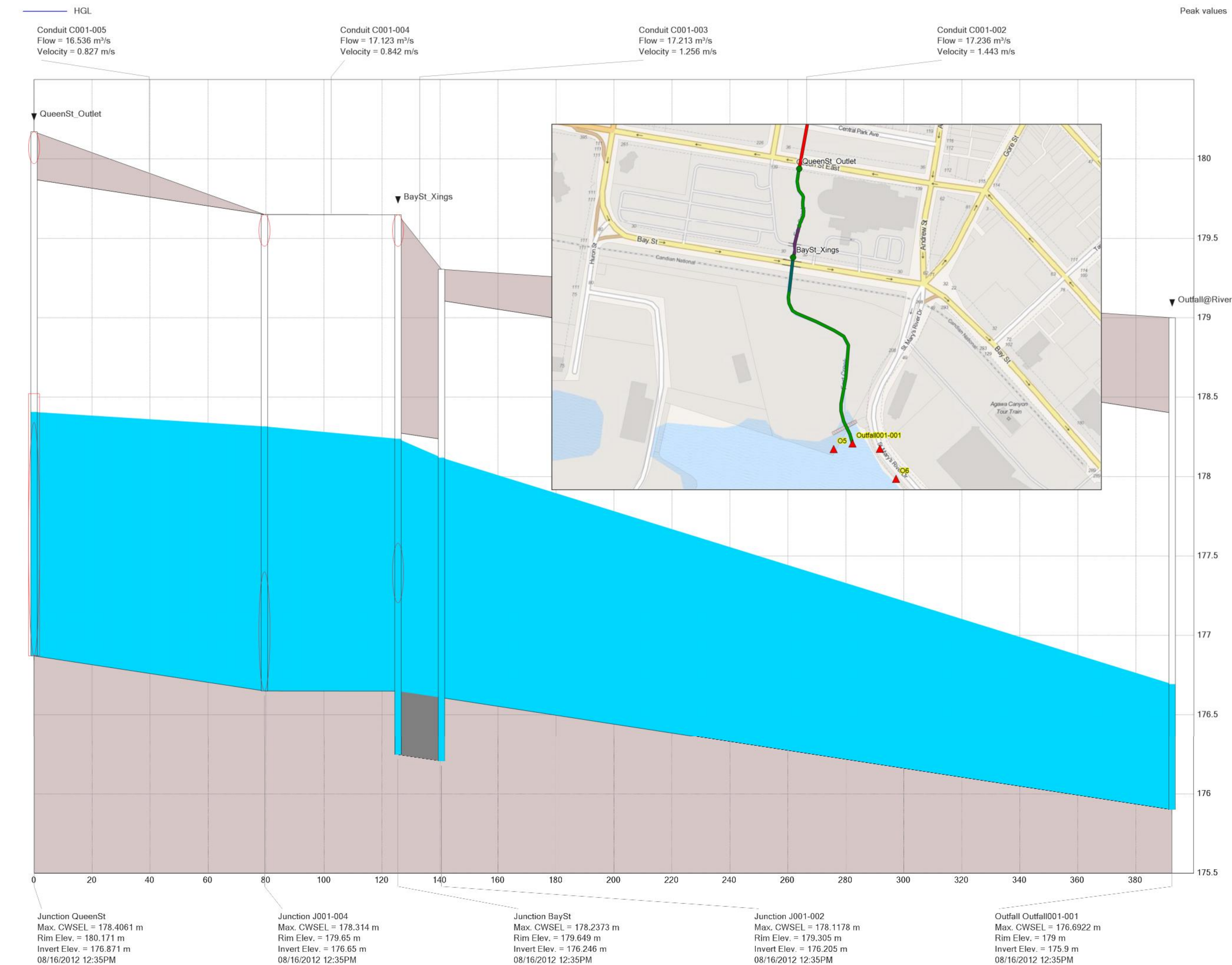
JOHN STREET LANEWAY AQUEDUCT

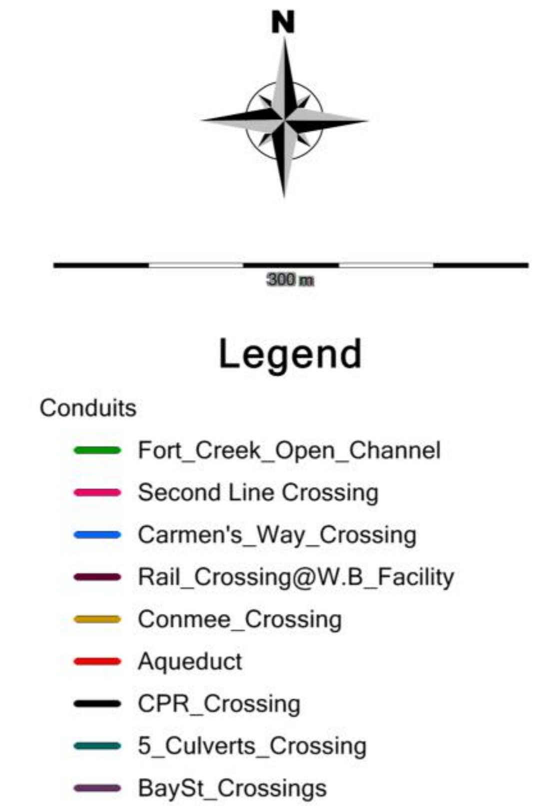
HYDRAULIC GRADE LINE AND PEAK VALUES

1Hr AES Type II - 100 Yr Return Period Storm Event

EXISTING CONDITIONS MODEL

FIG. 12B





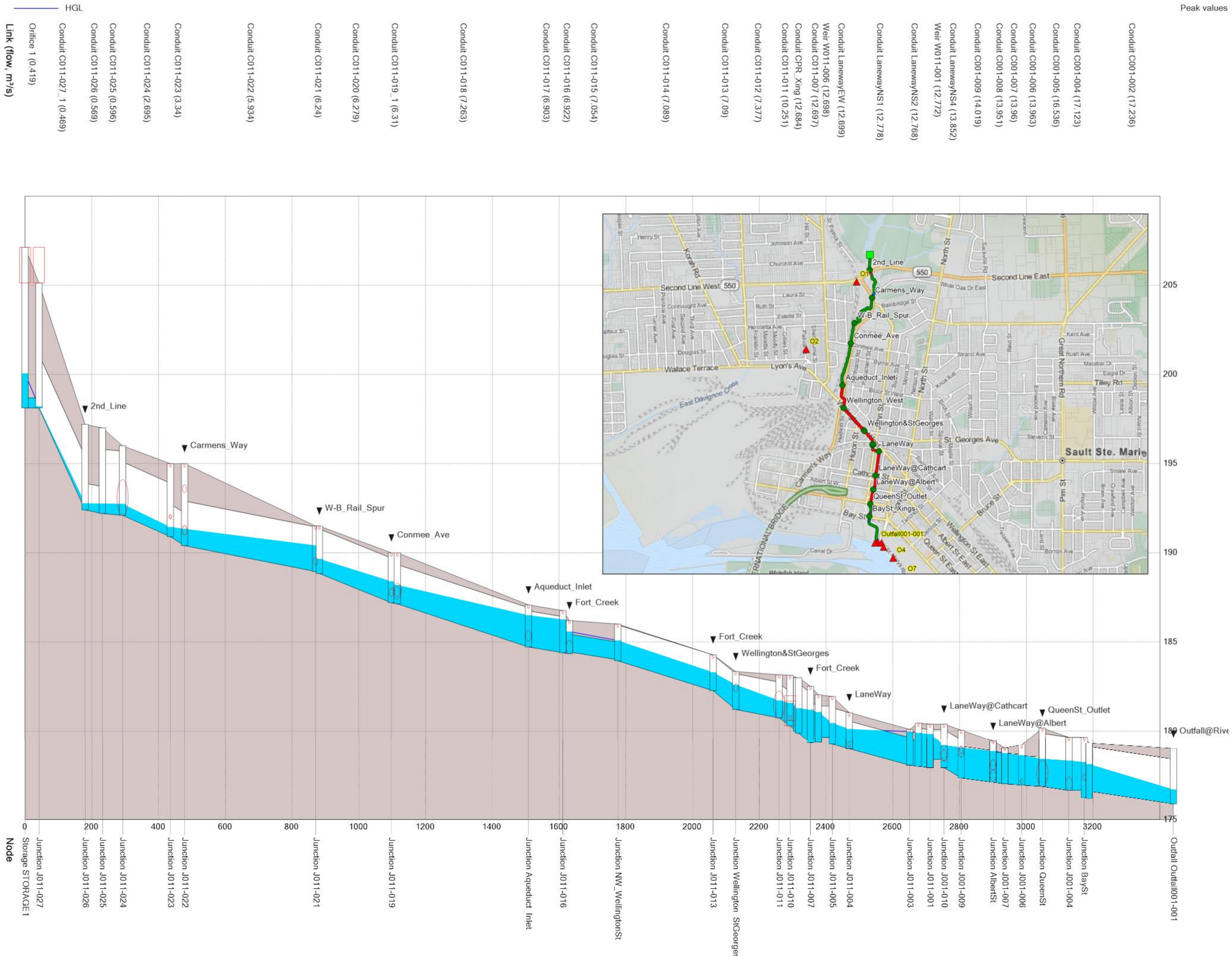
FORT CREEK OPEN CHANNEL SOUTH OF QUEEN ST.

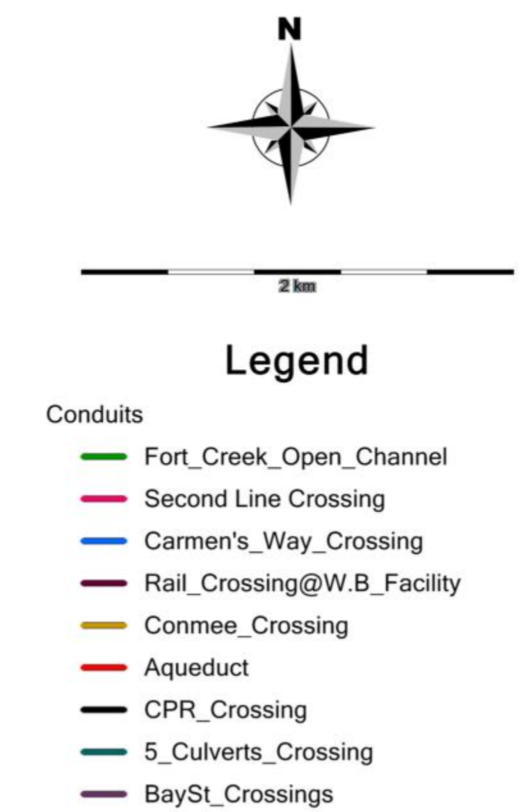
HYDRAULIC GRADE LINE AND PEAK VALUES

1Hr AES Type II - 100 Yr Return Period Storm Event

EXISTING CONDITIONS MODEL

FIG. 12C





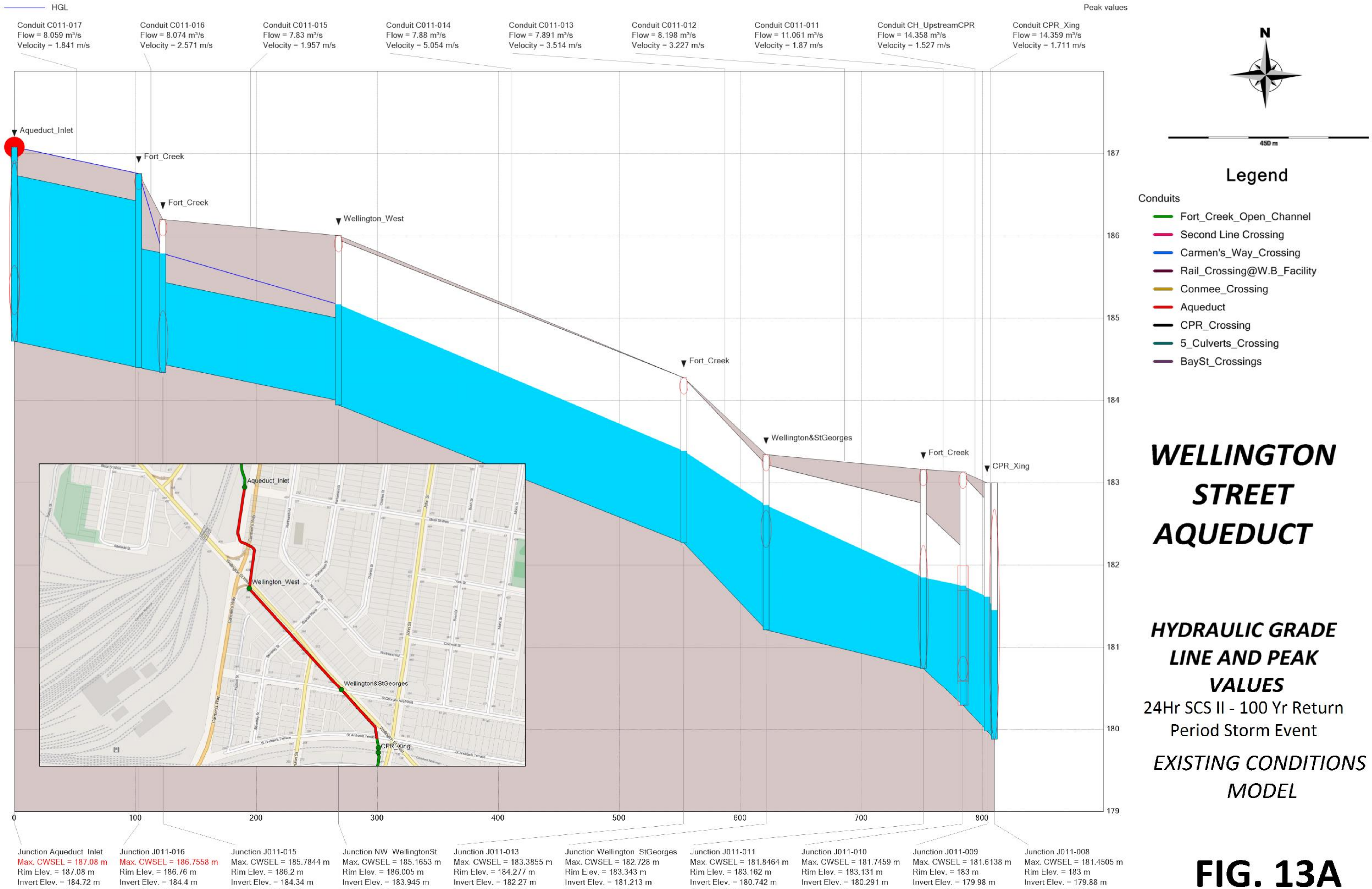
THE FORT CREEK SYSTEM: DAM TO ST. MARY'S RIVER

HYDRAULIC GRADE LINE AND PEAK VALUES

1Hr AES Type II - 100 Yr Return Period Storm Event

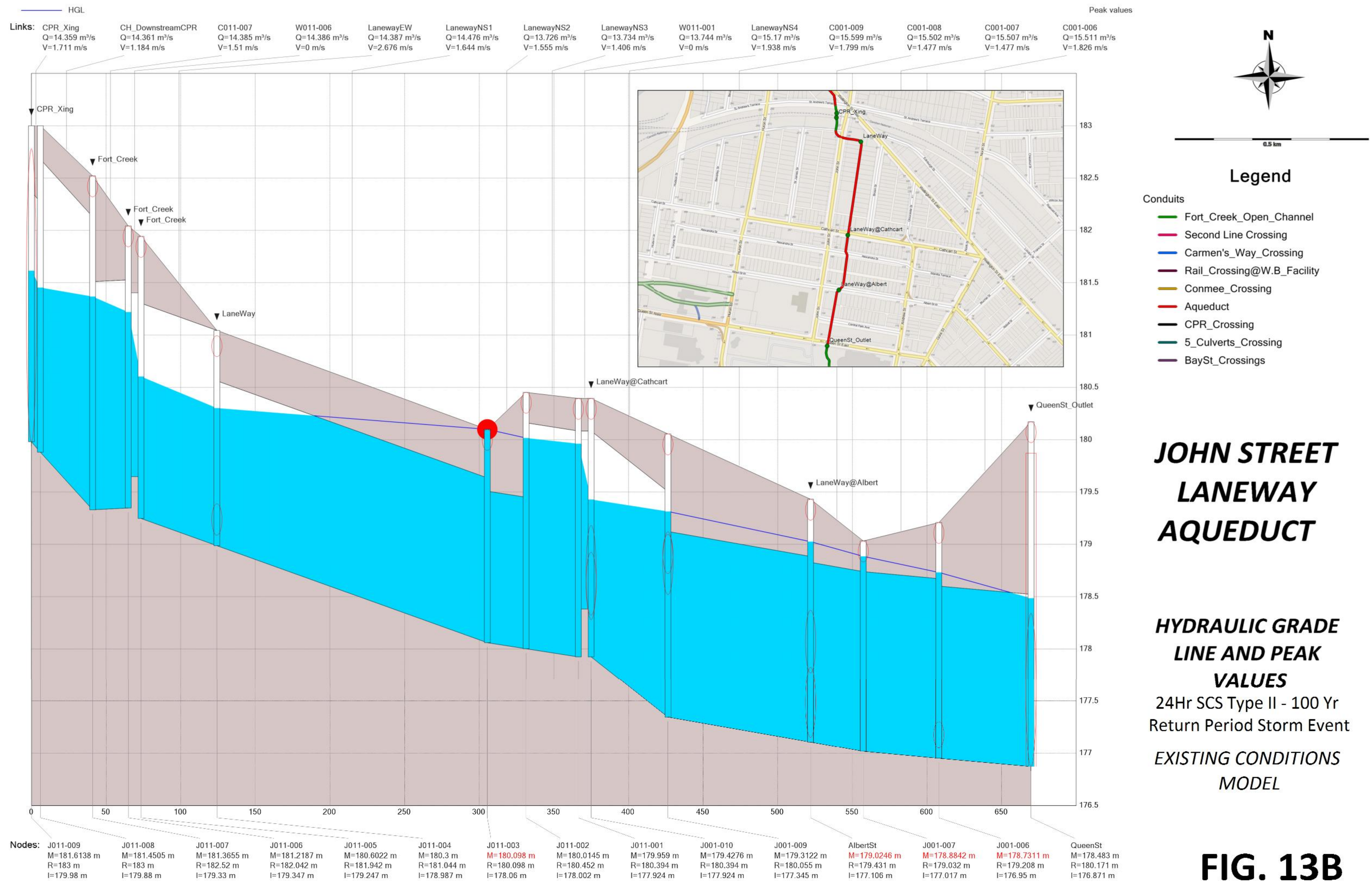
EXISTING CONDITIONS MODEL

FIG. 12D



HGL
Peak values
Conduit C011-017
Flow = 8.059 m³/s
Velocity = 1.841 m/s
Conduit C011-016
Flow = 8.074 m³/s
Velocity = 2.571 m/s
Conduit C011-015
Flow = 7.83 m³/s
Velocity = 1.957 m/s
Conduit C011-014
Flow = 7.88 m³/s
Velocity = 5.054 m/s
Conduit C011-013
Flow = 7.891 m³/s
Velocity = 3.514 m/s
Conduit C011-012
Flow = 8.198 m³/s
Velocity = 3.227 m/s
Conduit C011-011
Flow = 11.061 m³/s
Velocity = 1.87 m/s
Conduit CH_UpstreamCPR
Flow = 14.358 m³/s
Velocity = 1.527 m/s
Conduit CPR_Xing
Flow = 14.359 m³/s
Velocity = 1.711 m/s
Aqueduct_Inlet
Fort_Creek
Fort_Creek
Wellington_West
Fort_Creek
Wellington&StGeorges
Fort_Creek
CPR_Xing
187
186
185
184
183
182
181
180
179
0
100
200
300
400
500
600
700
800
N
450 m
Legend
Conduits
Fort_Creek_Open_Channel
Second Line Crossing
Carmen's_Way_Crossing
Rail_Crossing@W.B_Facility
Conmee_Crossing
Aqueduct
CPR_Crossing
5_Culverts_Crossing
BaySt_Crossings
WELLINGTON STREET AQUEDUCT
HYDRAULIC GRADE LINE AND PEAK VALUES
24Hr SCS II - 100 Yr Return Period Storm Event
EXISTING CONDITIONS MODEL
Aqueduct_Inlet
Wellington_West
Wellington&StGeorges
CPR_Xing
Junction Aqueduct Inlet
Max. CWSEL = 187.08 m
Rim Elev. = 187.08 m
Invert Elev. = 184.72 m
08/17/2012 12:05AM
Junction J011-016
Max. CWSEL = 186.7558 m
Rim Elev. = 186.76 m
Invert Elev. = 184.4 m
08/17/2012 12:05AM
Junction J011-015
Max. CWSEL = 185.7844 m
Rim Elev. = 186.2 m
Invert Elev. = 184.34 m
08/17/2012 12:05AM
Junction NW WellingtonSt
Max. CWSEL = 185.1653 m
Rim Elev. = 186.005 m
Invert Elev. = 183.945 m
08/17/2012 12:05AM
Junction J011-013
Max. CWSEL = 183.3855 m
Rim Elev. = 184.277 m
Invert Elev. = 182.27 m
08/17/2012 12:05AM
Junction Wellington StGeorges
Max. CWSEL = 182.728 m
Rim Elev. = 183.343 m
Invert Elev. = 181.213 m
08/17/2012 12:00AM
Junction J011-011
Max. CWSEL = 181.8464 m
Rim Elev. = 183.162 m
Invert Elev. = 180.742 m
08/16/2012 11:55PM
Junction J011-010
Max. CWSEL = 181.7459 m
Rim Elev. = 183.131 m
Invert Elev. = 180.291 m
08/16/2012 11:55PM
Junction J011-009
Max. CWSEL = 181.6138 m
Rim Elev. = 183 m
Invert Elev. = 179.98 m
08/16/2012 11:55PM
Junction J011-008
Max. CWSEL = 181.4505 m
Rim Elev. = 183 m
Invert Elev. = 179.88 m
08/16/2012 11:55PM

FIG. 13A



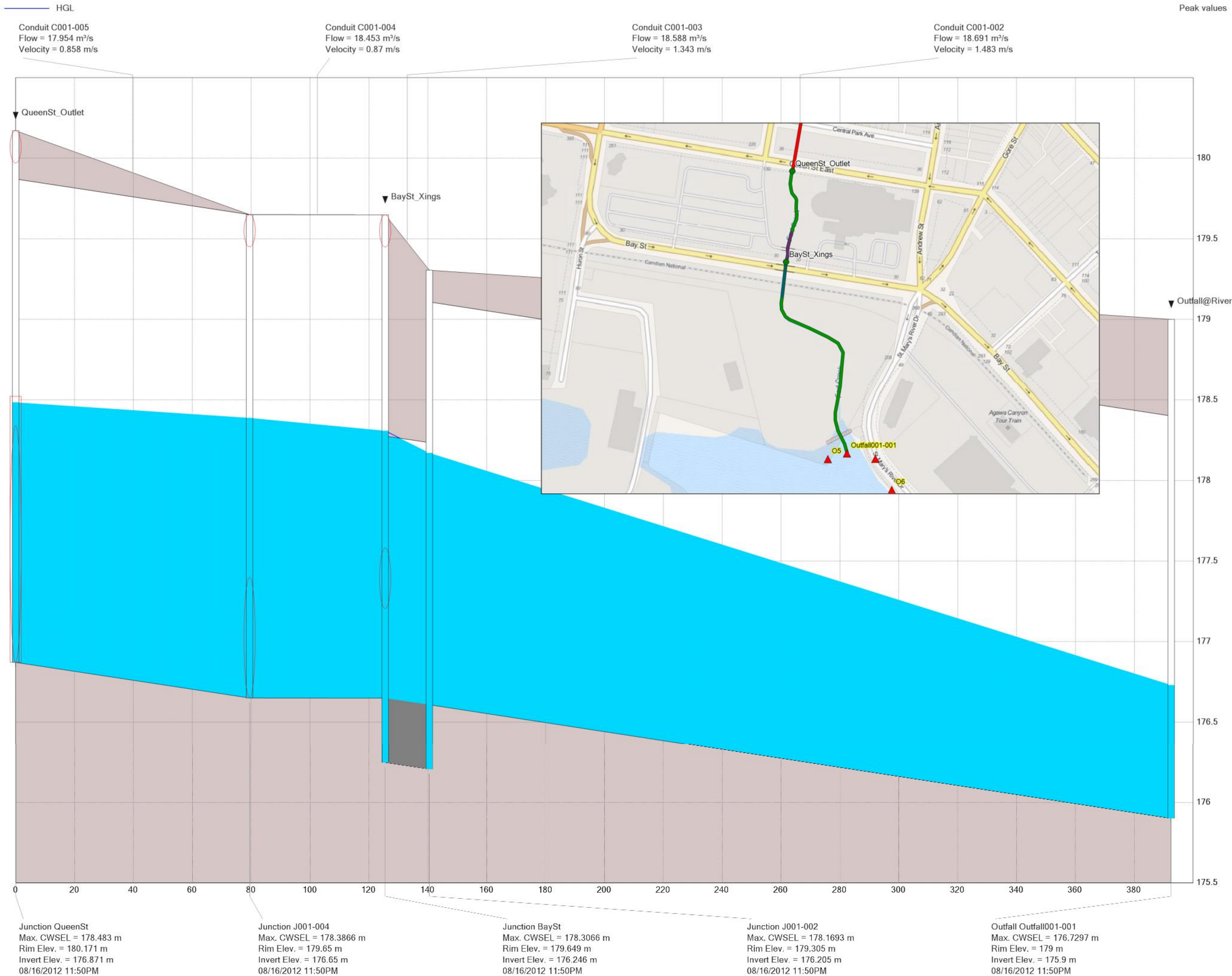
JOHN STREET LANEWAY AQUEDUCT

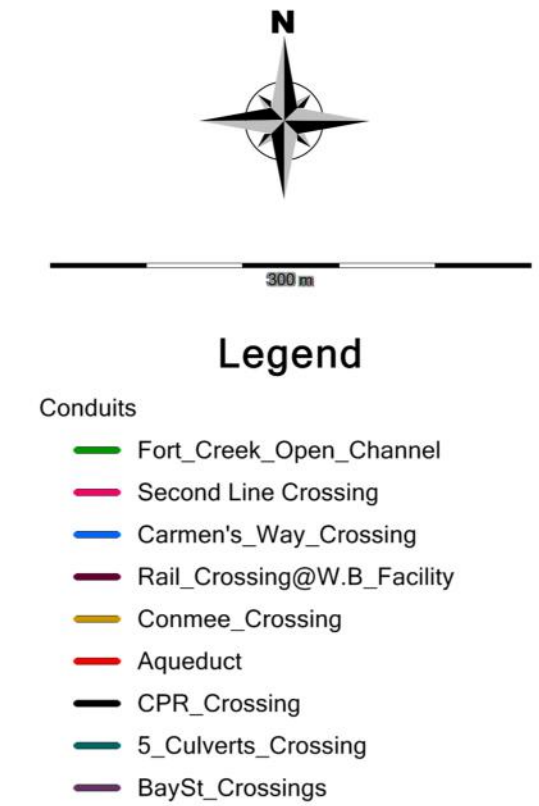
HYDRAULIC GRADE LINE AND PEAK VALUES

24Hr SCS Type II - 100 Yr Return Period Storm Event

EXISTING CONDITIONS MODEL

FIG. 13B





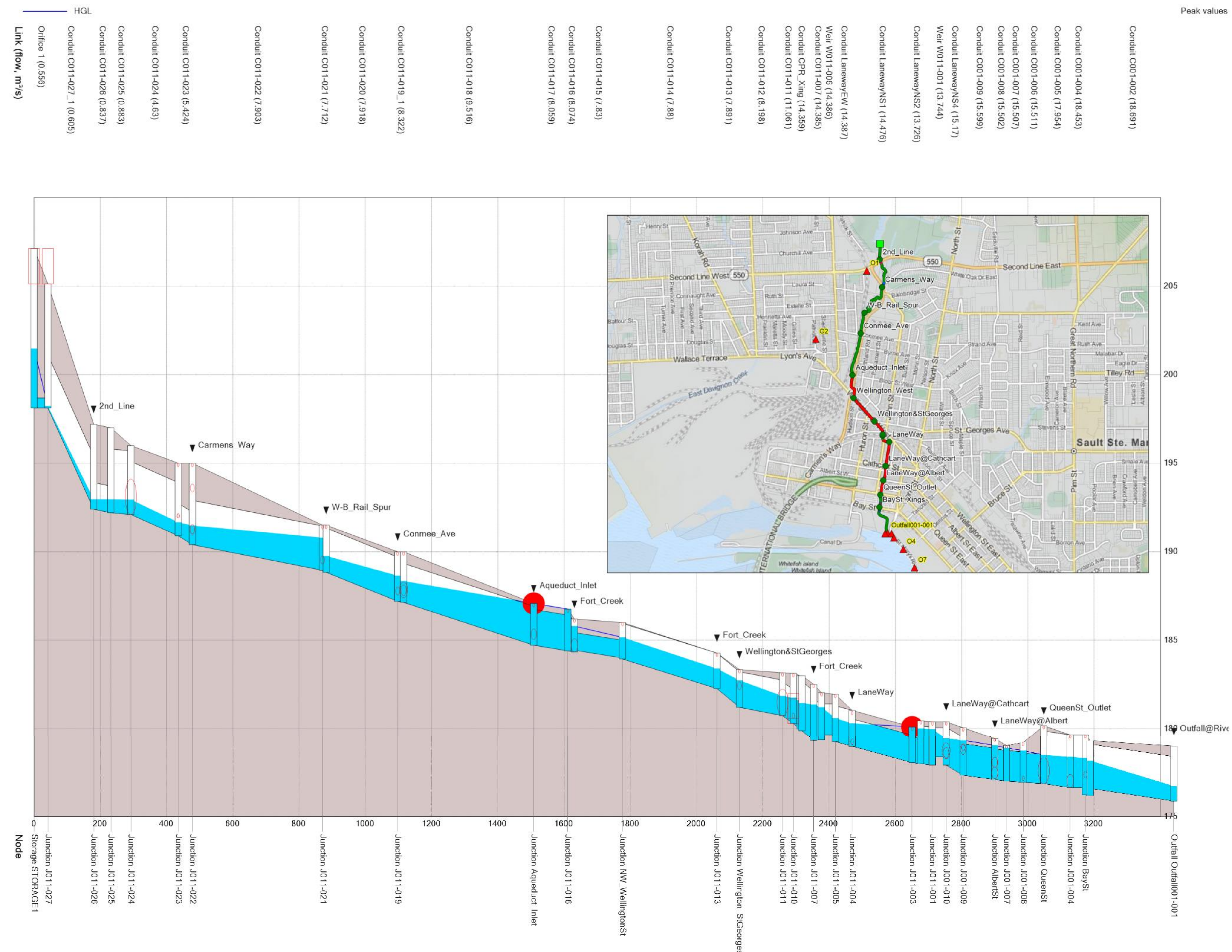
FORT CREEK OPEN CHANNEL SOUTH OF QUEEN ST.

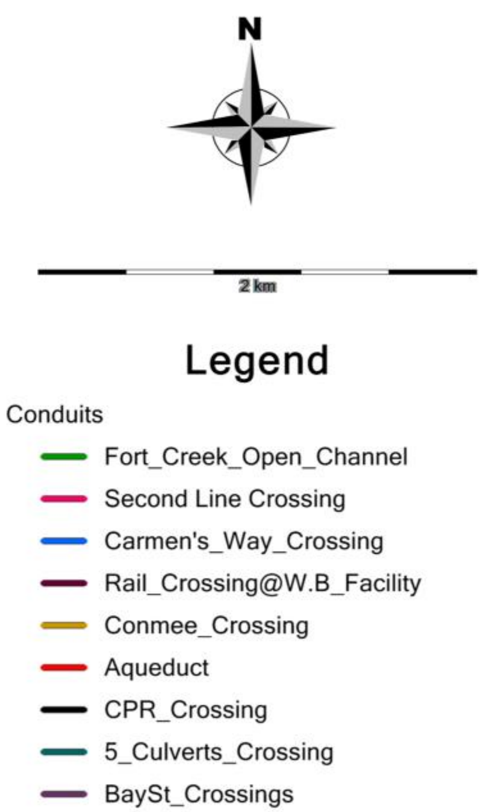
HYDRAULIC GRADE LINE AND PEAK VALUES

24Hr SCS Type II - 100 Yr Return Period Storm Event

EXISTING CONDITIONS MODEL

FIG. 13C





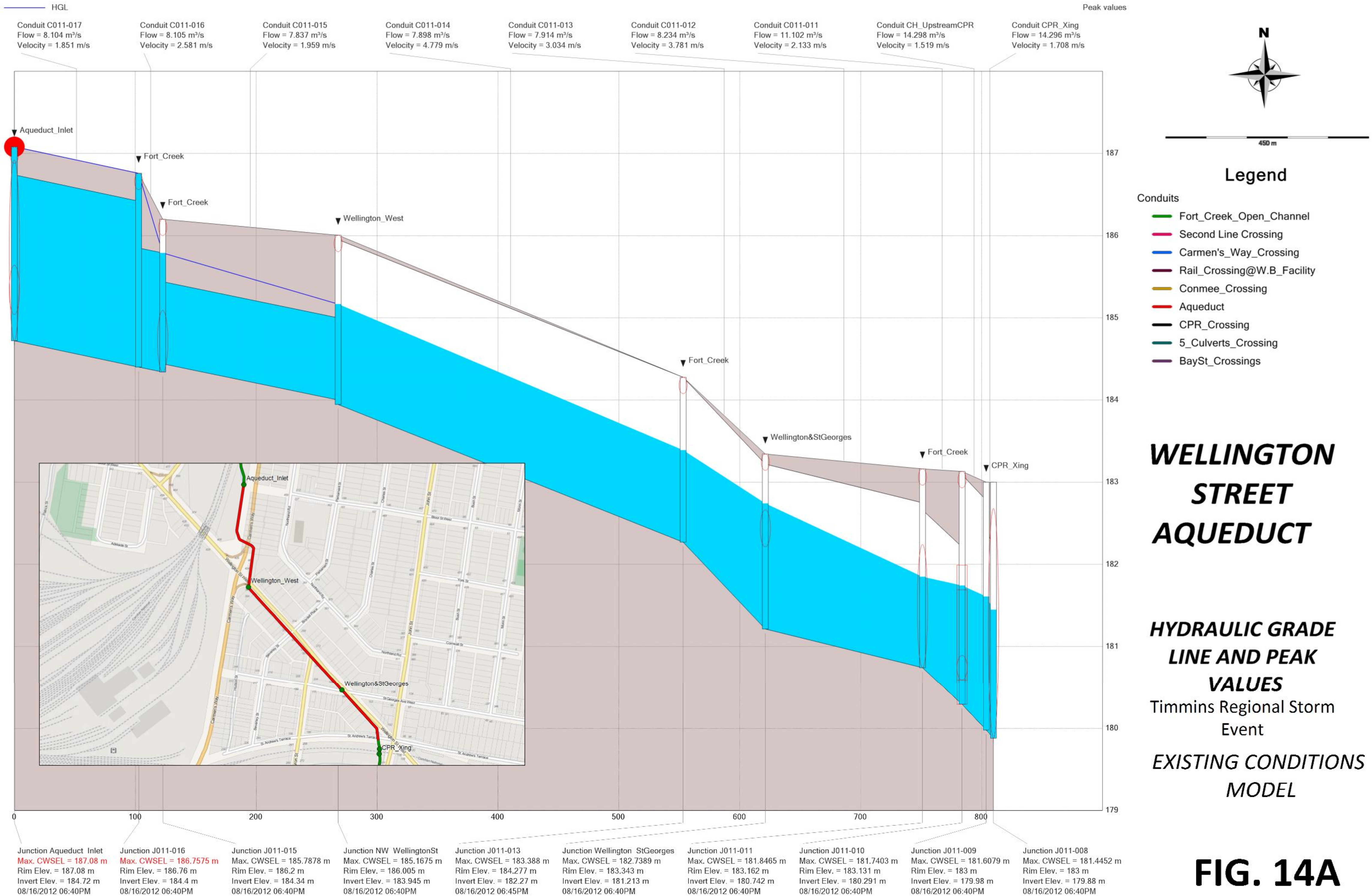
THE FORT CREEK SYSTEM: DAM TO ST. MARY'S RIVER

HYDRAULIC GRADE LINE AND PEAK VALUES

24Hr SCS Type II - 100 Yr Return Period Storm Event

EXISTING CONDITIONS MODEL

FIG. 13D



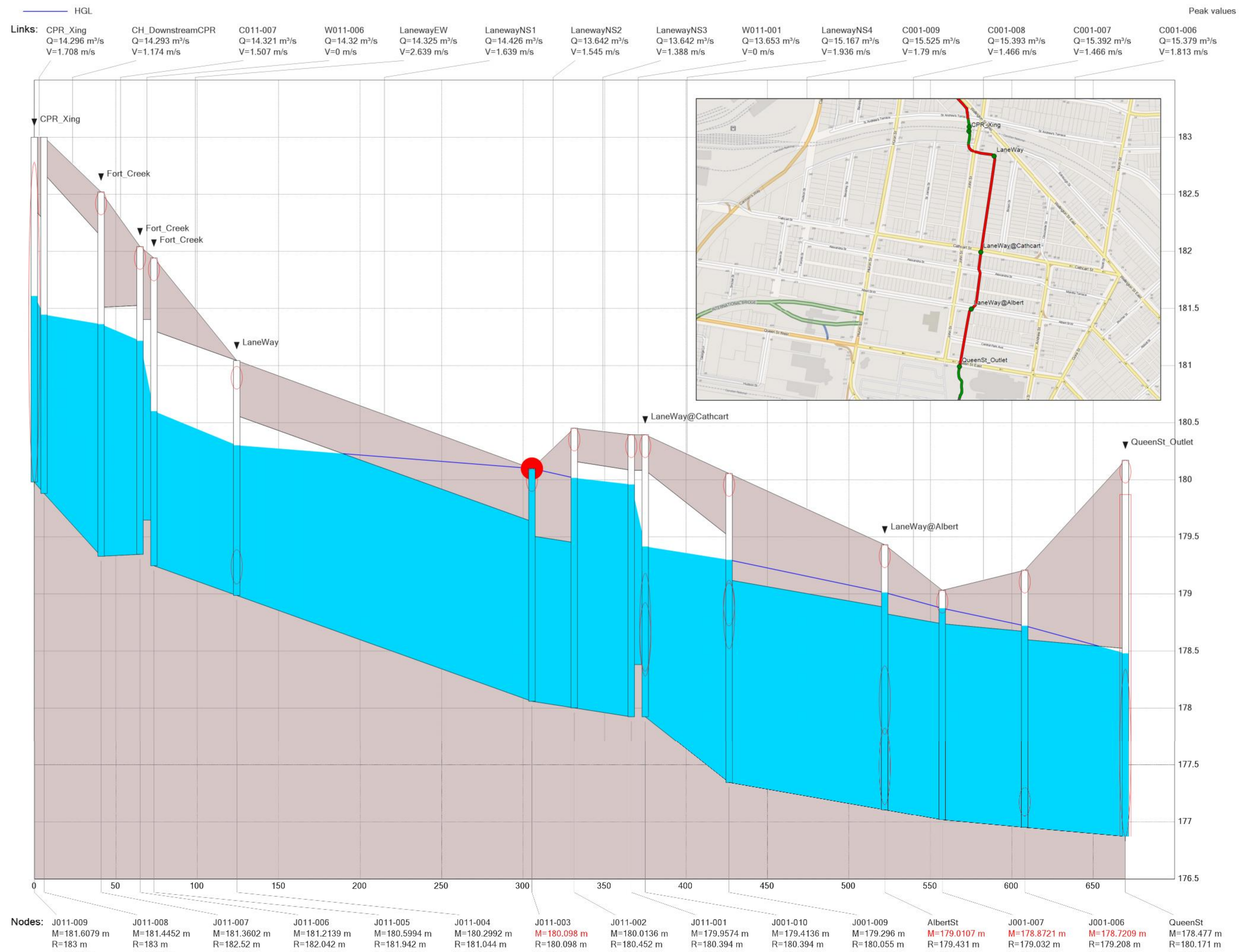
WELLINGTON STREET AQUEDUCT

HYDRAULIC GRADE LINE AND PEAK VALUES

Timmins Regional Storm Event

EXISTING CONDITIONS MODEL

FIG. 14A



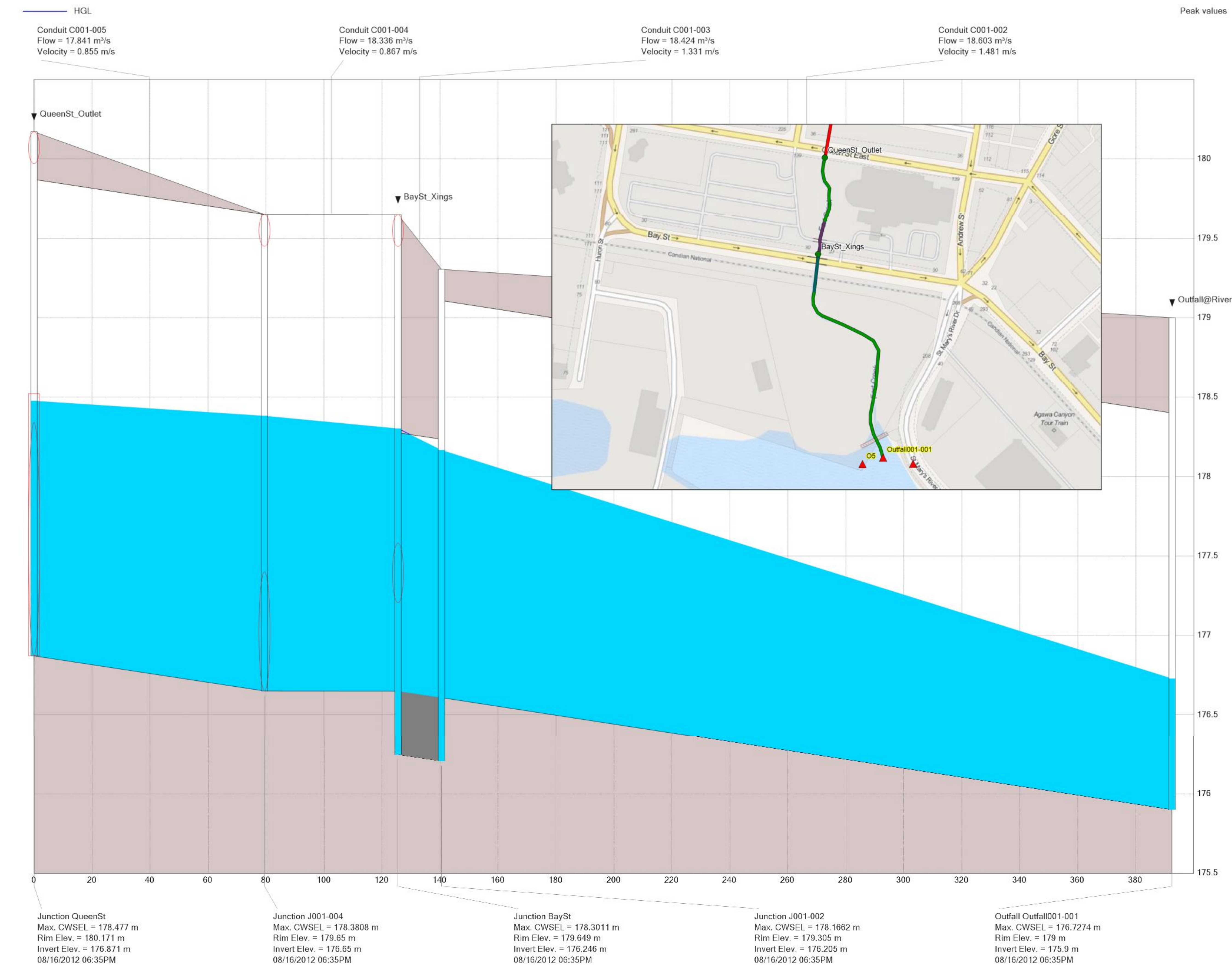
JOHN STREET LANEWAY AQUEDUCT

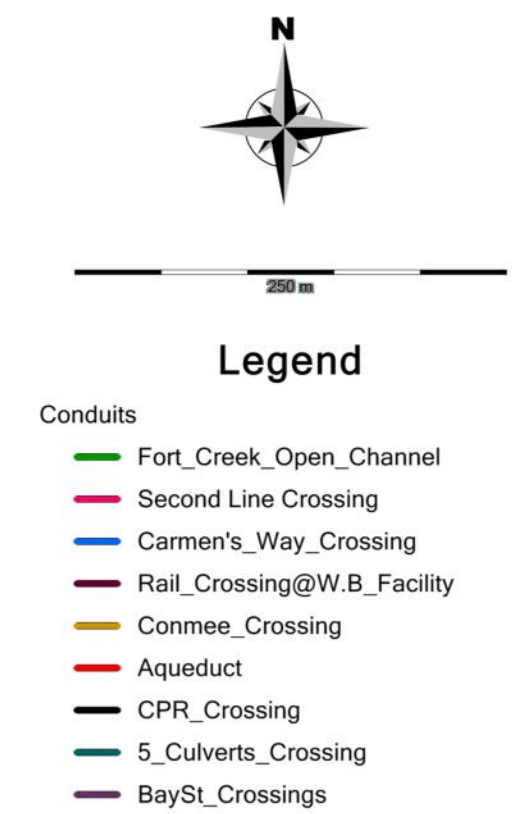
HYDRAULIC GRADE LINE AND PEAK VALUES

Timmins Regional Storm Event

EXISTING CONDITIONS MODEL

FIG. 14B





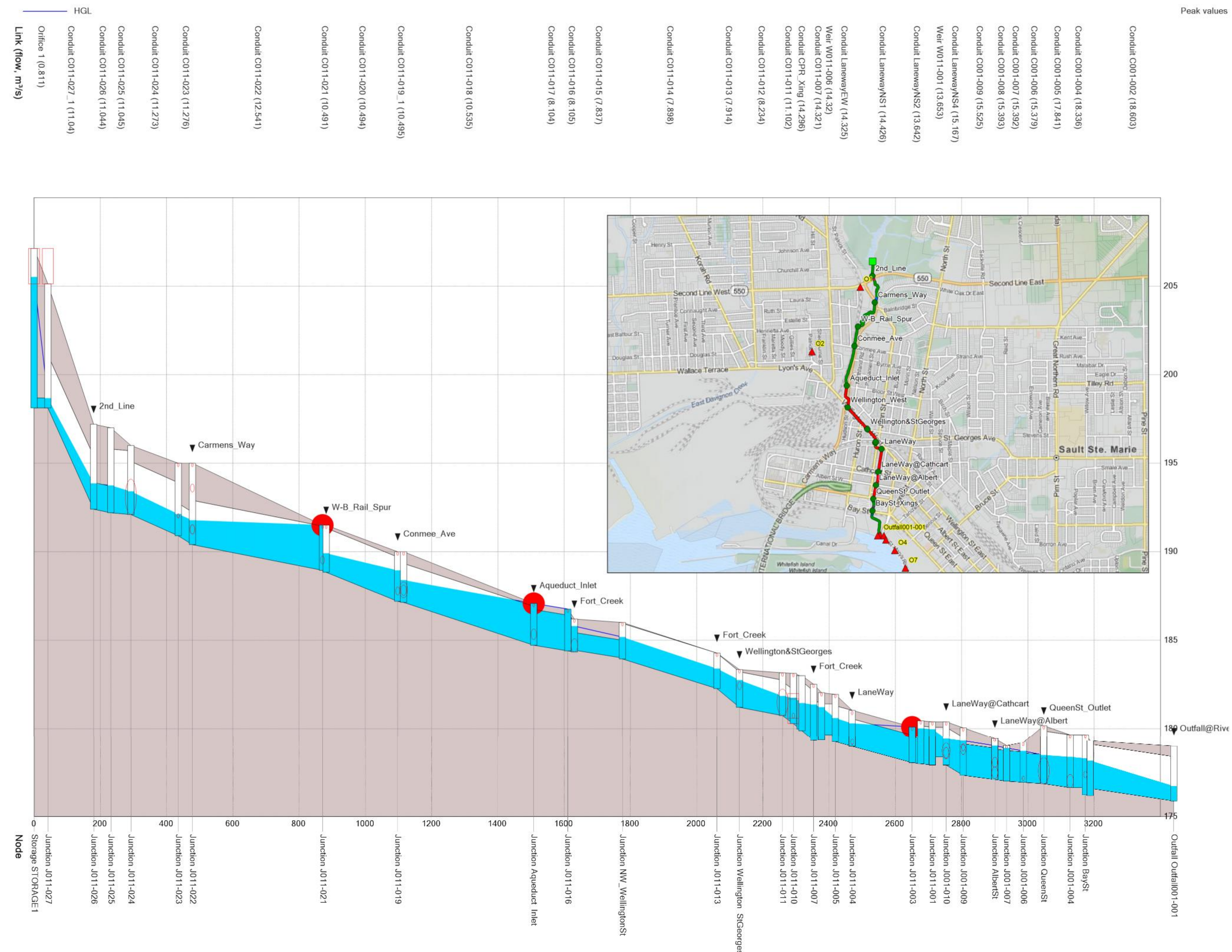
FORT CREEK OPEN CHANNEL SOUTH OF QUEEN ST.

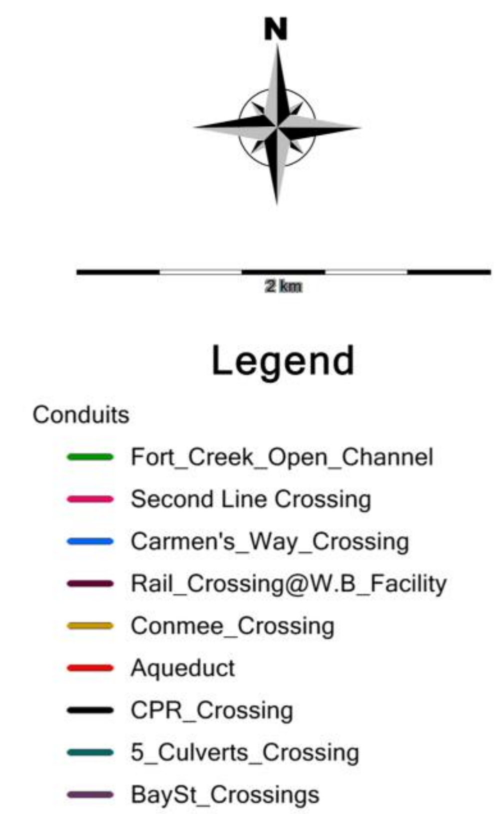
HYDRAULIC GRADE LINE AND PEAK VALUES

Timmins Regional Storm Event

EXISTING CONDITIONS MODEL

FIG. 14C





THE FORT CREEK SYSTEM: DAM TO ST. MARY'S RIVER

HYDRAULIC GRADE LINE AND PEAK VALUES

Timmins Regional Storm Event

EXISTING CONDITIONS MODEL

FIG. 14D

APPENDIX D

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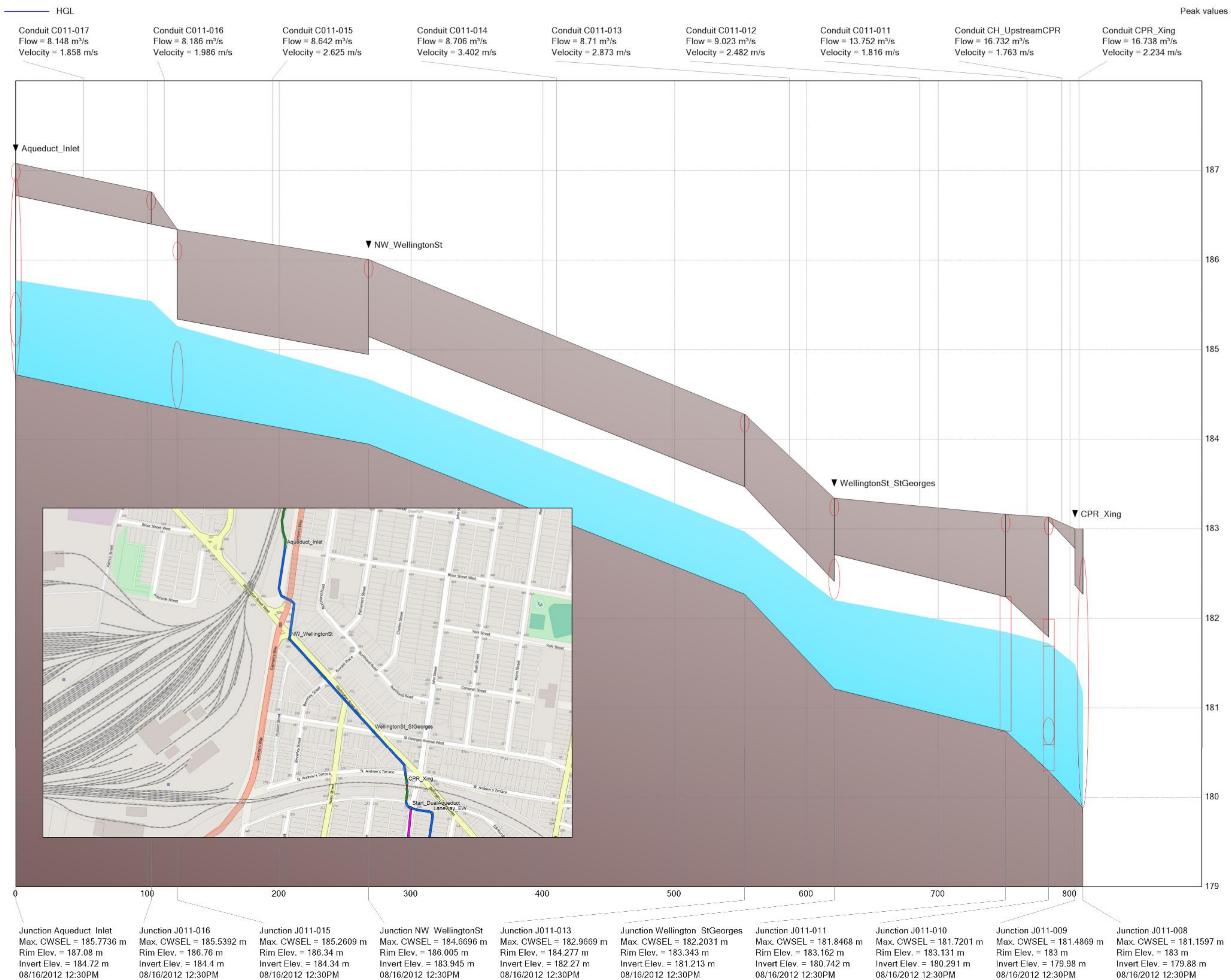
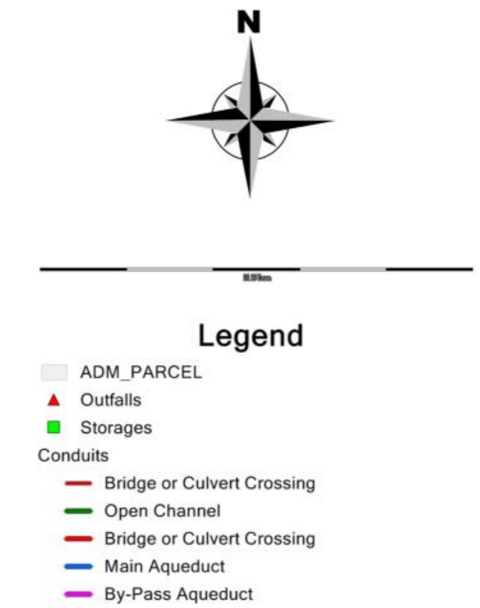
ENGINEERING





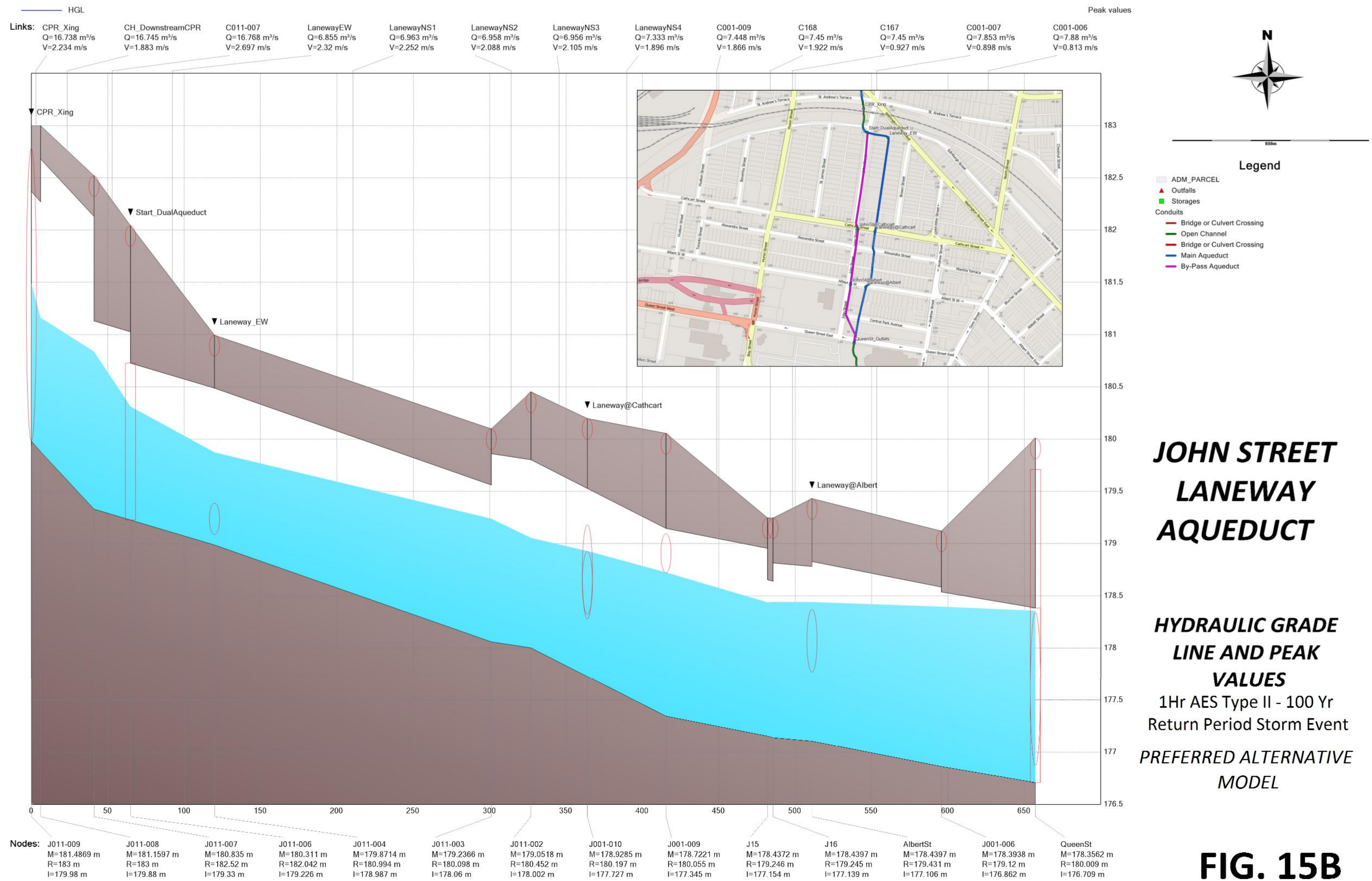
WELLINGTON STREET AQUEDUCT

HYDRAULIC GRADE LINE AND PEAK VALUES

1Hr AES II - 100 Yr Return Period Storm Event

PREFERRED ALTERNATIVE MODEL

FIG. 15A



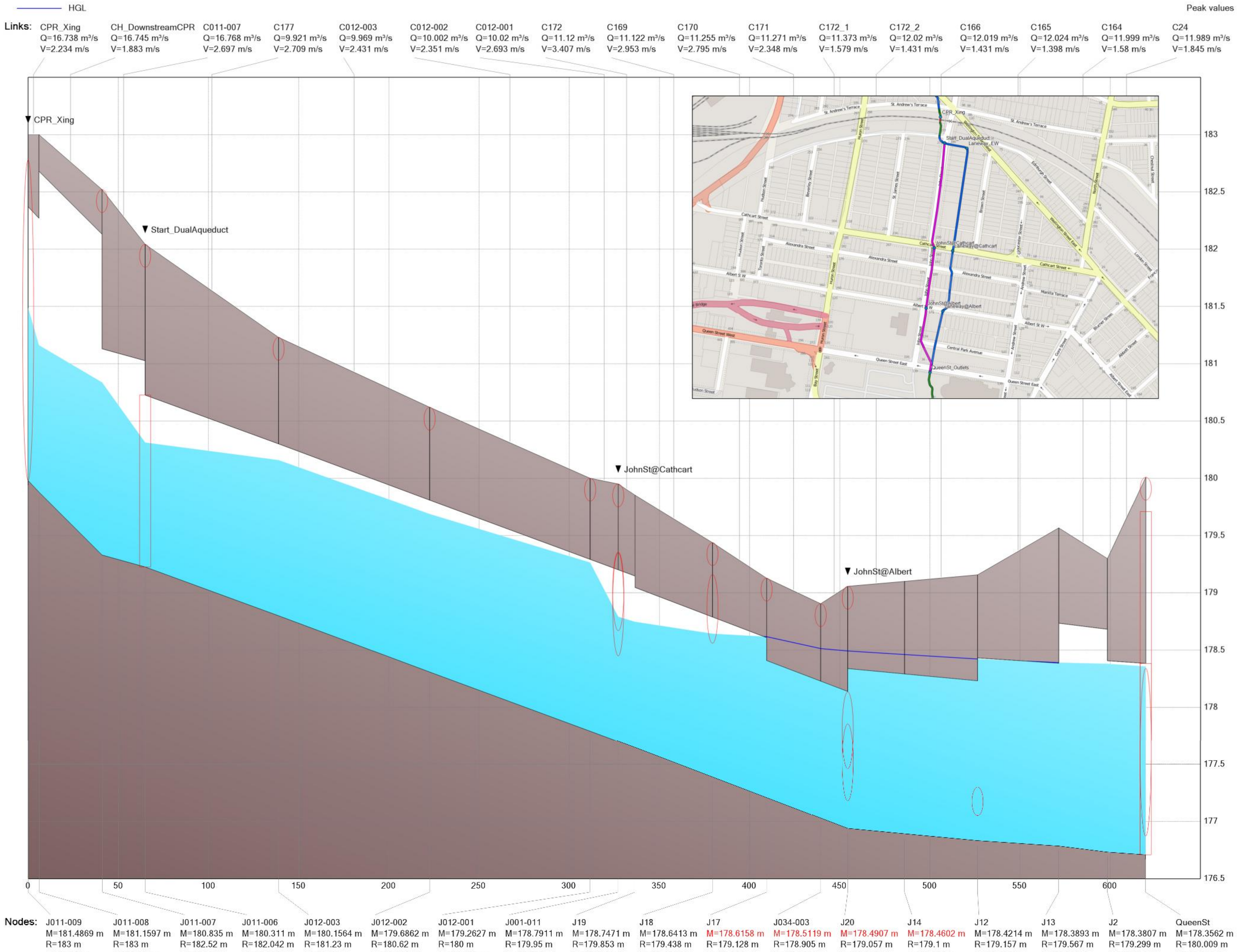
JOHN STREET LANEWAY AQUEDUCT

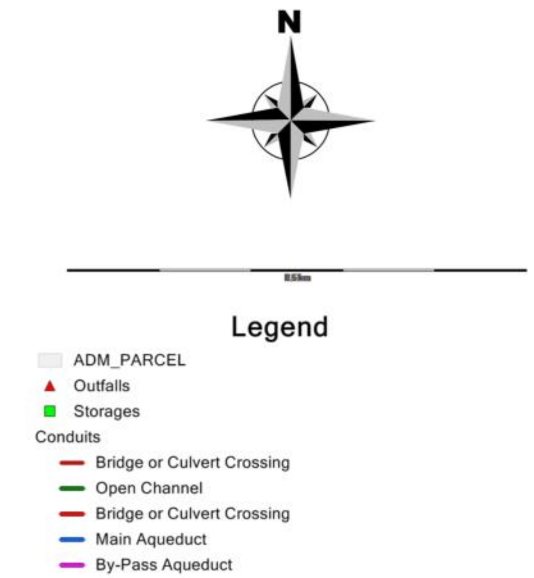
HYDRAULIC GRADE LINE AND PEAK VALUES

1Hr AES Type II - 100 Yr Return Period Storm Event

PREFERRED ALTERNATIVE MODEL

FIG. 15B





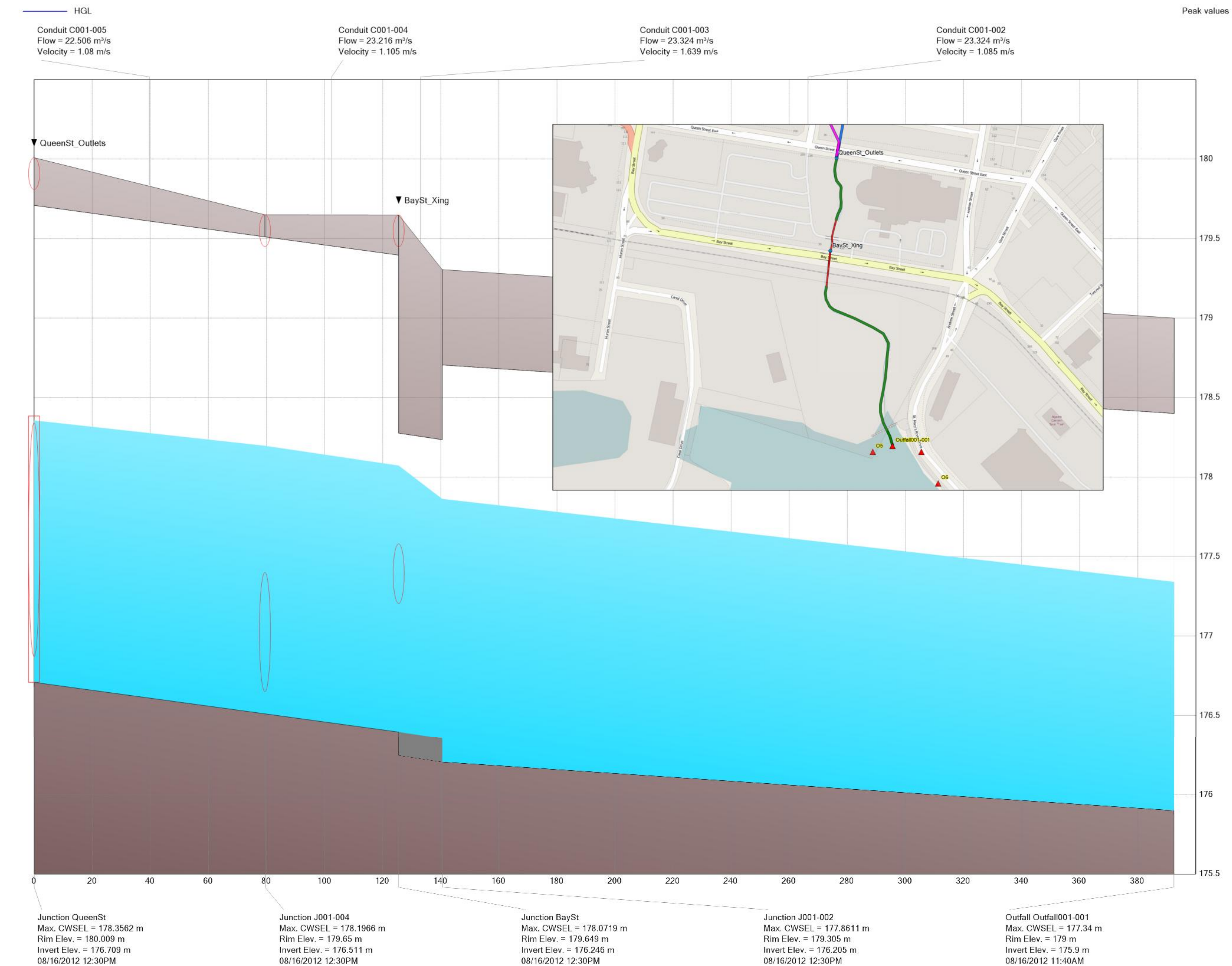
JOHN STREET RELIEF AQUEDUCT

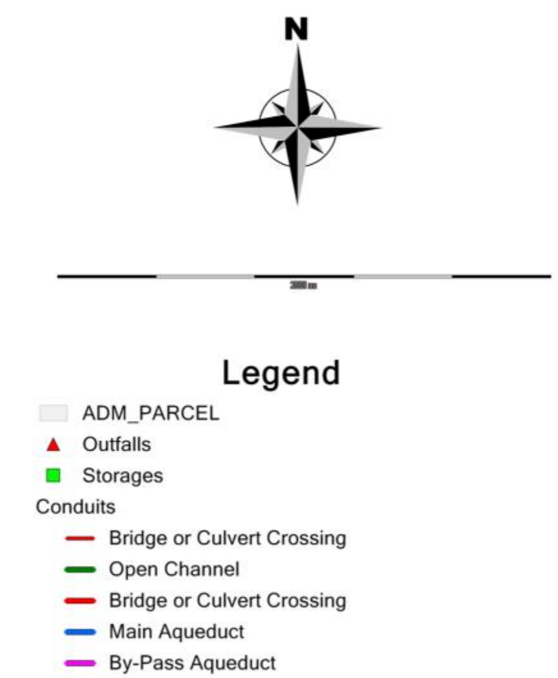
HYDRAULIC GRADE LINE AND PEAK VALUES

1Hr AES Type II - 100 Yr Return Period Storm Event

PREFERRED ALTERNATIVE MODEL

FIG. 15C





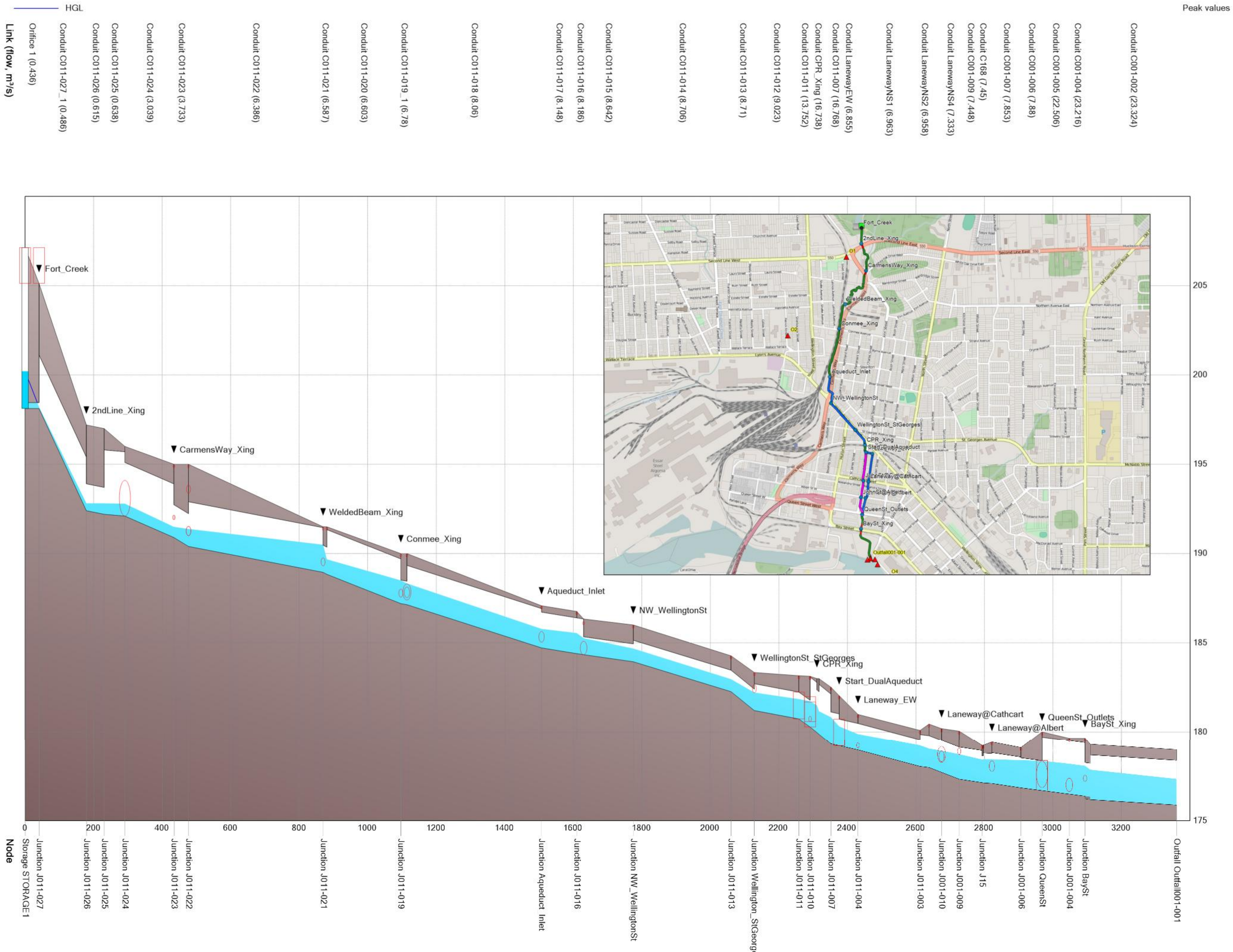
FORT CREEK OPEN CHANNEL SOUTH OF QUEEN ST.

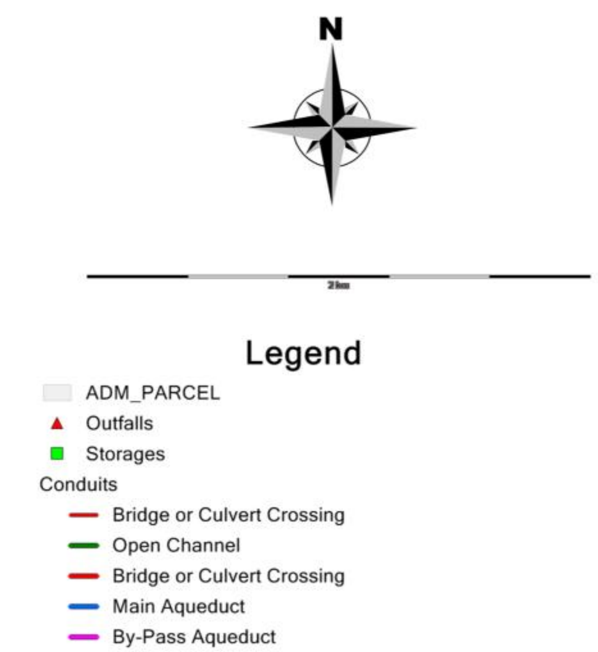
HYDRAULIC GRADE LINE AND PEAK VALUES

1Hr AES Type II - 100 Yr Return Period Storm Event

PREFERRED ALTERNATIVE MODEL

FIG. 15D





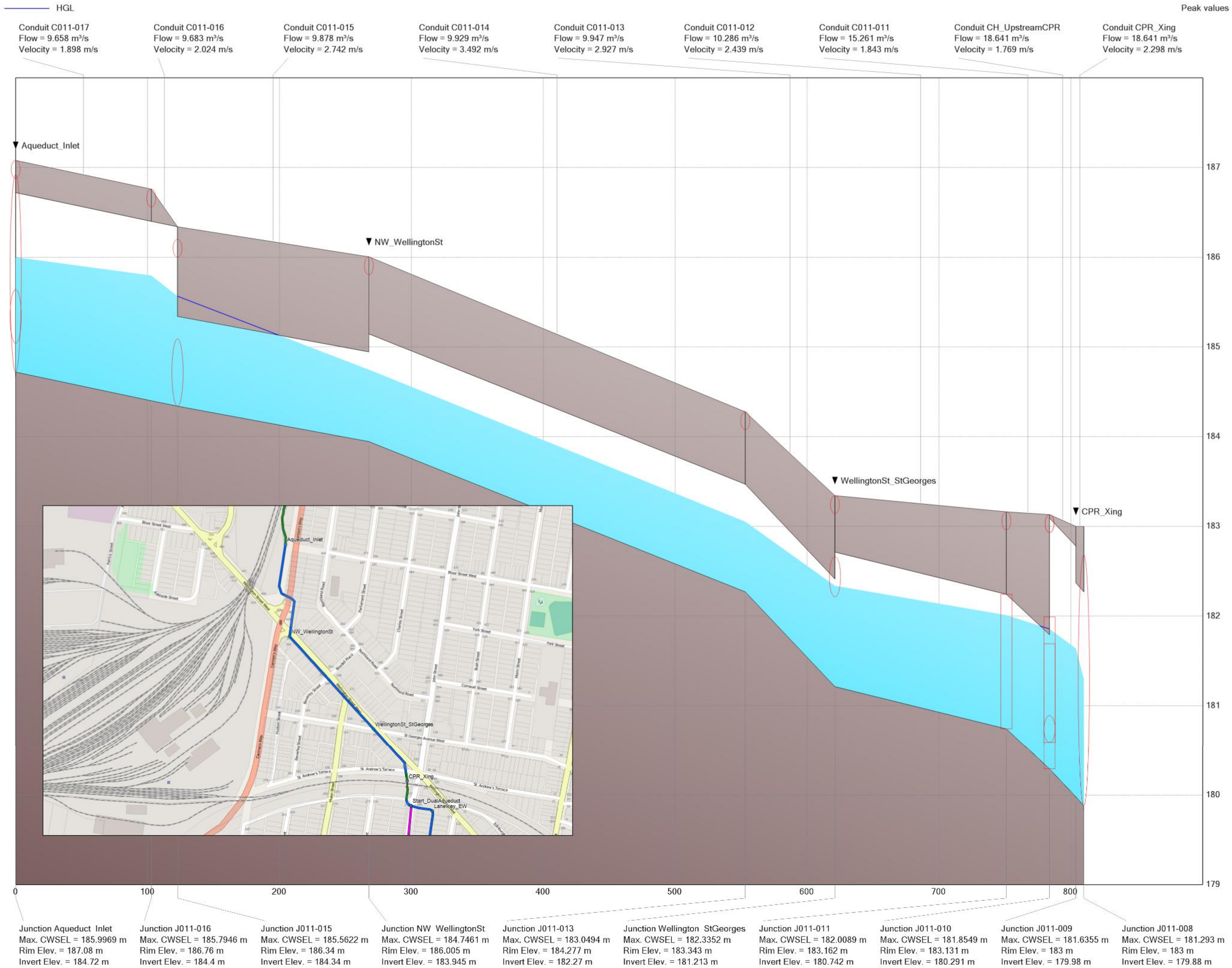
THE FORT CREEK SYSTEM: DAM TO ST. MARY'S RIVER

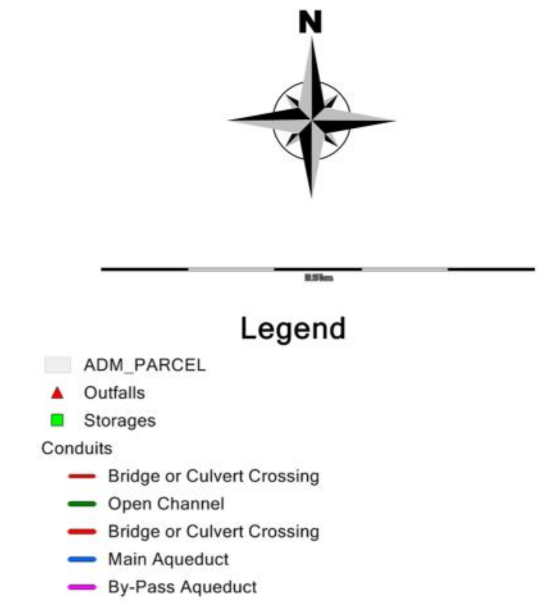
HYDRAULIC GRADE LINE AND PEAK VALUES

1Hr AES Type II - 100 Yr Return Period Storm Event

PREFERRED ALTERNATIVE MODEL

FIG. 15E





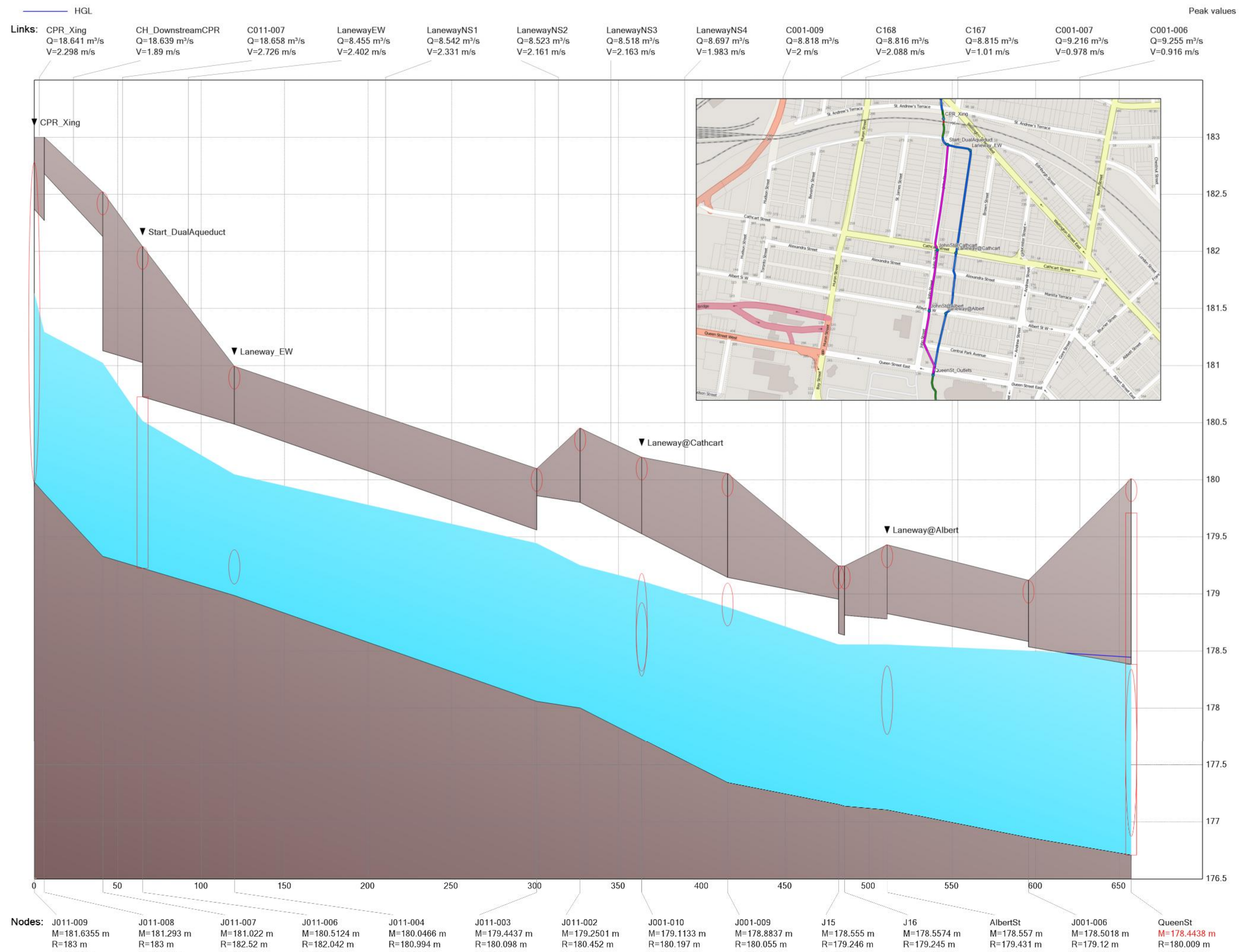
WELLINGTON STREET AQUEDUCT

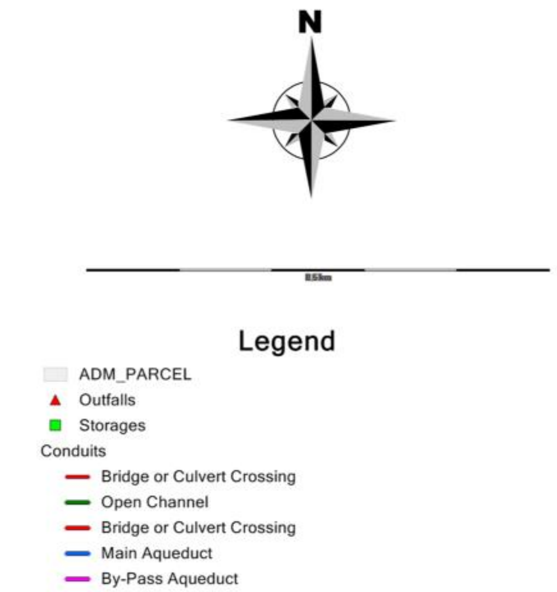
HYDRAULIC GRADE LINE AND PEAK VALUES

24Hr SCS II - 100 Yr Return Period Storm Event

PREFERRED ALTERNATIVE MODEL

FIG. 16A





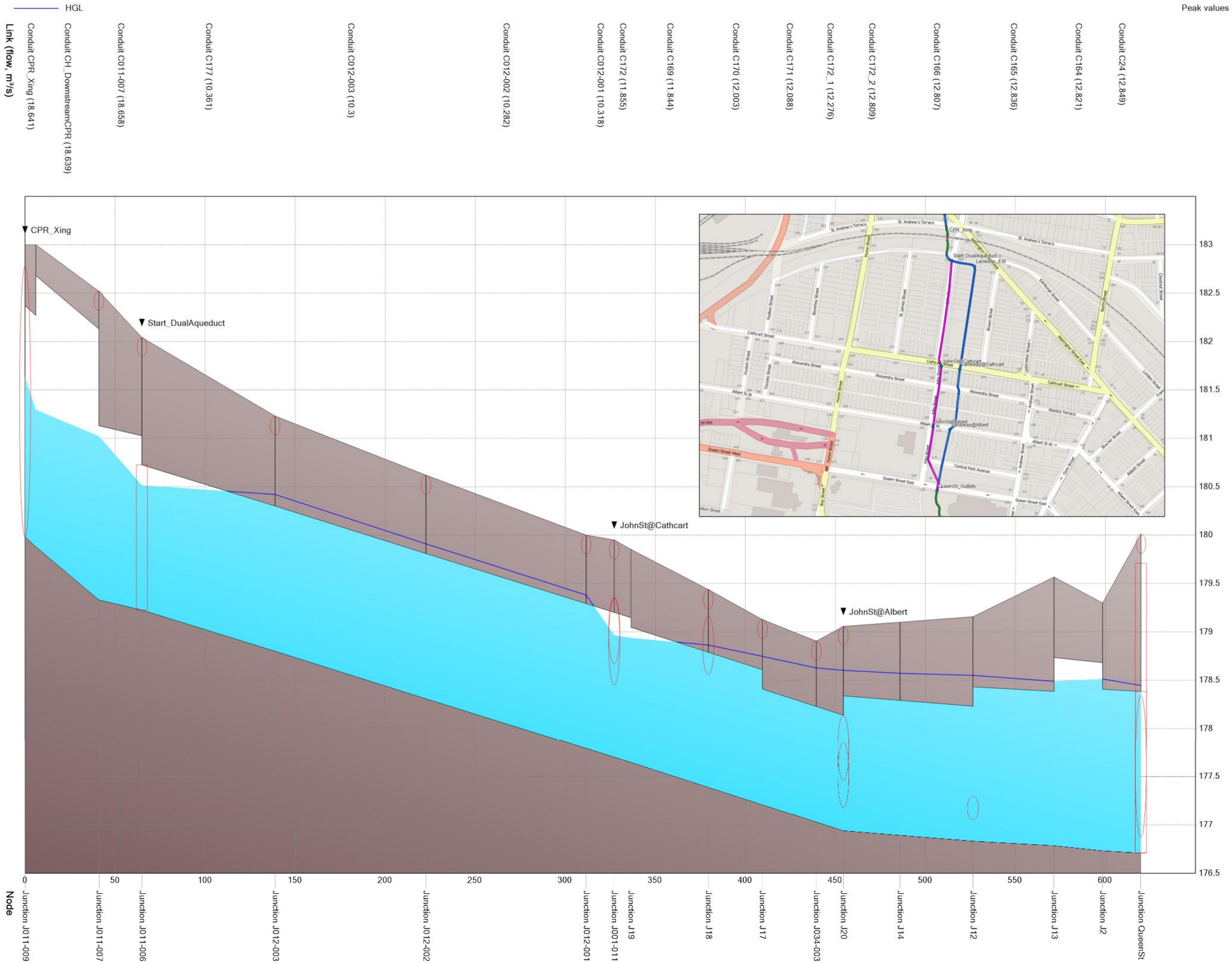
JOHN STREET LANEWAY AQUEDUCT

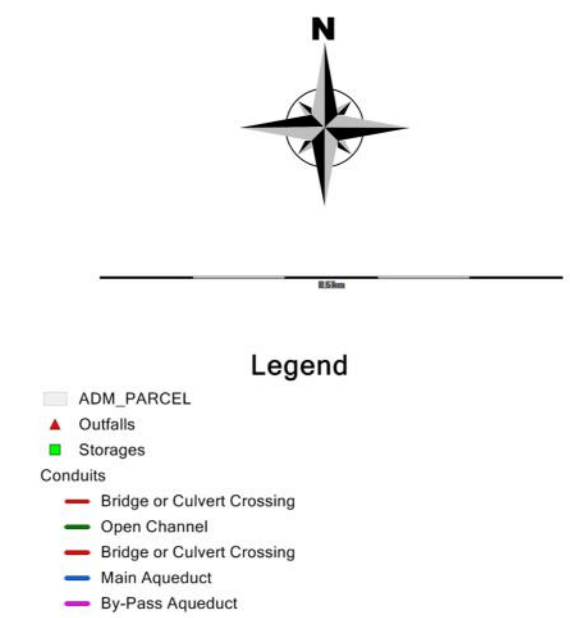
HYDRAULIC GRADE LINE AND PEAK VALUES

24Hr SCS Type II - 100 Yr Return Period Storm Event

PREFERRED ALTERNATIVE MODEL

FIG. 16B





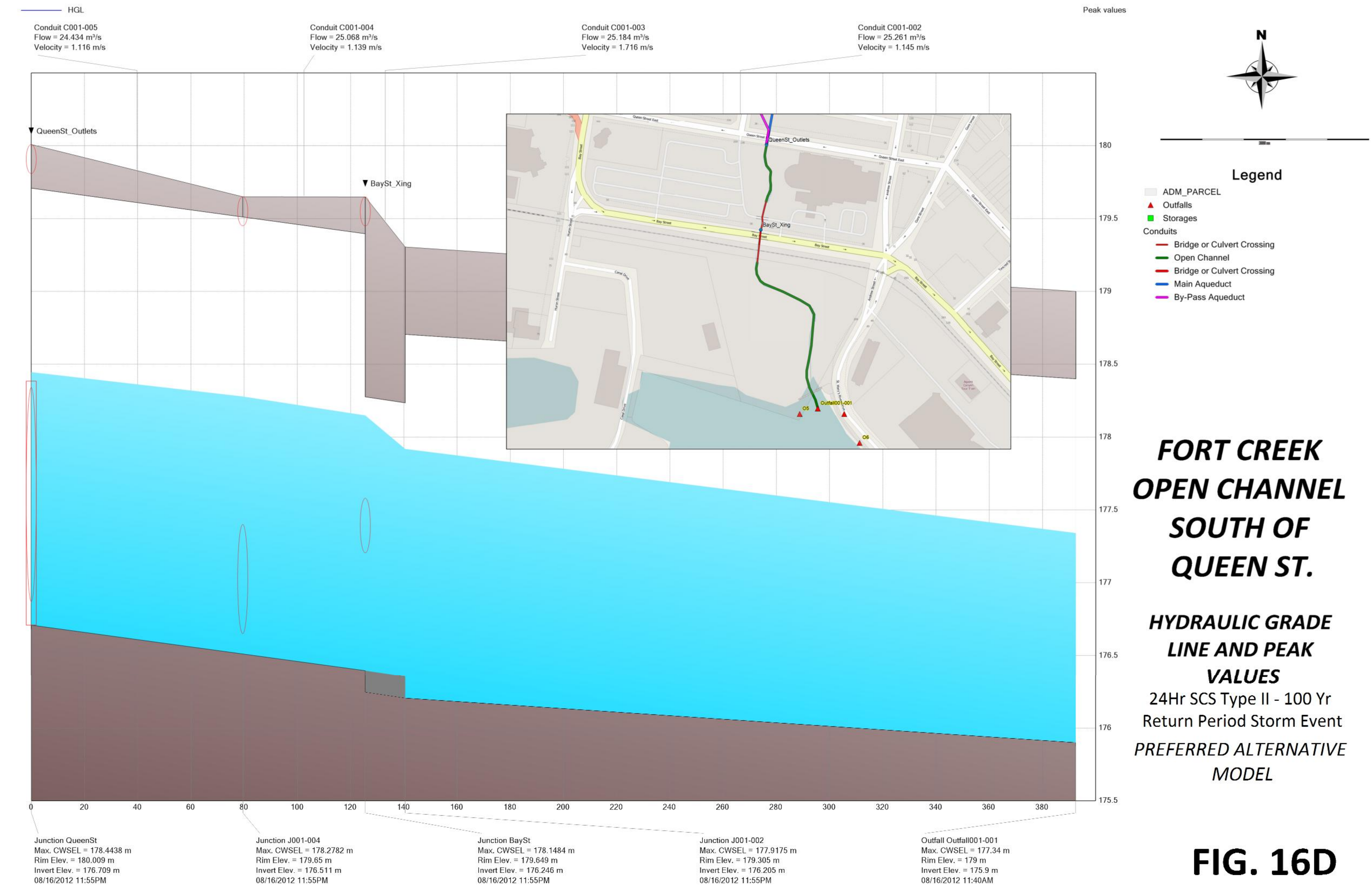
JOHN STREET RELIEF AQUEDUCT

HYDRAULIC GRADE LINE AND PEAK VALUES

24Hr SCS Type II - 100 Yr Return Period Storm Event

PREFERRED ALTERNATIVE MODEL

FIG. 16C



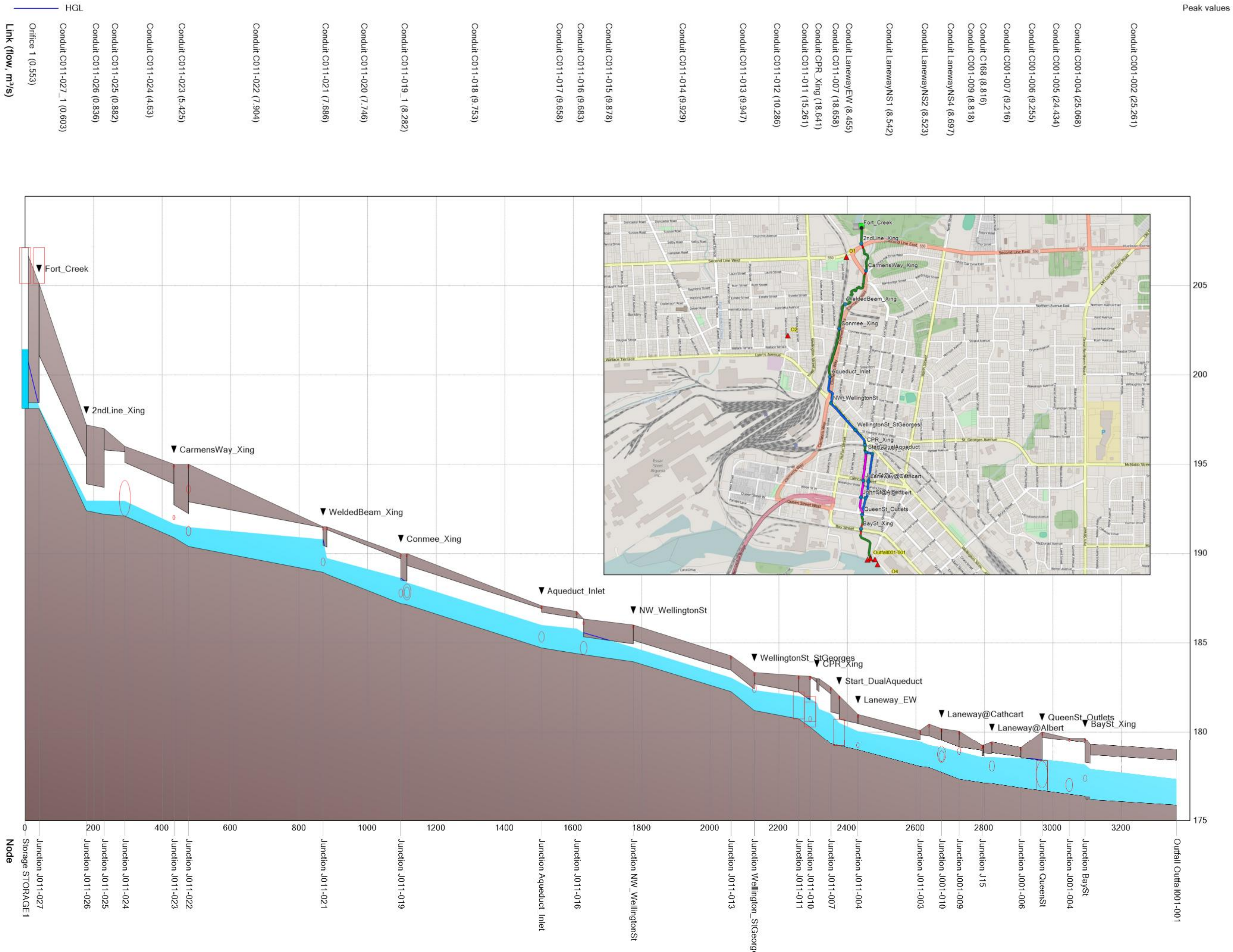
FORT CREEK OPEN CHANNEL SOUTH OF QUEEN ST.

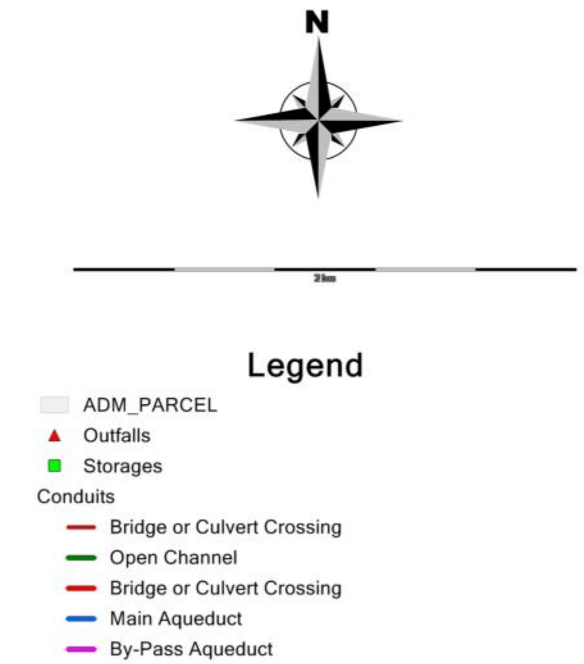
HYDRAULIC GRADE LINE AND PEAK VALUES

24Hr SCS Type II - 100 Yr Return Period Storm Event

PREFERRED ALTERNATIVE MODEL

FIG. 16D





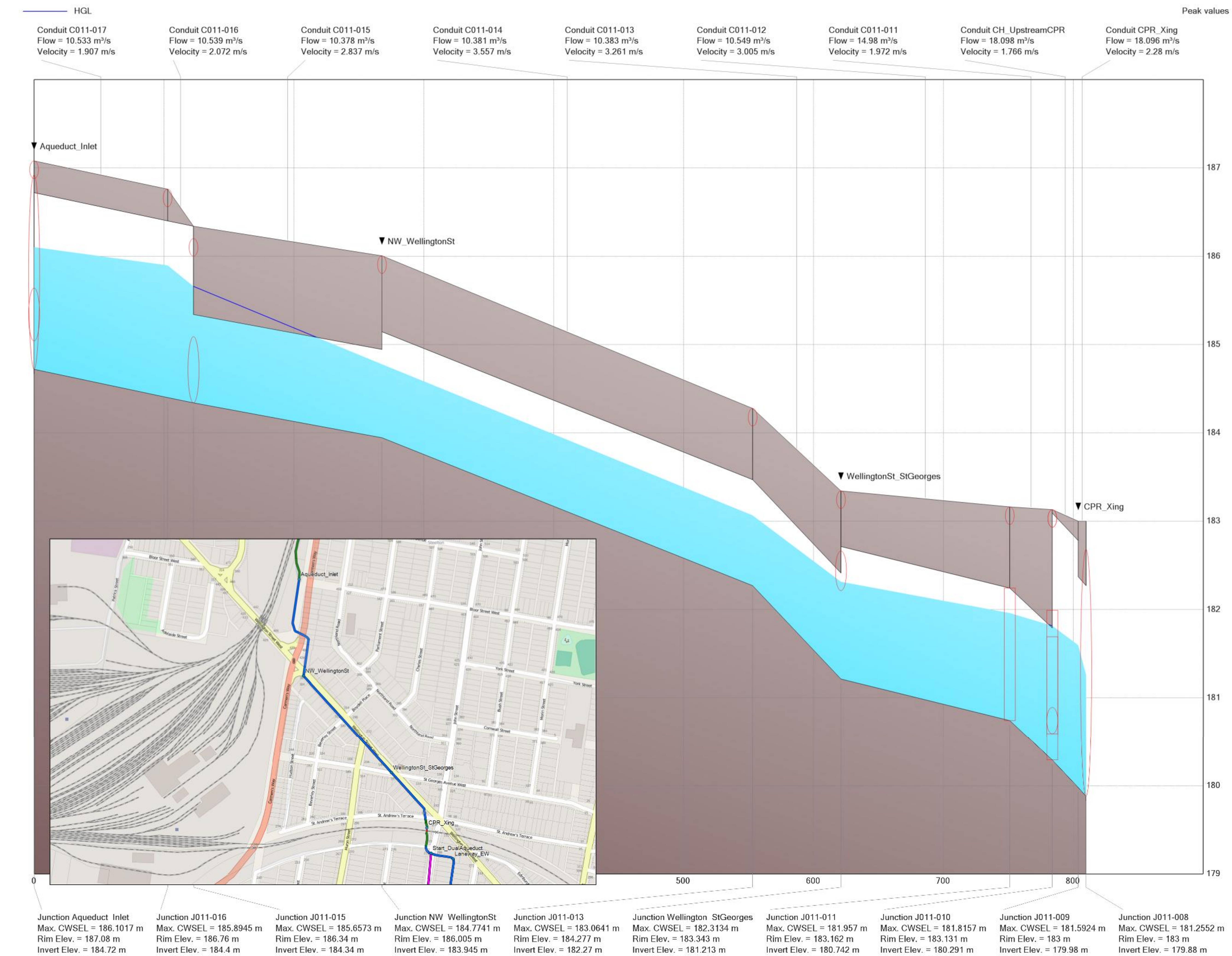
THE FORT CREEK SYSTEM: DAM TO ST. MARY'S RIVER

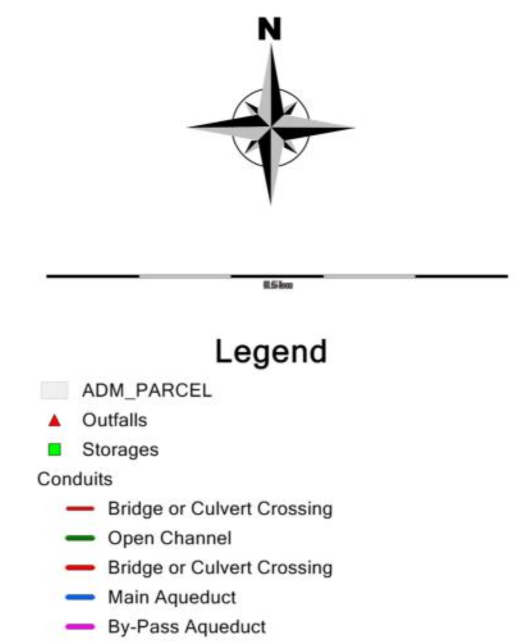
HYDRAULIC GRADE LINE AND PEAK VALUES

24Hr SCS Type II - 100 Yr Return Period Storm Event

PREFERRED ALTERNATIVE MODEL

FIG. 16E





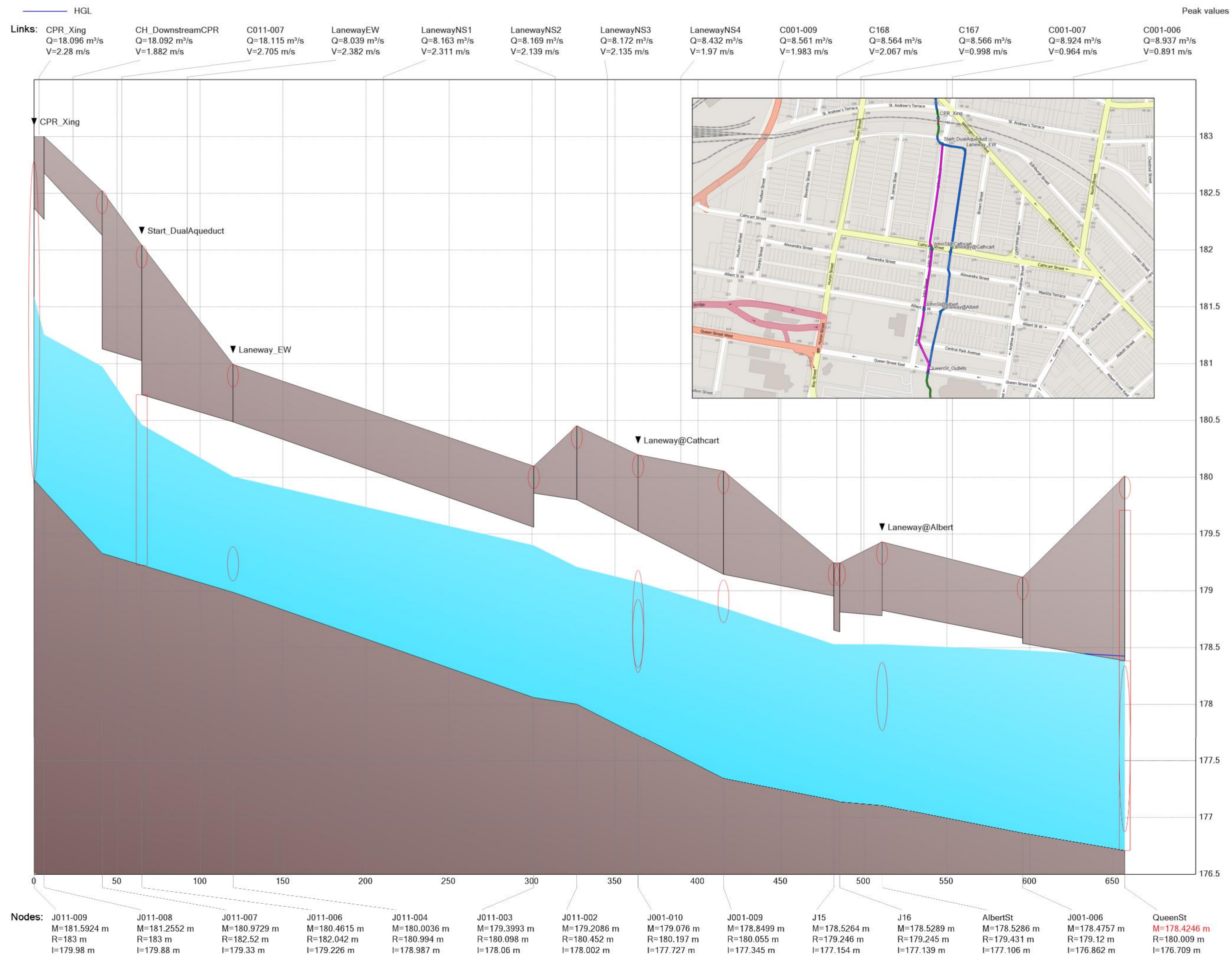
WELLINGTON STREET AQUEDUCT

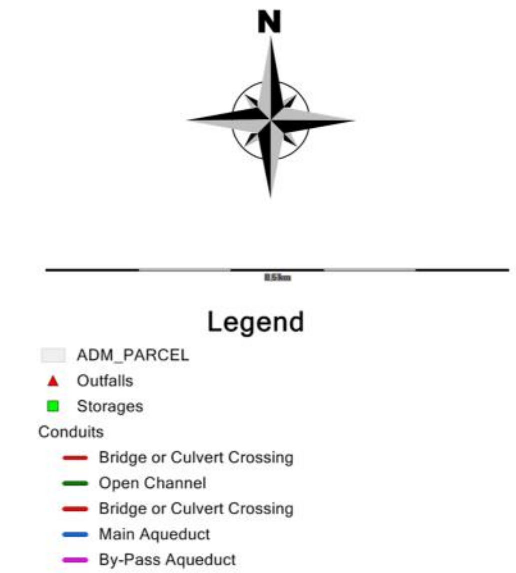
HYDRAULIC GRADE LINE AND PEAK VALUES

Timmins Regional Storm Event

PREFERRED ALTERNATIVE MODEL

FIG. 17A





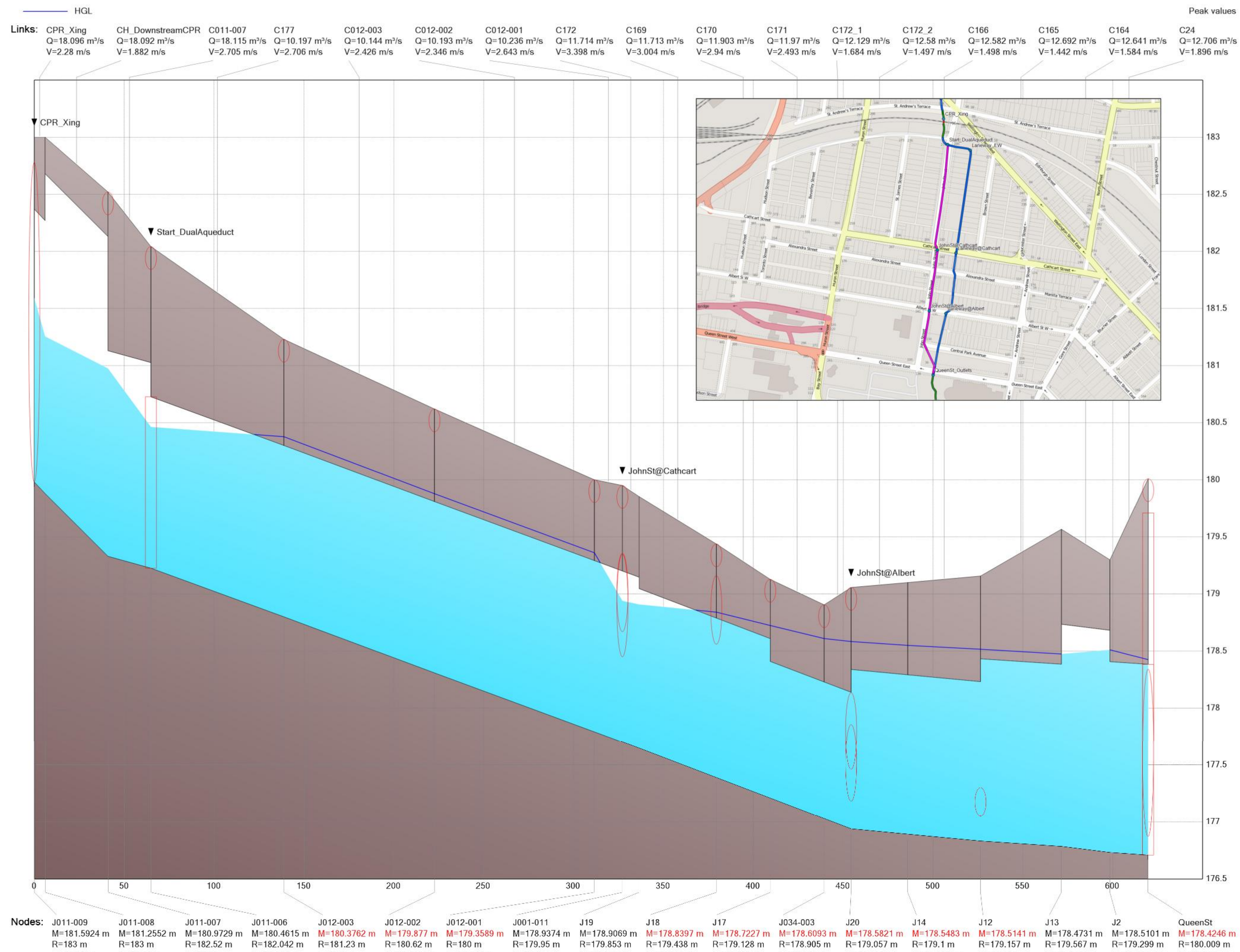
JOHN STREET LANEWAY AQUEDUCT

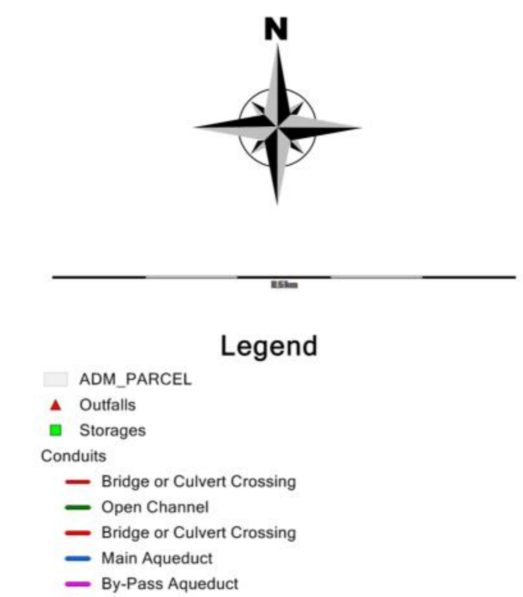
HYDRAULIC GRADE LINE AND PEAK VALUES

Timmins Regional Storm Event

PREFERRED ALTERNATIVE MODEL

FIG. 17B





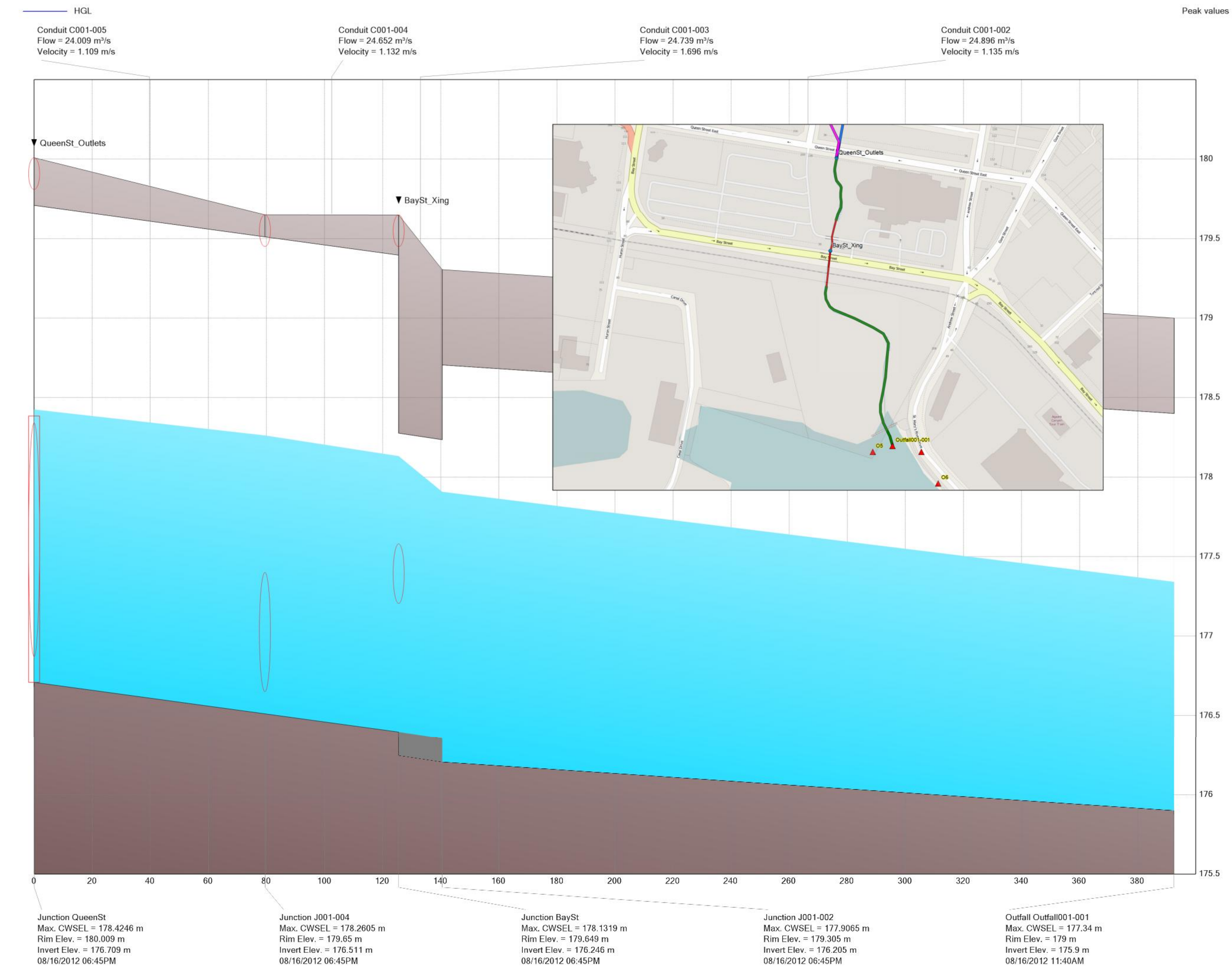
JOHN STREET RELIEF AQUEDUCT

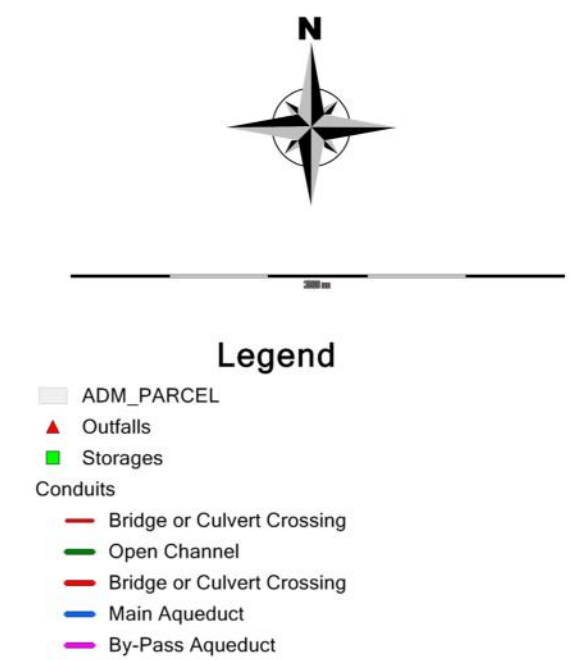
HYDRAULIC GRADE LINE AND PEAK VALUES

Timmins Regional Storm Event

PREFERRED ALTERNATIVE MODEL

FIG. 17C





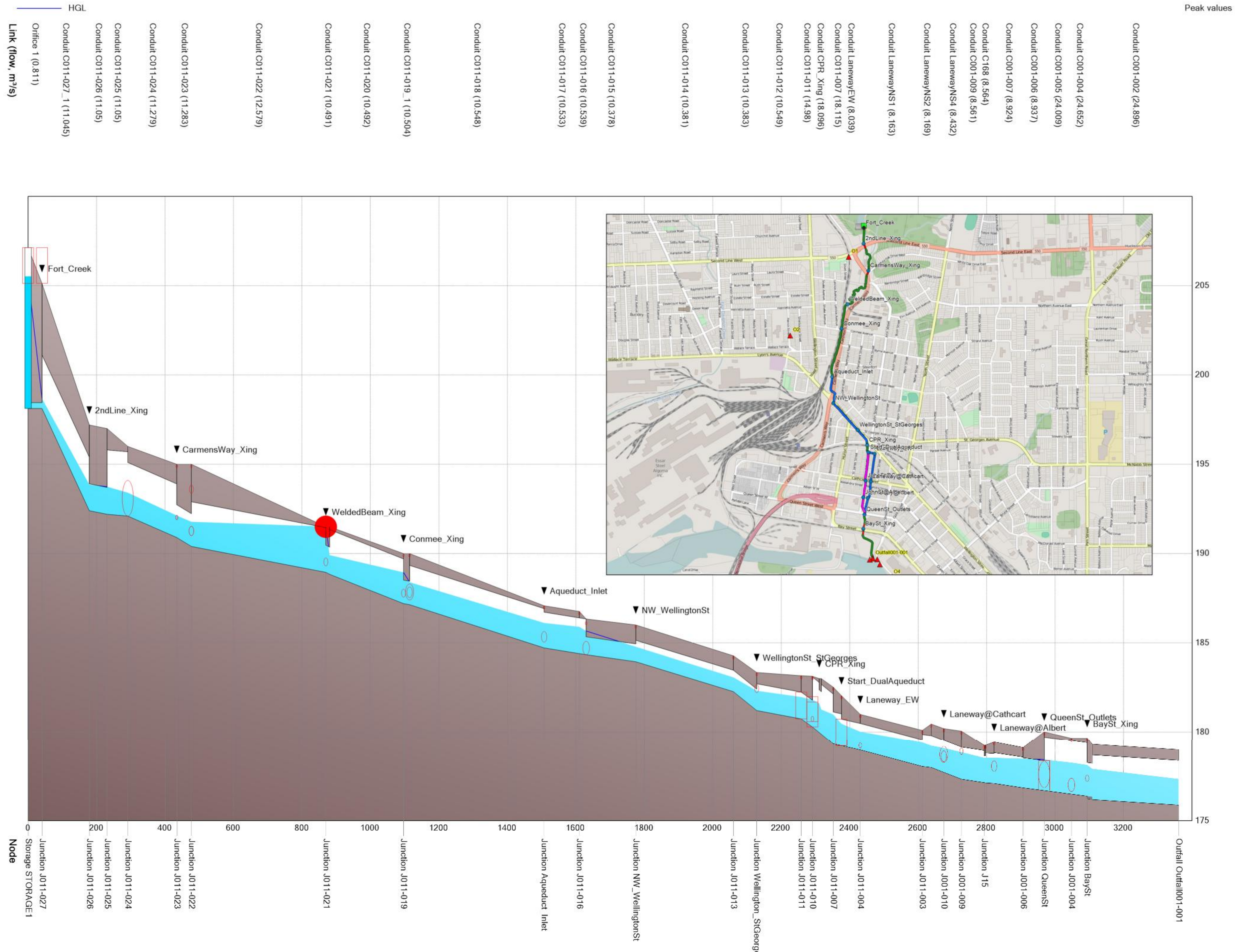
FORT CREEK OPEN CHANNEL SOUTH OF QUEEN ST.

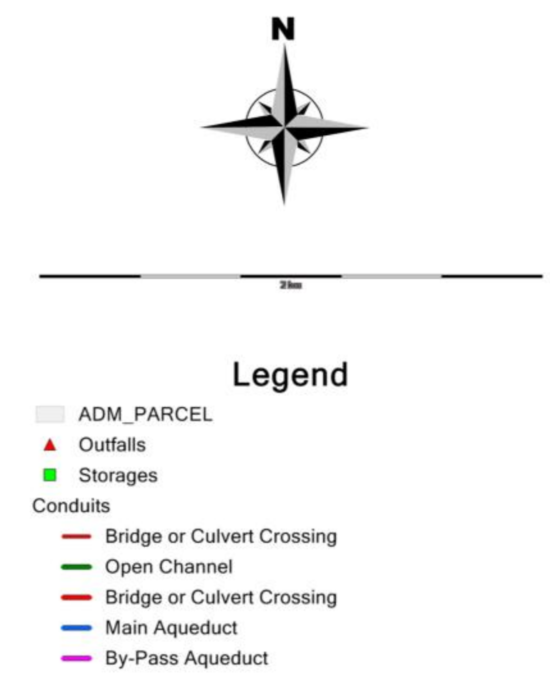
HYDRAULIC GRADE LINE AND PEAK VALUES

Timmins Regional Storm Event

PREFERRED ALTERNATIVE MODEL

FIG. 17D





THE FORT CREEK SYSTEM: DAM TO ST. MARY'S RIVER

HYDRAULIC GRADE LINE AND PEAK VALUES

Timmins Regional Storm Event

PREFERRED ALTERNATIVE MODEL

FIG. 17E