3.2 HYDROGEOLOGY

A hydrogeological report was completed by DST Consulting engineers in order to assess the existing hydrogeological conditions of the Former CFB Rockcliffe CDP site and to determine the expected potential impacts of the proposed redevelopment on groundwater and groundwater users (see Appendix A (4)).

General

It is important to note that there are two connected hydrogeological units in the area: the overburden unit and the shallow bedrock unit.

Groundwater Elevations

Groundwater on site flows from north to northwest, approximately following the local topography descending towards the Ottawa River.

Water table measurements were collected in 2006 and 2013 and subsequently in 2014 and are presented in Table 3 and Table 4 respectively.

In general, the overburden water table is slightly higher than the bedrock groundwater surface across much of the site. However, in the northern portion of the property, approaching the top of the escarpment, groundwater in the shallow bedrock has a higher piezometric head than the overburden water table, interpreted to be due to greater topographic control on the overburden water table than on the horizontally bedded bedrock.

All except one of the monitoring wells installed in bedrock which were assessed in 2013 were screened across shallow bedrock depths (between approximately 1 to 5m below the bedrock surface), and many include shallow zones of blocky and/or faulted limestone. Monitoring well BHMW 10 BR (2006) was installed with a screen at 8 to 11 m depth below the bedrock surface, below the fractured bedrock, and is noted as having a lower piezometric head than the adjacent monitoring wells. For this reason, it was interpreted by DST that the groundwater elevation in this monitoring well represents a deeper bedrock hydrogeological unit with a possible limited hydraulic connection to the shallow bedrock monitoring wells.

Horizontal hydraulic gradients were also estimated for the Former CFB Rockcliffe CDP site. The horizontal hydraulic gradient in the overburden is estimated to be approximately 0.038, and the horizontal hydraulic gradient in the shallow bedrock is estimated to range from approximately 0.008 to 0.06. The hydraulic gradient increases for both overburden and shallow bedrock groundwater from the North of the Former CFB Rockcliffe CDP site across the escarpment.

From the 2014 data analyzed by Aquafor Beech, the seasonal fluctuations of the water table appear to vary between 0.3 and 2 metres. Due to the proximity of the water table to the existing ground surface (note- a 1-2m grade raise is proposed for the site) and the proposed use of infiltration based SWM controls, it is possible that long-term (annual) monitoring may be required.

Groundwater Gradient

From the groundwater elevations, DST developed groundwater contours which indicate that contours of the groundwater surfaces based on the groundwater elevations the overburden and bedrock groundwater flow directions are north to northwest, generally following the local topography descending towards the Ottawa River. Overburden and shallow bedrock groundwater potentiometric contours measured in September, 2013 are plotted on Figure 9 and Figure 10.

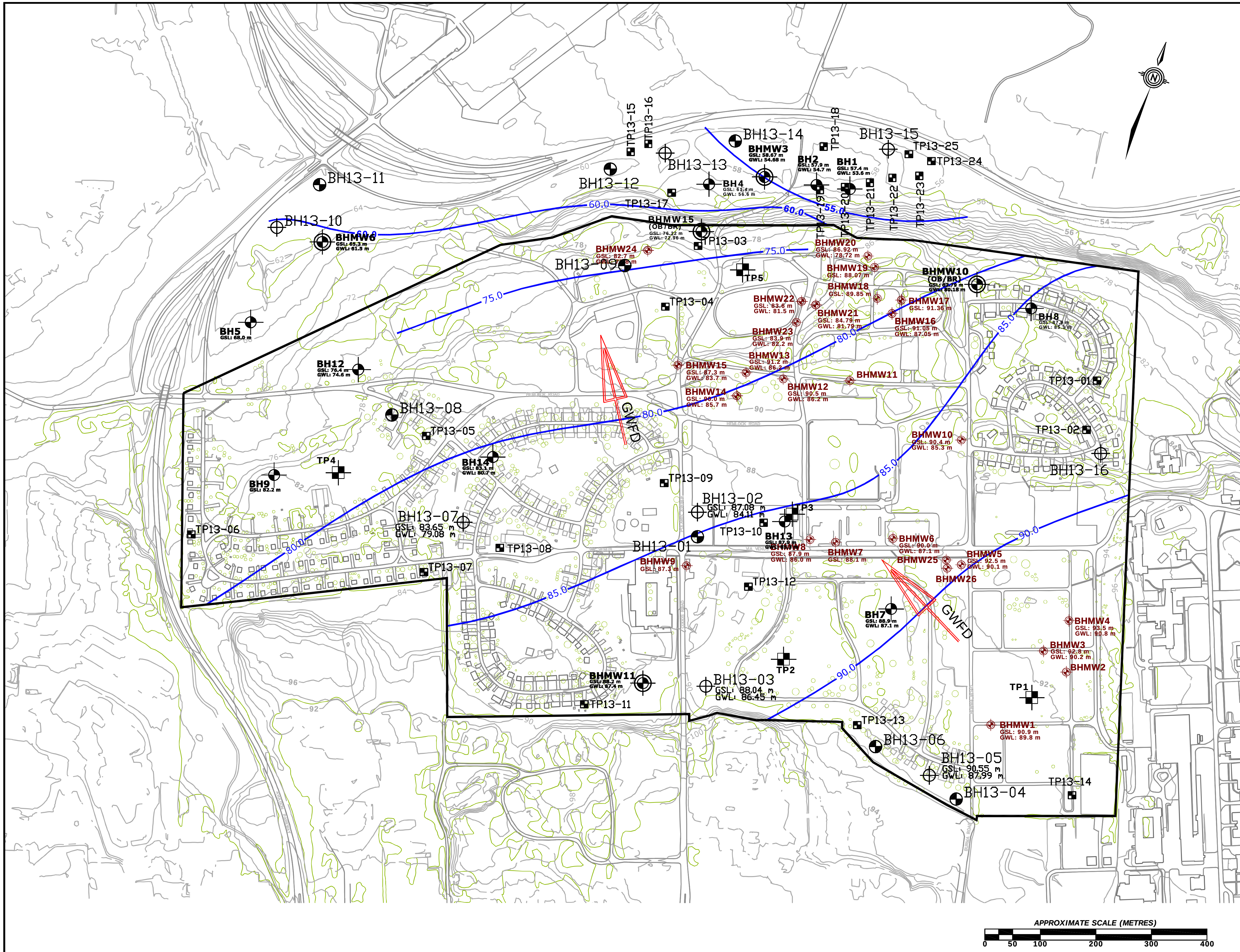
Table 3: Groundwater Elevation Measurements, 2006 and 2013 (NAD 83, Zone 18)

Monitoring Well ID (Year Drilled)	Screened Layer	Material Type	Surface Elevation (m)	Water Level - from Ground Surface (m)				Groundwater Elevation (m)			
				24-Aug-06	12-Jul-13	18-Jul-13	16-Sep-13	24-Aug-06	12-Jul-13	18-Jul-13	16-Sep-13
BHMW3 (2006)	Overburden	Gravel	58.67	3.6	-	-	3.99	55.07	-	-	54.68
BHMW10 BR (2006)	Bedrock	Limestone	82.79	4.9	9.56	9.59	9.73	77.89	73.23	73.20	73.06
BHMW10 OB (2006)	Overburden	Sand	82.79	2.5	2.31	2.42	2.61	80.29	80.48	80.37	80.18
BHMW11 (2006)	Overburden	Clay	88.2	0.8	0.74	0.60	1.05	87.40	87.46	87.60	87.15
BHMW15 OB (2006)	Overburden	Clay and weathered bedrock	76.22	3.2	3.09	3.15	3.26	73.02	73.13	73.07	72.96
BHMW2 (2004)	Bedrock	Limestone	-	3	-	2.97	3.04	-	-	-	-
BHMW3 (2004)	Bedrock	Limestone	92.8	2.6	2.19	2.20	2.23	90.20	90.61	90.60	90.57
BHMW4 (2004)	Bedrock	Limestone	93.49	2.7	-	2.66	2.47	90.79	-	90.83	91.02
BHMW5 (2004)	Overburden	Sand (fill) and silty sand (till)	92.51	2.4	-	2.54	2.91	90.11	-	89.97	89.60
BHMW6 (2004)	Overburden	Clay and sand	90.03	2.9	-	3.20	3.23	87.13	-	86.83	86.80
BHMW8 (2004)	Overburden	Clay	87.93	1.9	-	2.72	2.82	86.03	-	85.21	85.11
BHMW10 (2004)	Bedrock	Limestone	90.39	5.1	-	4.98	4.93	85.29	-	85.41	85.46
BHMW12 (2004)	Bedrock	Limestone	90.49	4.3	-	3.98	4.34	86.19	-	86.51	86.15
BHMW13 (2004)	Bedrock	Limestone	91.21	5	-	4.85	5.14	86.21	-	86.36	86.07
BHMW14 (2004)	Bedrock	Limestone	90.01	4.3	-	3.95	4.23	85.71	-	86.06	85.78
BHMW15 (2004)	Bedrock	Limestone	87.32	3.6	-	3.54	3.78	83.72	-	83.78	83.54
BHMW16 (2004)	Bedrock	Limestone - blocky, faulted	91.05	4	-	3.90	4.26	87.05	-	87.15	86.79
BHMW20 (2004)	Bedrock	Limestone - faulted	86.92	8.2	-	-	8.55	78.72	-	-	78.37
BHMW21 (2004)	Bedrock	Limestone	84.79	3	-	1.65	2.17	81.79	-	83.14	82.62
BHMW22 (2004)	Bedrock	Limestone - faulted	83.64	2.1	1.82	2.02	2.56	81.54	81.82	81.62	81.08
BHMW23 (2004)	Bedrock	Limestone	83.95	1.8	-	1.70	2.00	82.15	-	82.25	81.95
BHMW24 (2004)	Bedrock	Limestone - fractured	82.69	4.9	3.66	3.78	4.27	77.79	79.03	78.91	78.42
BHMW25 (2004)	Overburden	Sand (fill) and sand-silt (till)	-	-	-	2.44	2.75	-	-	-	-
BHMW26 (2004)	Overburden	Sand and gravel (fill)	-	-	-	2.34	2.84	-	-	-	-
BH13-02 (2013)	Overburden	Clay, sand	87.08	84.11	-	-	2.97	-	-	-	84.11
BH13-03 (2013)	Overburden	Clay	88.04	86.45	-	-	1.59	-	-	-	86.45
BH13-05 (2013)	Overburden	Clay	90.55	87.99	-	-	2.56	-	-	-	87.99
BH13-07 (2013)	Overburden	Clay	83.65	79.08	-	-	4.57	-	-	-	79.08

Note:
'-' denotes information unavailable

Table 4: Ranges of Water Levels Measured by DST in 2014 for various Monitor Wells (NAD 83, Zone 18) – (Adapted from DST BH, 2014)

Location	Water Level (metres below ground surface - 2014)	Measurement range, prior to 2014 (meres below ground surface)	Range of water levels (metres)
BH14-29	2.7	3	0.3
BHMW6	-	-	-
BHMW8	2.39	1.9 – 2.82	0.92
BHMW10	-	-	-
BHMW12	-	3.98 – 4.34	0.36
BHMW13	-	4.85 – 5.14	0.29
BHMW14	-	3.95 - 4.23	0.28
BHMW15	-	3.6 – 3.78	0.18
BHMW16	4.06	4.0 – 4.26	0.26
BHMW20	8.26	8.2 – 8.55	0.35
BH14-30	3.13	-	-
BHMW3	1.9 March	3.6 August - 3.99 September	2.09
BHMW11	0.46	0.8 – 1.05	0.5
BHMW22	1.82 – 2.56	-	0.74
BHMW26	2.54	? – 2.34	0.20
BH13-02	2.29	2.15 - 2.97	0.82
BH13-03	-	0.77	-
BH13-05	-	1.72 – 2.56	0.84
BH13-07	-	3.65	-
BH13-10	-	3.4	-
BH13-13	-	4.9 (Aug)	-
BH13-16	-	2.91	-



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LEGEND:

BHMW3	APPROXIMATE BOREHOLE / MONITORING WELL LOCATIONS INSTALLED IN 2004
BH2	APPROXIMATE BOREHOLE LOCATION INSTALLED IN 2006
BHMW3	APPROXIMATE BOREHOLE/MONITORING WELL LOCATION INSTALLED BY DST IN 2006
TP1	APPROXIMATE TESTPIT LOCATION COMPLETED IN 2006
	BOREHOLE LOCATION (DST 2013)
	BOREHOLE/MONITORING WELL (DST 2013)
	TESTPIT LOCATION (DST 2013)
GSL:	GROUND SURFACE ELEVATION (METRES)
GWL:	GROUND WATER ELEVATION (METRES)
80.0	APPROXIMATE GROUNDWATER CONTOUR WITH ELEVATION IN METRES
GWFD	INFERRED GROUNDWATER FLOW DIRECTION

NOTE:
1. THIS DRAWING SHALL BE READ IN CONJUNCTION WITH THE ASSOCIATED TECHNICAL REPORT.

REV	DATE	ISSUE	APPROVAL
1	02/10/13	PRELIMINARY	M.L.

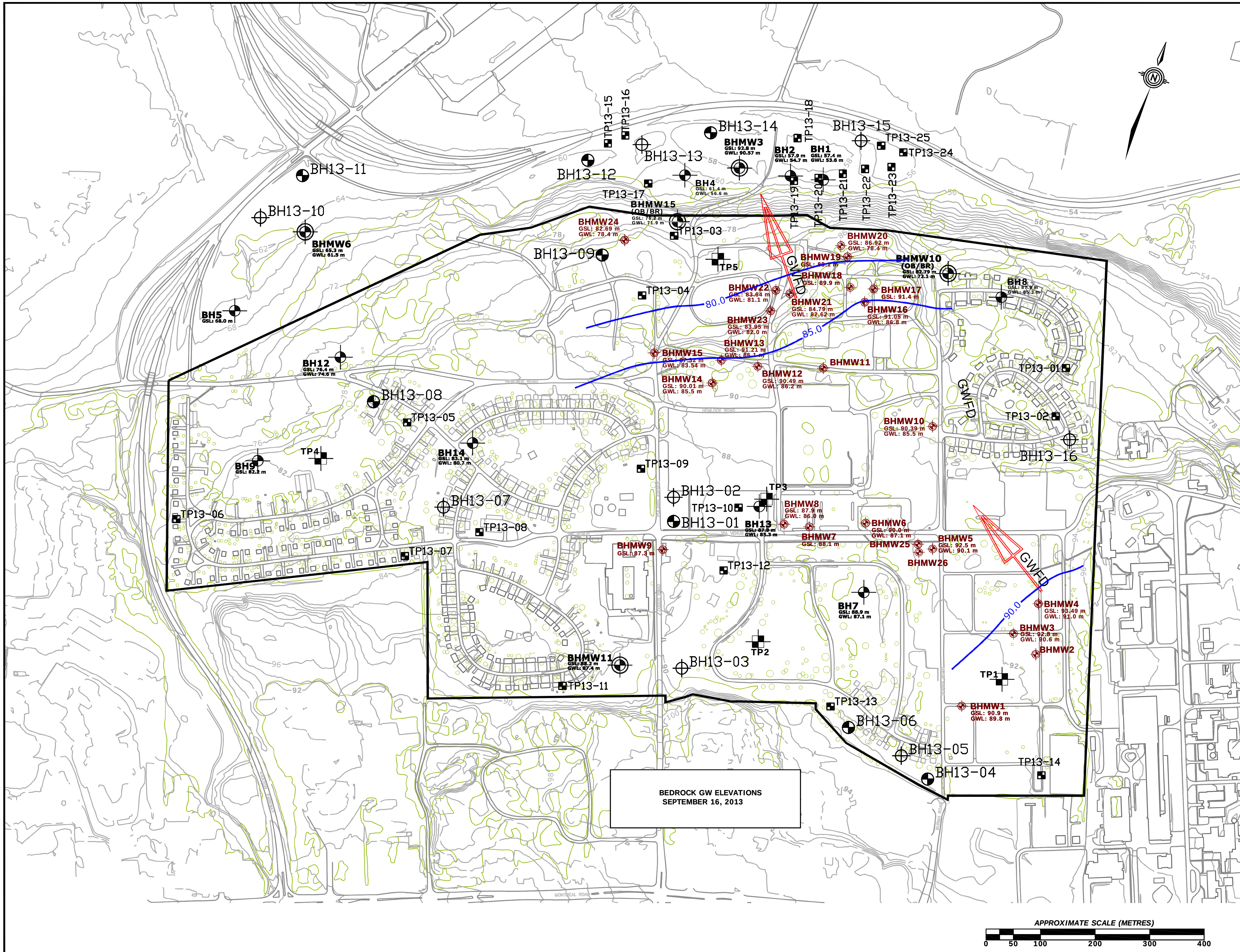
PROJECT TITLE

PRELIMINARY GEOTECHNICAL INVESTIGATION
ROCKCLIFFE REDEVELOPMENT
OTTAWA, ONTARIO

DRAWING TITLE

OVERBURDEN POTENTIOMETRIC CONTOURS

DESIGNED BY: M.L.	SCALE: AS SHOWN
DRAWN BY: R.P.	DATE: October 2013
PROJECT MANAGER: M.L.	PROJECT NO.: OE-OT-017184
FIGURE No.: Figure 9	



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	BOREHOLE LOCATION (DST 2013)
	BOREHOLE/MONITORING WELL (DST 2013)
	TESTPIT LOCATION (DST 2013)
GSL:	GROUND SURFACE ELEVATION (METRES)
GWL:	GROUND WATER ELEVATION (METRES)
80.0	APPROXIMATE GROUNDWATER CONTOUR WITH ELEVATION IN METRES
GWFD	INFERRED GROUNDWATER FLOW DIRECTION

NOTE:
1. THIS DRAWING SHALL BE READ IN CONJUNCTION WITH THE ASSOCIATED TECHNICAL REPORT.

REV	DATE	ISSUE	APPROVAL
1	02/10/13	PRELIMINARY	M.L.

PROJECT TITLE

PRELIMINARY GEOTECHNICAL INVESTIGATION
ROCKCLIFFE REDEVELOPMENT
OTTAWA, ONTARIO

DRAWING TITLE

SHALLOW BEDROCK POTENTIOMETRIC CONTOURS

DESIGNED BY: M.L.	SCALE: AS SHOWN
DRAWN BY: R.P.	DATE: October 2013
PROJECT MANAGER: M.L.	PROJECT NO.: OE-OT-017184

FIGURE No.: Figure 10

Local Groundwater Use

The following was reproduced from the Hydrogeological Report Stormwater Management Support Studies (DST, June 2014). Source materials and reports are included in Appendix A (4).

The Ontario Water Well Information System (WWIS) was reviewed for records of water wells located within and downgradient of the Former CFB Rockcliffe CDP site. The WWIS is a database of the MOE records of water wells in Ontario, and is up-to-date as of the end of 2012.

Up-gradient Water Wells

To the south of the Former CFB Rockcliffe site is the established rural estate community of Fairhaven. Located to the West of Codd's Road and north of Montreal Road, the community of Fairhaven is serviced by private septic and water well systems.

Downgradient Water Wells

Four (4) well records were found for the areas downgradient the Former CFB Rockcliffe CDP site. Three (3) wells drilled in 2007 are located at the Rockcliffe Airport, between the runway and the Ottawa River. One well is recorded as abandoned and the other two wells are noted as test holes, drilled to depths of 3.7 m and 4.9 m respectively through brown sand and silt (0 to 0.6 m) overlying grey clay and silt. No groundwater information is provided in these records.

There is one WWIS record for a well drilled in 2007 North-east of the Aviation Museum. The status of this well is recorded as abandoned.

There are no water wells located downgradient of site in the WWIS database that are recorded as in use. The complete records of the downgradient wells are included within the source document in Appendix A.

It should be noted that, DST made an inquiry to the City of Ottawa regarding municipal water supply to the areas adjacent to the project site. The response indicated that the adjacent areas are serviced by municipal water, however the Aviation Museum located at 11 Aviation Parkway is not on the municipal water supply. It is assumed that the Aviation Museum uses a local groundwater source, and is therefore a downgradient receiver of groundwater impacts from the Former CFB Rockcliffe CDP site.

Hydrogeological Conceptual Model

The following sections describe the characteristics of the hydrogeological units and groundwater flow regimes in the study area per DST (June, 2014) – see Appendix A (4).

Overburden Hydrogeological Unit and Unsaturated Zone

The overburden hydrogeological unit at the Former CFB Rockcliffe CDP site consists mainly of finely textured clay, silt and occasional sand as offshore marine sediments and glacial till. The dominantly clay-rich material has a generally low hydraulic conductivity through the unsaturated zone. Where the clay overburden has higher silt content, or where sand and gravel till are present, hydraulic conductivity is slightly higher.

Shallow Bedrock Hydrogeological Unit

The shallow bedrock hydrogeological unit is defined as the zone of weathered, jointed and faulted limestone bedrock with minor shale partings encountered in the upper 5m of bedrock under the

Former CFB Rockcliffe CDP site. It is interpreted that there is a level of hydraulic connectivity between the overburden and shallow bedrock hydrogeological units. The shallow bedrock unit may be locally connected to the overburden by the jointed structure and fractures, as logged in many of the boreholes.

The groundwater seeps observed along the slope of the escarpment north of the Former CFB Rockcliffe CDP site by DST in 2007 are assumed to have their source from this hydrogeological unit.

Note that bedrock was not encountered during field investigations in the NCC land area North of the Former CFB Rockcliffe CDP site; therefore the downgradient characteristics of the bedrock hydrogeological units are not known.

Deeper Bedrock Hydrogeological Unit

The deeper bedrock hydrogeological unit was only investigated at one monitoring well (BHMW 10 BR, 2006). The deeper bedrock hydrogeological unit is characterized by competent limestone and minor shale bedrock below the shallow bedrock hydrogeological unit. This unit is interpreted to have some level of limited connectivity with groundwater in the shallow bedrock, through less frequent, deeper fractures or jointing in the limestone formation.

Hydraulic Connection between Groundwater and Surface Water

The overburden hydrogeological unit is interpreted to be recharged directly by infiltration of precipitation. Infiltration occurs upgradient and within the Former CFB Rockcliffe CDP site, although certain overburden materials (i.e. silty clay) have reduced capacity to allow infiltration into the ground.

Downslope from the Former CFB Rockcliffe CDP site, groundwater contributes to surface water stream flow by emerging as localized seeps and forming the base flow.

Base flow measurements and characteristics are discussed in Section 3.3 and presented in Table 9.

Hydraulic Conductivity

Falling head slug tests were carried out by DST at monitoring wells during field investigations. Hydraulic conductivity estimates were calculated using the slug test data. Assessment and calculation methodologies are included in the respective reports (DST, 2006; DST, 2014) included as Appendix A.

Data on hydraulic conductivity (from rising or falling head slug tests) are sparse and are summarized in Table 5 below. Values derived from other methods (e.g. the Ontario Building Code or application of Hazen's formula) are not repeated here. The locations of the boreholes and monitoring wells tested are shown on Figure 4. From these data, It is apparent that:

- Sand and sandy gravel has K values between 10-6 and 10-7 (m/sec)
- Silty clay and clay till have K values between 10-6 and 10-8 (m/sec)
- Limestone bedrock has K values between 10-8 and 10-9 (m/sec)

Infiltration testing in test pits TP13-01 to TP13-14 was performed by DST using a Turf-Tec Infiltrometer. The results reproduced as Table 6 show extreme variability and raise concerns regarding their applicability at the design stage. As such additional testing was completed by Aquafor Beech in 2014 and is detailed in the subsequent section.

Table 5: Rockcliffe Hydraulic Conductivity Measurements (DST Slug Tests Only)

Monitor Well	Screened Interval	Layer Description	Hydraulic Conductivity K (m/sec)
BHMW3	3.1 – 6.1	Sandy gravel	4.5 x 10^{-6}
BHMW6	3.1 - 6.1	Straddles clay till and bedrock	2.1 X 10^{-7}
BHMW10	1.5 – 4.4	Sand & silt	3.0 x 10^{-6}
"	13.0 – 16.0	Bedrock Limestone	6.2 x 10^{-7}
BHMW11	3.1 – 6.1	Clay	2.0 x 10^{-7}
BHMW13	3.2 – 6.3	Bedrock Limestone	7.2 x 10^{-9}
BHMW15	12.8 – 15.8	Bedrock Limestone	4.9 x 10^{-8}
BH13-02	1.20 – 2.70	Clay over sand	1.0 x 10^{-8}
BH13-03	3.1 – 4.6	Clay	3.2 x 10^{-8}
BH13-05	1.6 – 3.0	Clay	7.3 x 10^{-6}
BH13-07	3.8 – 5.3	Clay	3.7 x 10^{-8}
BH13-10	10.1 – 11.6	Clay	3.8 x 10^{-8}
BH13-13	9.1 – 10.6	Clay	5.4 X 10^{-8}
BH13-17	8.5 – 10.0	Clay (NOTE: no log available)	7.3 x 10^{-6}

Table 6: Descriptions and Variability Regarding Infiltration Tests

TEST PIT @ DEPTH	DESCRIBED IN LOG	NEAREST BOREHOLE	DESCRIBED	INIFILTRATION RATE BASED ON SOIL DESCRIPTION ONLY (See Table 6)	MEASURED in cm/min (k = m/s)	EQUIVALENT TEXTURE CLASS	VARIABILITY CONCERN
13-01 @ 1.7m	Silty clay (firm to stiff)	13-16	Trace sand + gravel	high	0.06 (10-5)	Loam sand	Water in BH at 2.9 at completion
13-02 @ 1.8m	Silty clay (sewer pipe at 1.5m)	13-16	Trace sand + gravel	moderate	0.002 (3 x 10-7)	Silt clay	Sewer pipe compacted backfill
13-03 @ 1.5m	Silty clay (Refusal at 2.6m)	BHMW-15	Silty clay	moderate	0.037 6 x 10-6)	Clay loam	Fill & concrete in BHMW-15
13-04 @ 1.5m	Silt with sand + gravel	-	-	high	0.023 (4 x 10-6)	Sand loam	Roots & macropores
`113-05 @ 1.6m	Fill – sand + gravel	13-08	Fill?	high	0.053 (9 x 10-6)	Loam sand	Fill, refusal at 1.8m
13-07 @ 1.7m	Silty clay	13-07	Silty clay, roots	low	0.01 (1.6 x 10-6)	Loam	-
13-08 @ 1.7m	Silty clay (stiff)	13-07	Silty clay	low	0.003 (5 x 10-7)	Clay loam	-
13-09 @ 1.8m	Fill, silty sand, some gravel	-	-	high	0.033 (5.5 x 10-6)	Sand loam	Fill
13-10 @ 1.7m	Silty clay	13-01	Silty clay	low	0.03 (5 x 10-6)	Sand loam	-
13-11 @ 1.5m	Silty clay roots	BHMW-11	Silty clay WL=0.8m	moderate	0.065 (1.1 x 10-5)	Loam sand	Silty sand in BHMW-11
13-12 @ 1.5m	Silty clay	13-01	Silty clay	low	0.003 (5 x 10-7)	Clay loam	-
13-13 @ 1.5m	Silty clay roots	13-06	Silty clay roots	moderate	0.01 (1.7 x 10-6)	Loam	-
13-14 @ 1.3m	Silty clay roots?	13-04	Silty clay roots	moderate	0.03 (5 x 10-6)	Sand loam	Macropores

Hydraulic Conductivity Testing

In September, 2014, Aquafor Beech staff completed five (5) days of in-situ infiltration testing to determine the field saturated hydraulic conductivity (Kfs) of the in-situ soils at Former CFB Rockcliffe site. In-situ infiltration testing is used to confirm and to refine the coarse estimates performed as part of geotechnical assessment.

In–situ infiltration testing was completed using a combination of two (2) techniques:

1. Guelph Permeameter Testing apparatus and protocols were utilized to determine the in-situ field saturated hydraulic conductivity (Kfs) at depths ranging from 5 to 30cm (average depth of 15cm) below ground surface (BGS).

Guelph Permeameter at Former CFB Rockcliffe

2. Double Ring Infiltrometer was utilized to determine the in-situ field saturated hydraulic conductivity (Kfs) of shallow soils at the existing land surface. This was intended to provide an indication of Kfs at the likely interface of the proposed grade raise (as discussed previously), to account for the effects of micro-tubules, rootlets and other macropores.

Double Ring Infiltrometer at Former CFB Rockcliffe

In–situ infiltration testing was completed to more accurately define the infiltration capacity of the native (in-situ) soils, for the primary purpose of using these results as model inputs for the development of an accurate existing conditions water balance as detailed in subsequent sections.

The results of the In–situ infiltration testing were also utilized as part of:

- Infiltration based SWM techniques (i.e. LID) feasibility analysis
- Future forecasting of post-development SWM performance

In the future, these results may also be of value for preliminary design of infiltration based SWM techniques (i.e. LIDs), however it should be noted, in–situ testing is required to determine the infiltration capacity of the individual site specific soils as part of detailed design in order to determine the appropriate sizes of each facility, to determine if underdrains are required and to locate any drains within the facility cross-section. As such, this information contained within this section, should not be used for detailed design purposes.

Field Testing Program

The in-situ infiltration testing program was completed within four (4) areas of the Former CFB Rockcliffe site, referred to as:

1. Central,
2. Northeast,
3. Northwest and
4. Southwest.

Locations with undisturbed soils were selected corresponding to the two dominant soil types of silty clay and sand/ silty-gravel (see Figure 4A).

Silty Clay at Former CFB Rockcliffe

Sand/ Silty Gravel at Former CFB Rockcliffe

Selected locations also correspond to the contributing sub-watershed areas of the Western and Eastern Creek. Figure 10A illustrates the testing locations. In total twenty (20) permeameter tests were completed and four (4) double ring infiltrometers. Results are summarized in Table 7.

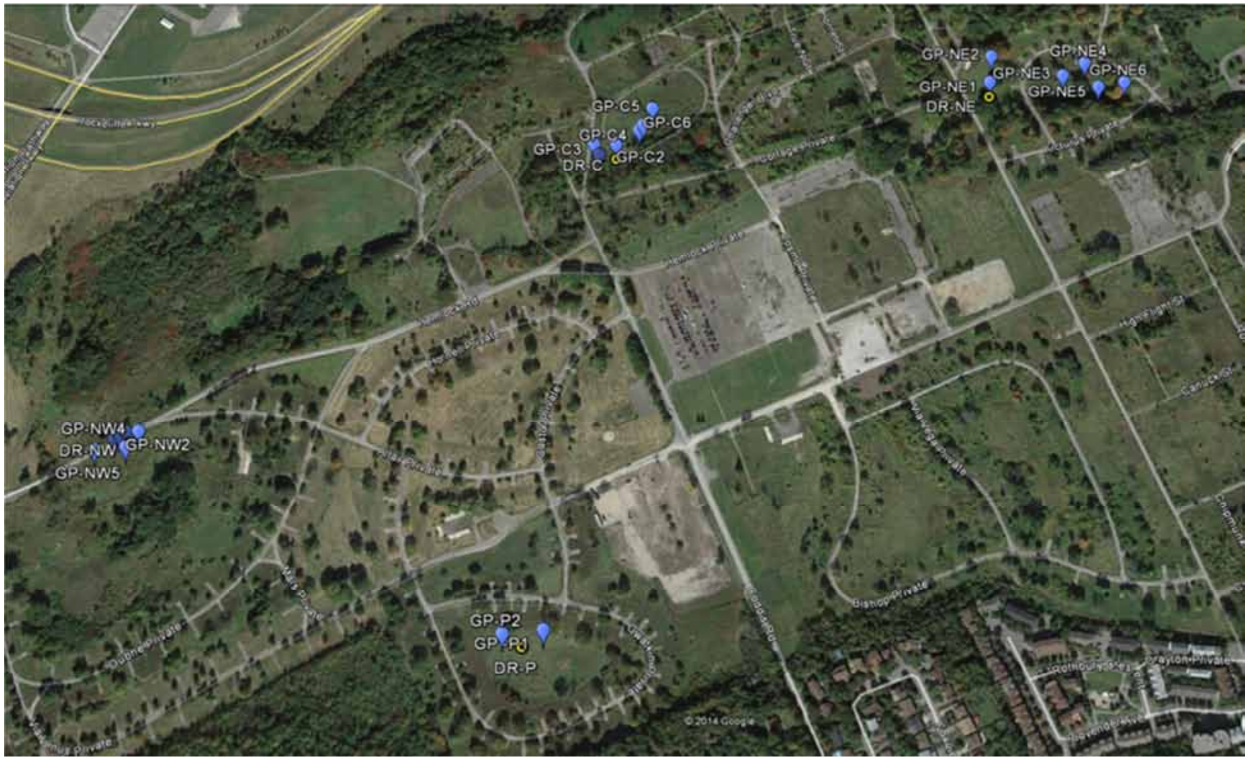


Figure 10A – In-situ Infiltration Testing Conducted by Aquafor Beech (September 2014)

Results

The results of the in-situ infiltration testing are summarized in Table 7. In general the result of the infiltration in-situ infiltration testing revealed the following:

- Field saturated hydraulic conductivity varies across the site per the various soil types
- Localized areas of high infiltration encountered (more than 490mm/hr) and generally correspond to localized areas of sandy soil
- Localized areas of low infiltration encountered (less than 1 mm/hr) and generally correspond to silty clay soils.
- Silty clay and clay till generally found to be lower in original estimates (Table 5) as compared to in-situ testing (Table 7).
- Table 8 summarizes the average hydraulic conductivity for the East and West Creek Subwatersheds.

Table 8: Average Hydraulic Conductivity from In-situ Testing

Subwatershed	Hdr. Conductivity (mm/hr)
Western Creek SWS	9.5
Eastern Creek SWS	5.2

Table 7 – In-situ Infiltration Testing Results (Aquafor Beech, September 2014)

Location	Guelph Permeameter			Double Ring Infiltrometer		
	Site ID	Kfs (cm/s)	Kfs (mm/hr)	Site ID	Kfs (cm/s)	Kfs (mm/hr)
Central	GP-C1	2.22×10^{-3}	80	DR-C	2.22×10^{-3}	720
	GP-C2	2.01×10^{-4}	7			
	GP-C3	2.28×10^{-4}	8			
	GP-C4	4.95×10^{-3}	178			
	GP-C5	7.00×10^{-4}	25			
	GP-C6	7.60×10^{-4}	27			
		Median	26		Median	n/a
		Average	30		Average	n/a
Northeast	GP-NE1	6.25×10^{-5}	2	DR-NE	6.25×10^{-3}	48
	GP-NE2	5.60×10^{-6}	0.2			
	GP-NE3	1.84×10^{-5}	1			
	GP-NE4	1.37×10^{-2}	493			
	GP-NE5	1.06×10^{-4}	4			
	GP-NE6	6.41×10^{-5}	2			
		Median	2		Median	n/a
		Average	1.8		Average	n/a
Northwest	GP-NW1	1.15×10^{-3}	41	DR-NW	6.66×10^{-4}	24
	GP-NW2	4.12×10^{-5}	1			
	GP-NW3	8.35×10^{-3}	301			
	GP-NW4	3.54×10^{-5}	1			
	GP-NW5	5.53×10^{-4}	20			
	GP-NW6	2.76×10^{-3}	99			
		Median	31		Median	n/a
		Average	21		Average	n/a
Southwest*	GP-P1	1.46×10^{-4}	5	DR-P	6.66×10^{-4}	24
	GP-P2	2.10×10^{-3}	76			
		Median	40		Median	n/a
		Average	40		Average	n/a

*Proposed LID Demonstration (Pilot) Area

RED = indicates outlier. Included win Median but not included in reported Average.

Hydrological Impacts

The Hydrogeological Report Stormwater Management Support Studies (DST, June 2014) outlined the following qualitative impacts. Each conclusion has been augmented by Aquafor Beech with ‘additional considerations’ based on the review of all completed studies as whole. It should be noted that at the time of the DST field studies in 2013, six (6) infiltration facilities were proposed, compared to the three (3) facilities currently proposed for the site per the preferred development concept.

Impacts to Groundwater Quantity

In general, the increase in impermeable surfaces area as part of development will decrease groundwater quantity as a result of decreased infiltration of precipitation. The change in volume of groundwater recharge by infiltration will be affected differently across the property by the development due to different land-uses and types of overburden materials. The following conclusions have been identified:

1. In the central and southern portion of the Former CFB Rockcliffe CDP site, the silty clay overburden presently has a low hydraulic conductivity; therefore the development in this area will not significantly decrease the quantity of groundwater recharge. The placement of large infiltration facilities in the silty clay material are not expected to significantly increase groundwater recharge due to the low infiltration rates. The low hydraulic conductivity of the overburden unit will also impede the horizontal movement of groundwater away from any large infiltration facilities, locally raising the water table in the area of the infiltration facilities and reducing the capacity for further infiltration. Note: large centralized infiltration facilities are no longer proposed within the central and southern portions of the Former CFB Rockcliffe development.

2. Development on the till material underlying the northern portion of the Former CFB Rockcliffe CDP site, and on the alluvial sediments underlying the western portion of the site, is in general expected to decrease infiltration and groundwater recharge. However, large infiltration facilities constructed in these materials are expected to effectively provide point sources for groundwater recharge. In this case, when runoff from the areas covered by clay material is diverted into large infiltration facilities situated in areas with surficial deposits with higher infiltration rates, the actual groundwater recharge conditions for the area may improve compared to predevelopment conditions.

Additional Groundwater Quantity Considerations:

1. It is anticipated that development will result in a development wide post-development infiltration deficit and a change in the overall hydrologic regime of the Eastern and Western Creeks. It is proposed that the use of distributed infiltration controls that maintain and or enhance groundwater recharge are applicable for all overburden material types. As such, a water balance is recommended to:

    a) Assess the hydrologic regime using long-term modelling for the pre and post development conditions

b) Asses the infiltration deficit in the post-development condition for district land-uses and catchment areas

The model results can then be utilized to develop LID SWM targets with an emphasis on infiltration targets to off-set the anticipated post-development infiltration deficits. Additional information is presented in Section 5.0.

2. Per 1 above, large centralized infiltration ponds are no longer proposed within the central and southern portions of the Rockcliffe development. The existing conditions infiltration function may be replicated using decentralized infiltration based controls instead.

Impacts to Groundwater Quality

1. The development of the Former CFB Rockcliffe CDP site as a mixed use (primarily residential) community is expected to have negligible impacts on groundwater quality. Anthropogenic activities may introduce contaminants to water that recharges the groundwater under and downgradient of the property. Potential contaminants include, amongst others, polyaromatic hydrocarbons and salts. Certain areas of the property are also known to have contamination from its previous historic use as a military base (DST, 2006 & 2014). An undertaking with regards to soils and groundwater remediation is underway and is expected to be completed prior to redevelopment in the environmentally impacted areas.

Additional considerations relating to the assessment of potential groundwater impacts is discussed in Section 5.

Impacts to Surface Water

Surface water flow measurements, completed for both the Eastern and Western creeks after a prolonged dry period, represent baseflow (groundwater discharge) conditions.

Baseflow for the Western Creek ranged from 0.4 to 1.0 L/s at the Former CFB Rockcliffe CDP site limit and increase to 5.3 to 6.3L/s East of Martin Ave. Baseflow for the Eastern Creek ranged from 0.1 to 1.0L/s immediately downstream of the Rockcliffe Parkway (see Figure 11, Table 9). From this it can be conclude that:

1. It is anticipated that development of the Rockcliffe lands will have an impact on groundwater recharge conditions and may impact baseflow conditions in both creeks.

Additional Surface Water Considerations: From DST (November, 2013), it was concluded that through the use of large infiltration ponds to enhance infiltration into groundwater the potential impact on surface water could be assumed to be minor. However at the time of the report, six (6) large infiltration facilities had been proposed, compared to the three (3) smaller facilities currently proposed (Figure 2). The three (3) proposed Dry Pond facilities are discussed further in subsequent sections.

It is important to consider the spatial distribution of infiltration controls and their hydrologic connection to each creek system when considering impact mitigation strategies. As discussed previously, it is proposed that through the use of distributed or decentralized infiltration controls that the maintenance and or enhancement of groundwater recharge can be achieved.

Impacts to surface water as it relates to aquatic (fish) habitat are discussed in subsequent sections.

Summary of Hydrogeology Findings

In general, hydraulic conductivity rates for the overburden materials across the Former CFB Rockcliffe CDP site are generally very low, particularly in the silty clay overburden material. More specifically, in the central and southern portion of the site, the silty clay overburden presently has low hydraulic conductivity. In the northern and eastern portions of the site, which are underlain by till material, hydraulic conductivity are higher. This also holds true for the western site section, which is underlain by alluvial sediments.

From the aforementioned, it is concluded that:

- Shallow bedrock is a limiting factor for infiltration-based approaches;

- Areas underlain by silty clay have unpredictable infiltration potential and require site-specific tests, as the Kfs values vary over two orders of magnitude, whether measured by slug tests in monitor wells or narrow-diameter double-ring infiltrometers in test pits; and

- Understanding of the seasonal fluctuations in groundwater levels is likely required.

3.3 SURFACE WATER FEATURES

There are three watercourses flowing within and around the Former CFB Rockcliffe site (Figure 11):

1. Western Creek
2. Eastern Creek
3. Northeastern Escarpment Tributary to the Eastern Creek

The Eastern and Western creeks have tributaries that originate within the Former CFB Rockcliffe CDP site, and both also merge with tributaries flowing from adjacent properties. The Northeastern Escarpment Tributary originates from an existing storm sewer outlet.

These creeks flow into the Ottawa River and have sections which flow through underground stormwater piping and culverts (Figure 12). Photo logs and source reports are provided as Appendix A and B.

Western Creek

The Western Creek has two major tributaries (reaches), which confluence to the east of Crispin Private and Blasdell Ave.

The headwater of the eastern most tributary of the Western Creek begins within the Former CFB Rockcliffe site, north of Dubhe private within a localized depression and is fed predominately by the stormwater sewer system with minor contributions from groundwater discharge as baseflow. The creek flows northwest towards Hemlock Drive, where it confluences with existing surface ditches and passes beneath the roadway. The creek continues north, alternating between sections of underground piping and surface (creek) systems several times along the way before flowing briefly subsurface beneath the Aviation Parkway (0.9mØ CSP), where it remerging to later confluence with the western most tributary of the Western Creek. The eastern most tributary is predominately surrounded by open grassland habitat.

The headwater of the western most tributary of the Western Creek begins as a vegetated swale upstream of Hemlock Ave. At the Hemlock Ave crossing it accepts discharge from an existing 1200mmØ storm sewer. The creek continues flowing northeast, parallel to Crispin Private, where it confluences with the eastern most tributary. Several outfall locations from the Rockcliffe Manor Park neighborhood storm sewer system are located in the vicinity of the confluence.

From the aforementioned confluence, the creek flows northwest for approximately 430m as an open channel, through a thin section of riparian habitat until it again flows subsurface within a 1050mmØ culvert for approximately 550m beneath the RCMP property and across the Rockcliffe Parkway. The culvert outlets to a surface channel approximately 20m upstream of the Ottawa River.

It was noted that during baseflow conditions both tributaries maintain some water flow at a shallow water depth throughout their length, with the exception of the final 20m section of surface drainage immediately after the 1050mmØ culvert (note: no water was observed exiting the 1050mmØ culvert). As such, there is no connection to the Ottawa River during baseflow conditions. Recent field studies have identified several culvert blockages that may have affecting these previous observations. This is discussed in the following sections.

Eastern Creek

The headwater of the Eastern Creek begins primarily as a series of roadside ditches along the southern extents of the Rockcliffe Parkway. The Eastern Creek is fed primarily by runoff from the Rockcliffe Parkway, surface drainage from the National Capital Commission (NCC) parcel that sits just south of the Rockcliffe Parkway, as well as from small waterfalls originating within the Former CFB Rockcliffe site and flowing over the cliffs that separate the site from the NCC lands to the north. This portion of the creek flows through open grasslands and mixed forests.

Water collected on the south side of Aviation Parkway is conveyed under the parkway (1.24m Ø CSP) and from there continues south through secondary deciduous forests as a surface channel. Downstream of the Rockcliffe Parkway culvert, the Eastern Creek flows as a straightened channel for approximately 150m before it confluences with a series of three small tributaries flowing form the west which contribute a relatively small proportion of the overall system flow. The Creek meanders towards to Ottawa River however the channel outlet is blocked by a raised recreational pathway. As such, the Eastern Creek has no direct connection to the Ottawa River.

During peak flows, it was observed that water at the outlet point was ponding and did not overtop the recreational path. It appears that water in this section is held back by the recreational path and slowly permeates through the ground.

It was noted that during baseflow conditions, the main body of the creek maintained some water flow at a shallow depth throughout its length, while all three tributaries were dry. Groundwater gradients (Figure 9 and 10) suggest that localized depressions bordering the Rockcliffe Parkway may be areas of groundwater discharge.

Northeastern Escarpment Tributary

The Northeastern Escarpment Tributary originates at the top of the escarpment from an existing storm sewer outlet via a 1050mm Ø CSP in the north east corner of the Former CFB Rockcliffe CDP site. The existing outlet services the eastern one-third of the Former CFB Rockcliffe CDP site including providing one of two outlets for the Burma Road Stormwater Management facility in Thorncliffe Park. Due to topographic constraints, the Northeastern Escarpment Tributary provides local drainage and a primary outlet for the lands bounded by Acturus Private and Burma Road (north-east corner).

From the outlet at the top of the escarpment the channel continues north, intermittently down the escarpment, towards the Rockcliffe Parkway where it merges with the Eastern Creek at the west of the Rockcliffe Parkway crossing.

The channel does not appear to maintain flow during baseflow conditions and is demonstrating signs of local erosion.

Surface Water Flow Measurements

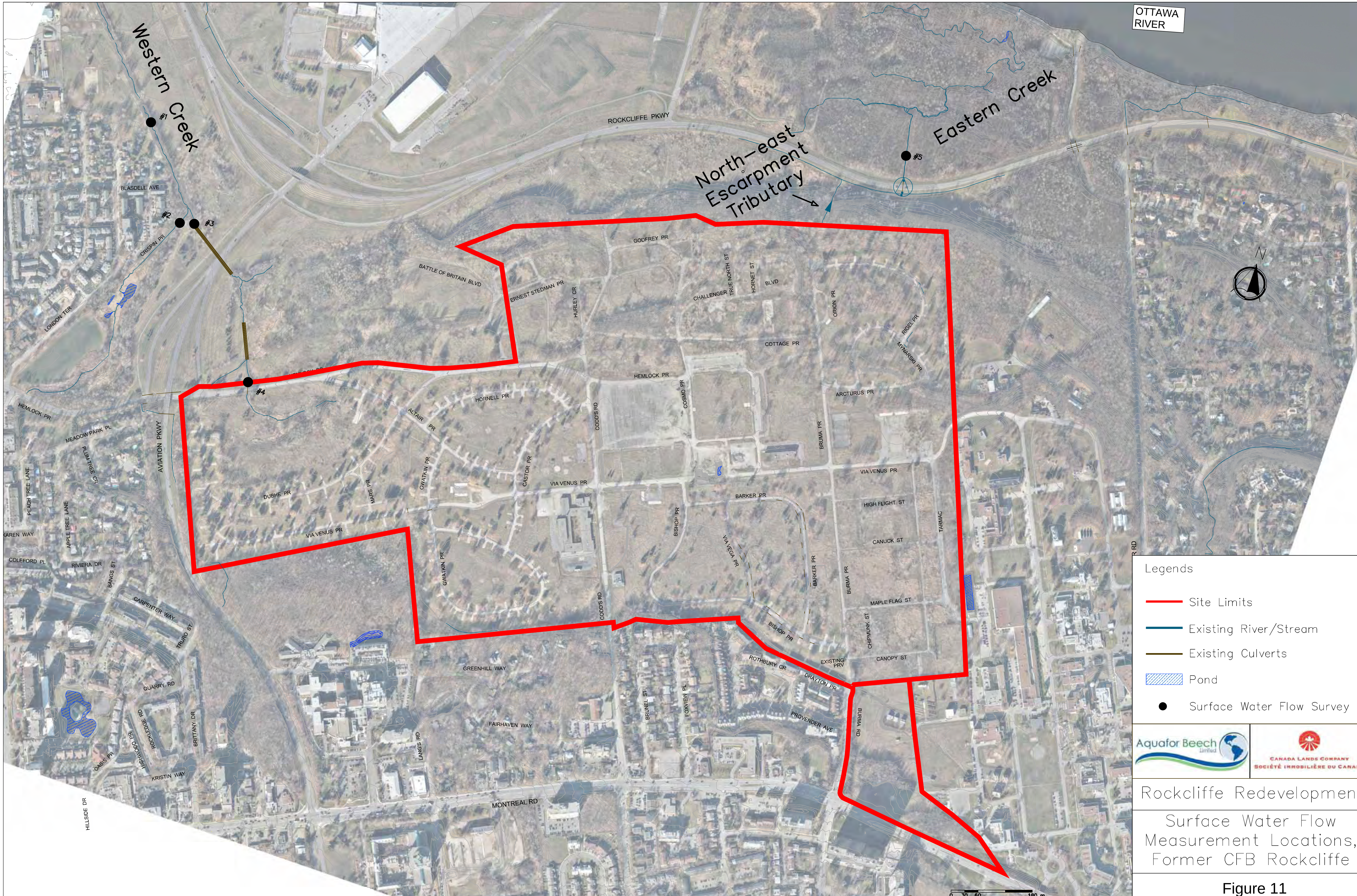
As part of the hydrogeological studies (June, 2014) in support of stormwater management approaches for the Rockcliffe lands, DST completed a series of surface water flow measurements for the Eastern and Western Creeks (Figure 11). Three (3) rounds of flow measurements were completed on July 18, July 25 and July 31, 2013. In selecting locations for stream flow measurements, the following guidelines were followed:

- The channel should have as much straight run as possible. Where the length of straight run is limited, the length upstream from the profile should be twice the downstream length;
- The channel should be free of flow disturbances. Clear any rocks, sediment, or other debris that might be on the bottom of the creek; and
- Areas immediately downstream from sharp creek bends or obstructions should be avoided.

Using standard area velocity and stream gauging techniques the flows presented in Table 9 were recorded. The measurements were taken after a prolonged dry period representing baseflow (groundwater discharge) conditions.

Table 9: Baseflow Measurements

Site ID	Flow (L/s)		
	07/18 2013	07/ 25 2013	07/ 31 2013
1	6.3	5.4	5.3
2	4.4	2.8	4.8
3	1.8	2	2.8
4	0.3	1.3	1.6
5	1.0	0.8	0.4
Rainfall (mm)	1.4	0.0	0.0



Rockcliffe Redevelopment

Surface Water Flow Measurement Locations, Former CFB Rockcliffe

Figure 11

3.4 EXISTING DRAINAGE

The natural surface pattern for the Former CFB Rockcliffe site is shown on Figure 12. Most of the site is relatively flat, generally sloping from the south boundary downward to the north property line. The elevation across the study area ranges from about 90 to 70 m. Several small terraces bisect the plateau areas and provide additional grade downward from the southeast to the northwest; however the site is characterized by two distinct escarpments. One runs along the south property line and the second is adjacent to the north property line. To the north, the base of the escarpment descends to approximately 55 m at the Rockcliffe Parkway, and about 45 m at the Ottawa River.

Historical drainage patterns include a drainage swale which bisected the former CFB Rockcliffe site, originating in the southeast corner on NRC lands and discharging at the northwest limit crossing the Aviation Parkway via a culvert and outletting to the Ottawa River along the north limit of the RCMP Lands. Most of the plateau lands of the site naturally slope in this direction. A second drainage swale within the former CFB Rockcliffe site drains the northeast quadrant of the site northward over the north escarpment and crosses the Aviation Parkway via a culvert ultimately discharging to the Ottawa River. These two original overland drainage routes remain the primary outlets for major storm events.

Due to the location of the southern escarpment, there are natural external drainage areas located to both the south and east of the Rockcliffe property. Surface drainage from these areas will have to be accommodated as part of the stormwater management plan during development.

Surface drainage from the NRC Campus, which is located to the east of the Rockcliffe site routes onto the subject site. The southern portion of the NRC property discharges surface flow towards the existing Burma Road SWM Facility. However, the central portion of the NRC property directs surface runoff towards the subject site near Tarmac Street.

The natural topography through most of Thorncliffe Village is towards the north into the existing SWM facility. There is also surface drainage tributary to the subject site from existing developments located on top of the southern escarpment including Foxview, Fairhaven and the Montfort Hospital. The Foxview residential development, located at Codd's Road, includes a minor storm drainage system but major storm runoff flows over the escarpment onto the subject site. The Fairhaven community, which is a rural estate development located immediately east of the Montfort Hospital, does not have a piped storm sewer collection system but rather uses road side ditches to convey surface runoff. Surface drainage from the northern portion of that development is directed toward the subject site.

The northeastern portion of the Montfort Hospital site is provided with a SWM Facility that outlets to an existing swale through the adjacent Montfort Hospital Woods. Outflow from the SWM facility, as well as major system flow from the northeastern portion of the hospital site, and flow from the Montfort Hospital Woods all makes its way to the road side ditch system near Via Venus Private.

Surface runoff discharges at the northwest limit crossing the Aviation Parkway via a culvert (Western Creek) and outletting to the Ottawa River along the north limit of the RCMP Lands. Surface runoff from the northeast quadrant of the site discharges northward over the north escarpment and crosses the Aviation Parkway via a culvert (Eastern Creek) ultimately discharging to the Ottawa River.

Existing Combined Sewer Drainage

The previous CFB development includes a combined sewer network (Figure 12 and 13). As the property has been developed over the 60 years, combined sewers were installed to handle the wastewater and most of the storm runoff generated on the site. Development of the existing sewage collection system followed the natural topography of the site, resulting in two distinct combined sewer systems on the Rockcliffe site. Codd's Road acts as the approximate drainage divide between the two. The combined sewer system to the east of Codd's Road collects all wastewater and most surface runoff from frequent storm events in that portion of the site, and eventually outlets to a trunk sewer to the east of the site. A second combined sewer system collects wastewater flows and some surface runoff west of Codd's Road and discharges to the same major trunk sewer, but to the west of the site.

Existing Storm Sewer Drainage

The following section has been summarized from the Former CFB Rockcliffe MSS (IBI, 2015).

In addition to the existing predominant combined sewer system on the Rockcliffe site, there are also two (2) dedicated storm sewer systems (Figure 13). The two existing storm sewers are both 1050 mm in diameter and convey discharge from the existing Burma Road Stormwater Management (SWM) Facility. These two sewers both flow north from the facility, one east of Via Vega Private and the other west of that road and are discussed below. The balance of the existing site storm drainage system consists of road side ditches that direct runoff to either the combined sewers or the dedicated storm sewers.

The western 1050 Ø mm storm sewer collects surface runoff from the central portion of the site near Via Venus Private and Codd's Road and routes those flows westward and outlets to an open ditch system located behind Dubhe Private. The ditch has a fairly steep gradient and naturalized cobble bottom in its headwaters east of Aviation Parkway.

According to the MSS (IBI, 2015) The channel crosses Aviation Parkway via a 900 mm (36") diameter corrugated steel pipe (CSP) outletting to a trapezoidal channel, referred to as the Western Creek, adjacent to Crispin Private. The downstream end of the culvert is heavily silted to above the obvert, severely restricting flow through the culvert. Due to the limited available data for the culvert, the following information has been compiled from field survey work. The culvert measures approximately 137 m in length. The upstream and downstream inverts were surveyed at elevations 60.25 m, and 60.16 m, respectively, resulting in a slope of 0.07%.

Downstream of the Aviation Parkway culvert, the sediment in the channel continues for approximately 30 m westerly to the point of confluence with another channel from the Alvin Heights neighbourhood to the south. The combined channel flows westerly at a relatively flat slope through several short 900 mm diameter culverts supporting pedestrian crossings and an access road within the RCMP training grounds. The inlet to one crossing within the RCMP site is completely blocked and restricts flow to only that which can filter through the crossing embankment. At the west end of the RCMP site, the channel enters a 900 mm diameter CSP culvert that crosses under the Rockcliffe Parkway and eventually outlets to the Ottawa River.

The channel between the Aviation Parkway and the Rockcliffe Parkway crossing is a relatively straight trapezoidal channel and is heavily treed along the section outside the RCMP site. There is also an existing concrete-encased sewer that partially blocks the invert of the channel immediately west of the pedestrian bridge at the end of Crispin Private.

The northern 1050 mm diameter storm sewer extends northward through the site, intercepting surface runoff from the lands east and north of Via Vega Private, prior to discharging directly over the escarpment at the north limit of the study area, referred to as the Northeastern Tributary. Flow at the base of the escarpment is directed southeasterly in the southern road side ditch of the Rockcliffe Parkway.

The culvert crossing at the Rockcliffe Parkway at the eastern edge of the site, considered the eastern outlet, is a 1200 mm diameter (48”) CSP. Based on the limited available data for the culvert, the following information has been compiled from field survey work. The culvert measures approximately 28 m in length. The upstream and downstream inverts were surveyed at elevations 52.52 m and 52.15 m, respectively, resulting in a slope of 1.3%. The culvert discharges to a channel, referred to as the Eastern Creek, which is relatively straight and destabilized in the vicinity of the Rockcliffe Parkway, graduating into a wider meandering creek with several sub-reaches as it approaches the Ottawa River. The channel is completely blocked by a pedestrian trail crossing located near the edge of the Ottawa River with no culvert, thereby restricting the discharge rate of the creek to the flow that can infiltrate through the pathway fill.

There are several sources of external runoff to the site. As noted above, the Burma Road SWM Facility provides treatment of runoff from the Thorncliffe Village development. In addition, an overland ditch at the south end of the NRC Campus at Montreal Road is intercepted by the Burma Road SWM Facility. The outlets from the facility are described above. Runoff from the central portion of the NRC Campus that is in excess of that intercepted by the NRC combined sewer system drains onto the site along the common east boundary. This flow is intercepted by the combined sewer system and/or the road side ditch and dedicated storm system.

The Fairhaven community uses roadside ditches to convey storm flow to the south boundary of the former CFB Rockcliffe site where the flow is intercepted and eventually directed to the western channel via road side ditches. The Foxview development contributes major system flow to the Rockcliffe site. The northeastern portion of the Montfort

Hospital site is provided with a SWM Facility that outlets to an existing swale through the adjacent Montfort Hospital Woods. Outflow from the SWM facility, as well as major system flow from the northeastern portion of the hospital site is conveyed to the Rockcliffe site and into road side ditches near Via Venus Private. The runoff is eventually conveyed to the western outlet at the Aviation Parkway.

There is an undeveloped area located between the site and the Rockcliffe Parkway. The western portion contributes runoff to the existing west outlet, while the eastern portion contributes runoff to the existing east outlet.

The balance of the existing storm drainage system on the site consisted of road side ditches which directed runoff to either the combined sewer system or the dedicated storm sewer systems.

External Runoff Sources

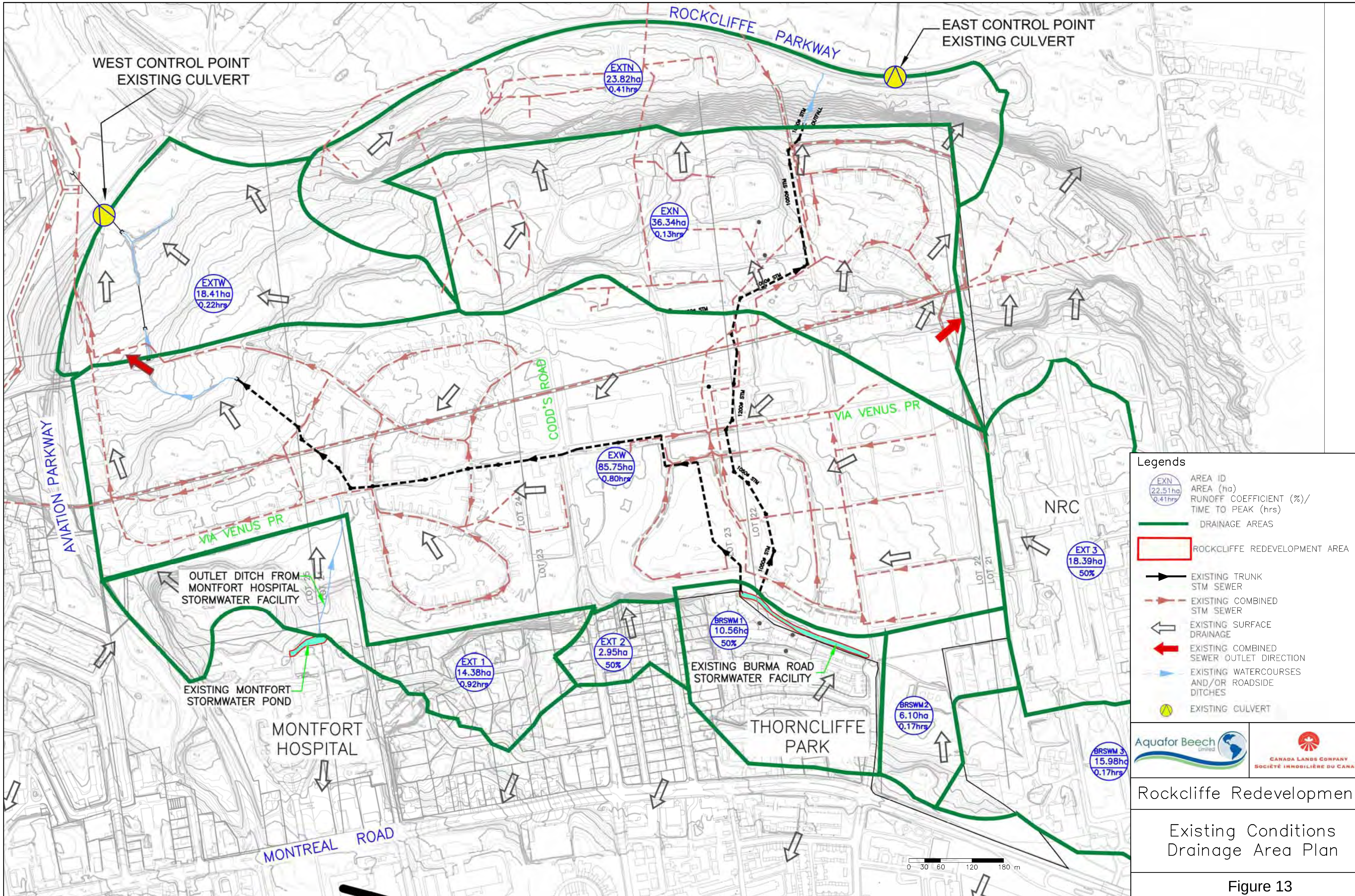
Besides runoff from the Thorncliffe Park which flows to the Burma Road stormwater management facility (BRSWM1, 2 and 3), there are several additional sources of external runoff to the south and east of the Rockcliffe site including (Figure 13):

1. External Area 1 (EXT 1) – includes the existing rural estate lands east of the Montfort Hospital which drain north into the Former CFB Rockcliffe CDP site and the Montfort Hospital which has a new stormwater facility located near the southern escarpment. It outlets at approximately 24 l/s (Stormwater Management Report – Montfort Long Term Care Facility City of Ottawa" (Stantec, 2002) into an existing swale above the southern escarpment which discharges to the site and into the roadside drainage system near Via Venus. That flow is eventually conveyed to the dedicated storm sewer which discharges behind Dubhe Private to the Western Creek.

2. External Area 2 (EXT 2) – includes the existing residential lands at Codd's Road which contribute major system flow to the Former CFB Rockcliffe CDP site.

3. External Area 3 (EXT 3) - contribute surface runoff from the south end of the NRC campus west of Blair Road drains towards the eastern edge of the Former CFB Rockcliffe CDP site.

Sheet No.	Drawing Title	Project Title	Scale
FIGURE 12	NATURAL SURFACE DRAINAGE PATTERNS	FORMER CFB ROCKCLIFFE MASTER SERVICING STUDY	N.T.S.


ROCKCLIFFE PARKWAY
EAST CONTROL POINT
EXISTING CULVERT
WEST CONTROL POINT
EXISTING CULVERT
EXTN
23.82ha
0.41hrs
EXN
36.34ha
0.13hrs
EXTW
18.41ha
0.22hrs
EXW
85.75ha
0.80hrs
CODD'S ROAD
VIA VENUS PR
VIA VENUS PR
AVIATION PARKWAY
NRC
EXT 3
18.39ha
50%
OUTLET DITCH FROM
MONTFORT HOSPITAL
STORMWATER FACILITY
EXISTING MONTFORT
STORMWATER POND
MONTFORT
HOSPITAL
EXT 1
14.38ha
0.92hrs
EXT 2
2.95ha
50%
BRSWM 1
10.56ha
50%
EXISTING BURMA ROAD
STORMWATER FACILITY
THORNCLIFFE
PARK
BRSWM 2
6.10ha
0.17hrs
BRSWM 3
15.98ha
0.17hrs
MONTREAL ROAD
LOT 24
LOT 23
LOT 23
LOT 22
LOT 22
LOT 21
0 30 60 120 180 m
Legends
AREA ID
AREA (ha)
RUNOFF COEFFICIENT (%)/
TIME TO PEAK (hrs)
DRAINAGE AREAS
ROCKCLIFFE REDEVELOPMENT AREA
EXISTING TRUNK
STM SEWER
EXISTING COMBINED
STM SEWER
EXISTING SURFACE
DRAINAGE
EXISTING COMBINED
SEWER OUTLET DIRECTION
EXISTING WATERCOURSES
AND/OR ROADSIDE
DITCHES
EXISTING CULVERT
Aquafor Beech
CANADA LANDS COMPANY
SOCIÉTÉ IMMOBILIÈRE DU CANADA
Rockcliffe Redevelopment
Existing Conditions
Drainage Area Plan
Figure 13

3.5 HYDRAULICS

A hydraulic analysis was conducted by IBI as part of the MSS for the Former CFB Rockcliffe site in order to identify constraints within the surface water systems.

As described previously, the flow discharge from the study area is currently conveyed to two existing culvert crossings at the Aviation and Rockcliffe Parkways, Western and Eastern Creeks respectively.

The first crossing at the Aviation Parkway on the west edge of the Former CFB Rockcliffe CDP site (West Outlet, see Figure 13) consists of a 0.91m diameter (36") CSP. The following information has been compiled from field survey work.

- The upstream and downstream inverts were surveyed at elevations 60.25m, and 60.16m, respectively. The culvert is approximately 137m in length with slope of 0.07%.

The second culvert crossing at the Rockcliffe Parkway at the eastern edge of the Former CFB Rockcliffe CDP site (East Outlet – see Figure 13) is comprised of a 1.24m diameter (48") CSP. The following information has been compiled from field survey work.

- The upstream and downstream inverts were surveyed at elevations 52.52m and 52.15m, respectively. The culvert is approximately 28m in length with slope of 1.3%.

An analysis has been completed based on available data for the culvert crossings. Culvert design nomographs for the existing culverts and normal depth of flow within the existing ditches downstream of the culverts were analyzed to determine maximum capacity of the existing outlet systems. Culvert design nomographs are included within the MSS (IBI, 2015). Based on the analysis, flow release from the study area is currently constrained by the capacity of the existing culvert crossings at the western and eastern limits of the Former CFB Rockcliffe CDP site. The following sections outline the results of the analysis.

Western Outlet Capacity Analysis

The results of the analysis within the MSS (IBI, 2015) indicate that the peak capacity of the existing culvert is approximately 0.85cms, with water surface elevation at 61.75m at the upstream end of the culvert.

Eastern Outlet Capacity Analysis

The results of the analysis indicate that the peak capacity of the existing culvert is approximately 3.5cms, with water surface elevation of 54.80m at the upstream end of the culvert. As discussed in previous sections, no outlet to the Ottawa River currently exists for the Eastern Creek.

Table 10 summarizes the existing culvert capacity for the Easter and Western outlets at the Rockcliffe and Aviation Parkway.

Table 10: Summary of Existing Culvert Capacity

Outlet	Capacity (cms)
Eastern (Rockcliffe Pkwy)	3.50
Western (Aviation Pkwy)	0.85

Conclusion

From the hydraulic analysis, the existing outlets from the system are constrained by capacity limitations at the Eastern outlet and Western outlet at the Rockcliffe and Aviation Parkways respectively. Therefore, assuming a suitable outlet for the Eastern Creek can be constructed at the Ottawa River, from a hydraulics perspective any future stormwater management system must either:

a) Control post-development flows to respect the capacity of the existing infrastructure

b) Provide sufficient capacity to convey post-development flows such that major flows can be safely conveyed to the Ottawa River (where flood control is not required).

3.5 HYDROLOGY

In support of the MSS (IBI, 2015) an existing conditions hydrologic model using a SWMHYMO computer simulation was developed. The purpose of this section is to summarize the findings of the analysis.

Synopsis of Previous Studies

The report "Stormwater Management Study - Burma Road Development City of Ottawa" prepared by JL Richards (February 1991) outlined the existing Burma Road stormwater management facility servicing the Thorncliffe Park development. That report presented the drainage areas tributary to the SWM facility, described the downstream receiving storm sewers within the Rockcliffe lands, and outlined the peak controlled discharge rate to each of the dual receiving outlet sewers.

The report "Stormwater Management Report – Montfort Long Term Care Facility City of Ottawa" prepared by Stantec (September 2002) outlined the existing Montfort SWM facility that currently services a portion of the Montfort Hospital. That report indicated a peak controlled discharge rate of 24 l/s from the Montfort SWM facility.

The findings of the above reports, including drainage areas and peak outflows from the SWM facilities, have been incorporated and updated with the current existing conditions hydrological analysis for the Former CFB Rockcliffe CDP site, and are used as the basis for the current stormwater management analysis presented below.

3.5.1 EXISTING CONDITIONS HYDROLOGY

A hydrologic analysis of the study area was conducted using SWMHYMO computer model. This technique offers a single storm event flow generation and routing. Land use, selected modeling routines, hydrologic input parameters and simulated results are discussed in the following sections.

Model Parameters

Area and imperviousness: Catchment areas are based on review of the topography of the site and external areas. Please refer to Figure 13 for delineation of the drainage boundaries.

Infiltration: A composite CN Number was applied to the developed portion of the Former CFB Rockcliffe CDP site within the study area. Based on review of the available topographic mapping and aerial photography for the subject site, the area was conservatively estimated to contain 25% impervious surfaces. This impervious area was assigned a CN value of 99, and used to calculate a composite CN value for the area. A summary of the calculations are provided within Table 11.

Initial Abstraction: Initial abstraction depths were set to 1.5mm for un-developed areas of the catchment.

Depression Storage: Depression storage depths of 0.8 mm and 1.5 mm were used for impervious and pervious areas, respectively.

Manning's roughness: Manning's roughness coefficients of 0.013 and 0.25 were used for impervious and pervious areas, respectively.

Design Storms: The 2, 5, and 100 year, 12 hour SCS Type II design storm events were used in the evaluation of the site.

A summary of the Existing Conditions Drainage Area Characteristics is presented in Table 12.

Table 11: Existing Conditions Drainage Area Characteristics Calculations

Area ID	Total Area (Ha)	Impervious Area		Pervious Area		Composite CN
		Area (ha)	CN	Area (ha)	CN	
EXW	86.06	21.52	99	64.54	78	83
EXN	36.34	9.09	99	27.25	76	82

Table 12: Existing Conditions Drainage Area Characteristics

ID	Description	Area (ha)	CN	Imp. (%)		Length (m) LGI	Time to Peak (hr)
				Tot.	Dir.		
EXT1	Open Space, Rural Estate	2.95	N/A	61	61	140	N/A
EXT2	Residential Open Space, Rural Estate	6.18	80	N/A	N/A	N/A	0.16
EXT 3	Residential Open Space, Rural Estate	8.12	80	N/A	N/A	N/A	0.66
BRSWM1	Residential	11.08	N/A	71	71	261	N/A
BRSWM2	Residential	7.30	N/A	65	65	260	N/A
BRSWM3	Residential	17.40	N/A	71	71	508	N/A
EXT- NRC	NRC Lands	18.39	N/A	71	71	450	N/A
EXTW	Vacant Lands	17.52	72	N/A	N/A	N/A	0.22
EXTE	Existing Rockcliffe	24.68	71	N/A	N/A	N/A	0.41
EXM	Montfort Hospital	2.02	N/A	31	31	114	N/A

Note: * - Composite CN value

Hydrological Model Results of Existing Conditions

Results of the SWMHYMO computer simulation for the existing conditions are summarized within Table 13.

Table 13: Existing Conditions Model Results

Design Storm	Peak Flow to Western Outlet (cms)	Peak Flow to Eastern Outlet (cms)
2 Year, 24 Hr, SCS Type II	3.91	2.97
5 Year, 24 Hr, SCS Type II	5.76	4.37
100 Year, 24 Hr, SCS Type II	11.56	8.83

3.5.2 EXISTING CONDITIONS WATER BUDGET

A water budget analysis is required to understand the overall hydrology of the study area, and evaluate the relationship between precipitation and the response of the study area to precipitation events over a long timeframe.

Modification of the hydrologic cycle through urban development has impacts on water quantity, water quality, and stream morphology. More specifically, urbanization reduces evaporation, evapotranspiration, and infiltration, thereby, increasing surface runoff and resulting in more frequent and higher peak flows in rivers and streams causing increased flooding and erosion. Reduction of infiltration decreases groundwater recharge, potentially affecting cool baseflow to streams and wetlands.

The overarching objective of this section of the report is to analyze existing conditions hydrology in order to establish baseline conditions, which will be used later to carry out an impact assessment of future development and develop stormwater management targets. There are many benefits that result from a water balance assessment. These benefits include:

- Identifying key elements in the Hydrological Cycle, including surface runoff, evapotranspiration, and infiltration;
- Defining streamflow regime and variability; and
- Providing baseline conditions to help develop stormwater management targets.

In order to carry out the water budget analysis, background information was obtained from several sources including and not limited to the following:

- Field investigations
    - o Field confirmation of watercourse mapping
    - o Evaluation of Infiltration capacity using the Guelph Permeameter
- Technical reports covering the study area, including
    - o DST and University of Ottawa, 2013. Fluvial Geomorphology Assessment
    - o DST, 2013. Preliminary Geotechnical Investigation Study Report (Draft)
    - o DST, 2014. Hydrogeological Report. Stormwater Management Support Studies
    - o IBI, 2014. Existing Conditions Report
- Historical metrological data (Environment Canada and City of Ottawa Weather Stations), including
    - o Hourly precipitation values
    - o Air temperature
    - o Soils data, including
    - o IBI, 2014. Existing Conditions Report
    - o Field Observations

Water Budget Analysis Approach

The hydrologic cycle is a complex process and its natural components are dependent on many factors: soils, topography, vegetation, geology, climate, etc. Any change to these natural factors will result in a change to the hydrologic cycle; these changes accelerate with urbanization. A tool often used in water resources management is Water Budget Analysis, which sums the various components of the hydrologic cycle for a watershed by balancing precipitation input, evapotranspiration output, infiltration, and surface runoff input and output.

Climate Data

Precipitation data from four (4) rain gauges belonging to the City of Ottawa were reviewed. These stations included Lemieux, Hawthorne, Lee's, and Ropec. Ropec rain gauge was eventually used because of long record (1996-2013) and proximity to the study area. Since the Ropec gauge included only rainfall data, snowfall data was augmented by Environment Canada station (45°23'00.000" N, 75°43'00.000" W, 79.2 masl). Average annual precipitation for the record (1996-2013) was 887 mm (Figure 14).

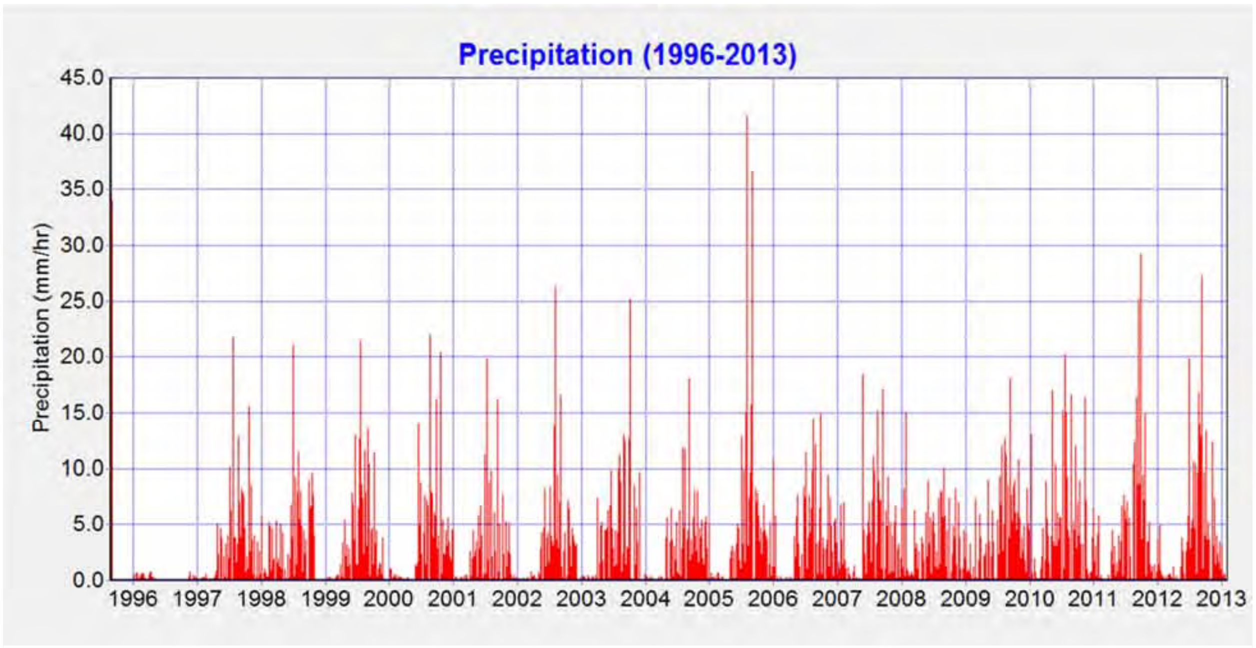


Figure 14: Precipitation Record (mm/hr) from 1996 to 2013

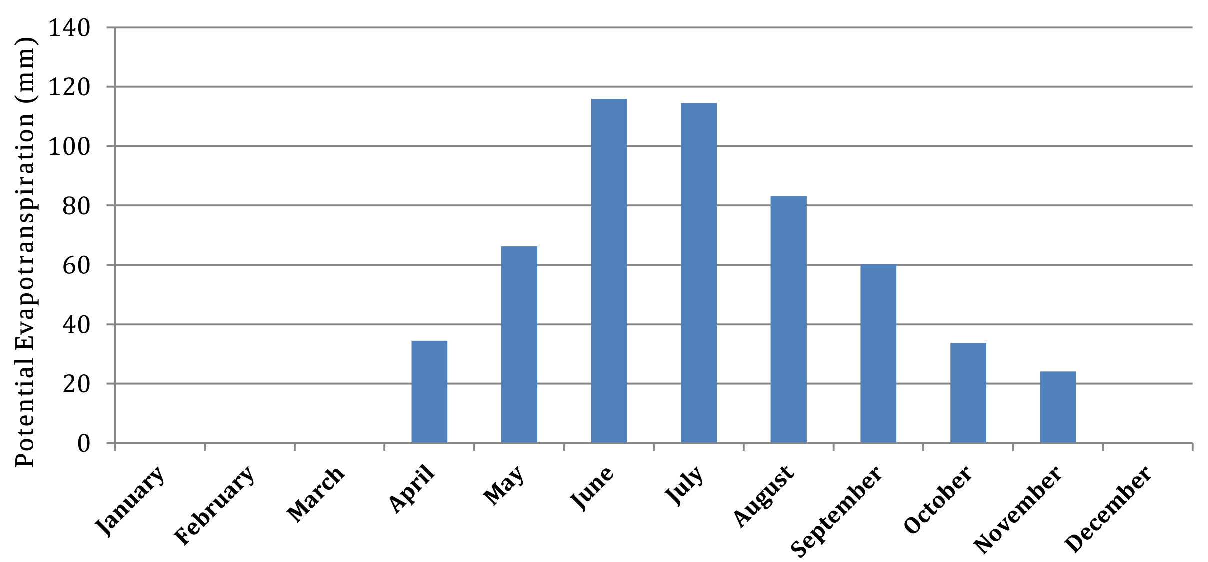


Figure 15: Average Potential Evapotranspiration Values between 1996 and 2013

The climate normals (1996 – 2013) from Environment Canada (45°23'00.000" N, 75°43'00.000" W, 79.2 masl) were used to obtain monthly potential Evapotranspiration (ET) values using the Thornthwaite and Mather (1957) Procedure (Figure 15). Based on the ET calculations, the annual evapotranspiration for the study area is 532 mm, which amounts to approximately 60% of annual precipitation; the remaining 40% (355 mm annually) is the water surplus that may be partitioned between infiltration and runoff.

Hydrologic Model Development

In order to conduct hydrologic analysis for the water budget within the study area, computer models in conjunction with other desktop analyses were developed. The hydrologic analysis of the existing hydrologic conditions in the study area was carried out using the EPA SWMM (Environmental Protection Agency Storm Water Management Model) model, version 5.0.022, a deterministic event-based and continuous hydrologic model that can be used to simulate surface runoff quantity and quality, water budget, and various hydrologic and hydraulic variables.

EPA SWMM version 5.0.022 is suitable for modeling pre- and post-development conditions because it includes several modules that can simulate physical conditions in both cases. For post-development scenarios, EPA SWMM models the hydrologic performance of Low Impact Development (LID) controls, such as porous pavement, bioretention areas, rain barrels, infiltration trenches, and vegetative swales. The model allows engineers and planners to determine the effectiveness of LID measures in managing stormwater and combined sewer overflows under future development conditions.

Subcatchment Characteristics

The EPA SWMM model has used drainage areas and boundaries that had been delineated earlier. Figure 16 shows a schematic of Western Creek and Eastern Creek subcatchments.

Imperviousness values and depression storage assumptions were added to the model using the values shown in Table 14. Aerial photos and previous land cover assessments were used to assign these values. Higher imperviousness values were assigned to areas to the south and to the east of the study area. In addition, depression storage was assumed to be higher for areas with mature trees and vegetation cover (e.g. EXTW).

Infiltration Model

The infiltration model was based on the EPA SWMM Green-Ampt model. According to geotechnical studies and previous field analyses conducted by Aquafor Beech in 2014, the study area consists mostly of loam and silt loam, with an average saturated hydraulic conductivity of 9.5 mm/hr for Western Creek subwatershed, and 5.2 mm/hr for Eastern Creek subwatershed (Table 15).

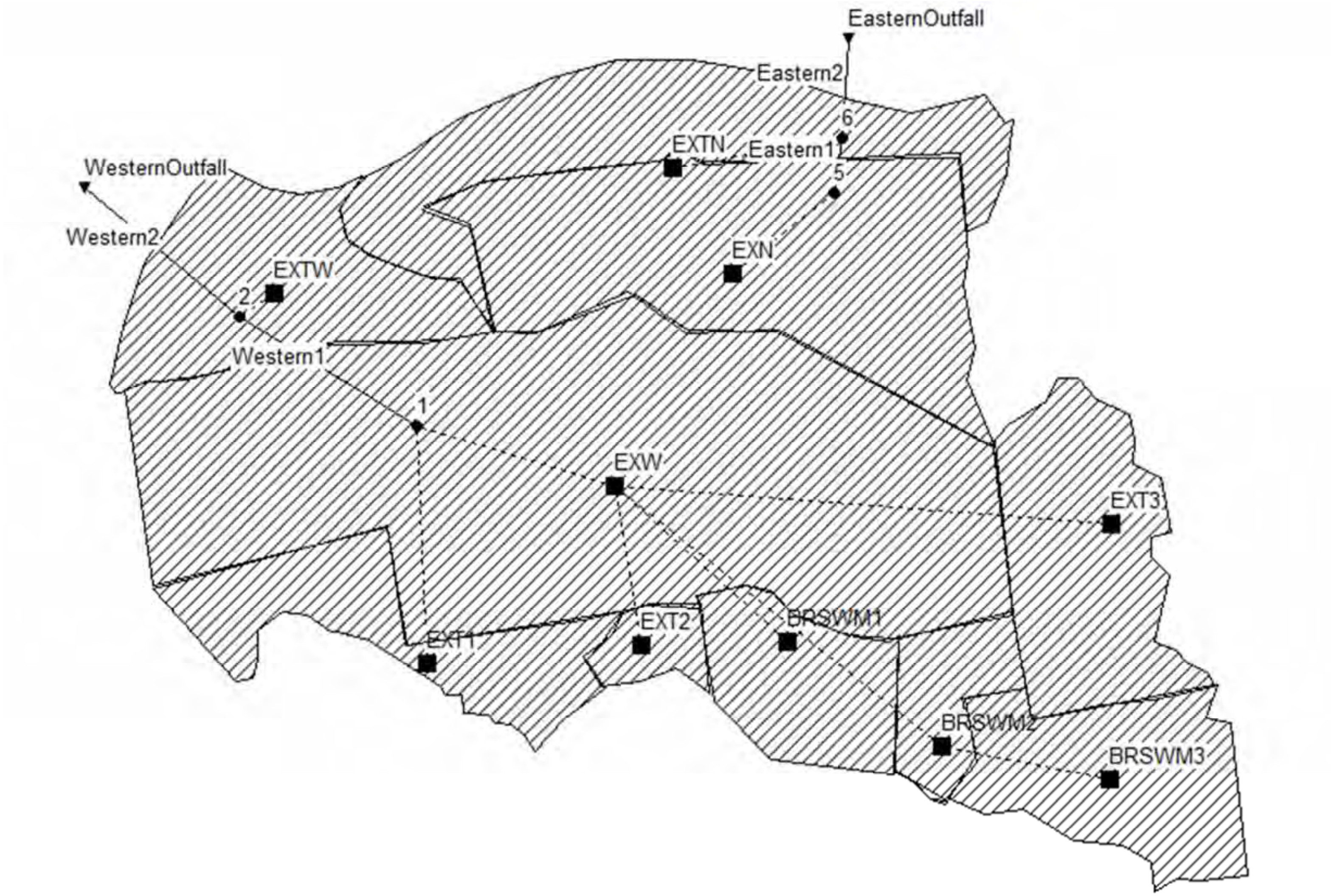


Figure 16: The EPA SWMM Model Schematic and Subcatchment Delineation

Table 14: Subcatchment Characteristics in the Study Area

Subwatershed	Subcatchment	Area (ha)	Imperviousness	Depression Storage in mm (Pervious)	Depression Storage in mm (Impervious)
Western Creek Subwatershed	EXW*	85.75	20	3	1
	EXTW	18.41	10	5	1
	EXT3	18.39	20	3	1
	EXT2	2.95	50	3	1
	EXT1	14.38	20	3	1
	BRSWM3	15.98	50	3	1
	BRSWM2	6.1	50	3	1
	BRSWM1	10.56	50	3	1
Eastern Creek Subwatershed	EXN*	36.34	20	5	1
	EXTN	23.82	10	5	1
· EXW and EXN is where future development is planned.					

Table 15: Infiltration Parameters for the Soils Encountered in the Study Area

Subwatershed	Suction Head (mm)	Conductivity (mm/hr)	Initial Deficit (fraction)
Western Creek Subwatershed	133.9	9.5	0.180
Eastern Creek Subwatershed	188.6	5.2	0.159

Hydrologic Model Results

The EPA SWMM model was developed to evaluate the hydrology of the study area for a long term spanning from 1996 to 2013. The water budget analysis based on climate data input and the EPA SWMM was processed according to the following expression:

Precipitation (P) = Surface Runoff (R) + Infiltration (I) + Evapotranspiration (ET)

With evapotranspiration and infiltration generally referred to as Losses in EPA SWMM platforms.

For the study area, the results were tabulated for all years and average annuals were estimated as presented in Table 16.

Table 16: Water Budget Analysis Results for the Study Area

Water Budget Component	Volume (mm/year)
Total Precipitation	887
Surface Runoff	120
Evapotranspiration	532
Infiltration	235

The results for average annual infiltration volume (235 mm/year) and surface runoff volume (120 mm/year) agree with previous experience with silt loam and fine sandy loam soils with similar land cover types.

Table 16 shows how evapotranspiration and infiltration represent a major part of the annual water budget for the study area. For demonstration purposes, Figure 17 shows how losses (infiltration and evapotranspiration) represented a major part of the water budget in the study area in 2013.

Surface Runoff Regime at Outfalls

The results in Table 17 show annual surface runoff volumes and Mean Annual Flows (MAF) for the two creeks at their respective outfalls (Western Outfall and Eastern Outfall; Figure 16). Observed baseflow values were added to Table 17 to provide a context for the streamflow regime at the outfalls of both creeks.

Figures 18 and Figure 19 show the surface runoff hydrographs for Western Creek and Eastern Creek outfalls, respectively.

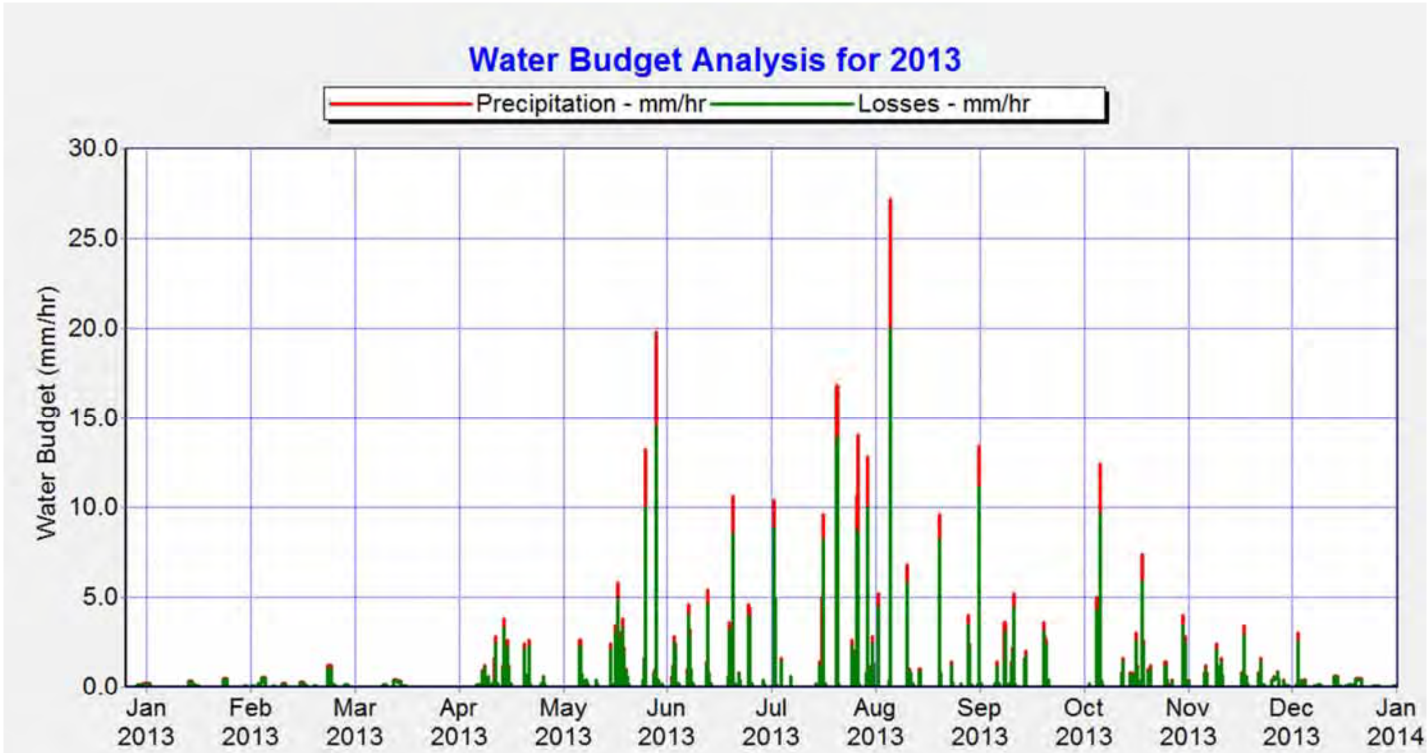


Figure 17: Water Budget Analysis Results for the months of 2013.

Table 17: Surface Runoff Volumes and Rates at the Outfalls of Western and Eastern Creeks

Subwatershed	Runoff volume/year (x 1000 m^3)	Mean Annual Flow (m^3/s)	Max Annual Flow (m^3/s)	Observed Baseflow (L/s) (DST, 2014)
Western Creek	197.6	0.025	5.5	5.3 – 5.4
Eastern Creek	79.2	0.013	4.9	0.4 – 0.8

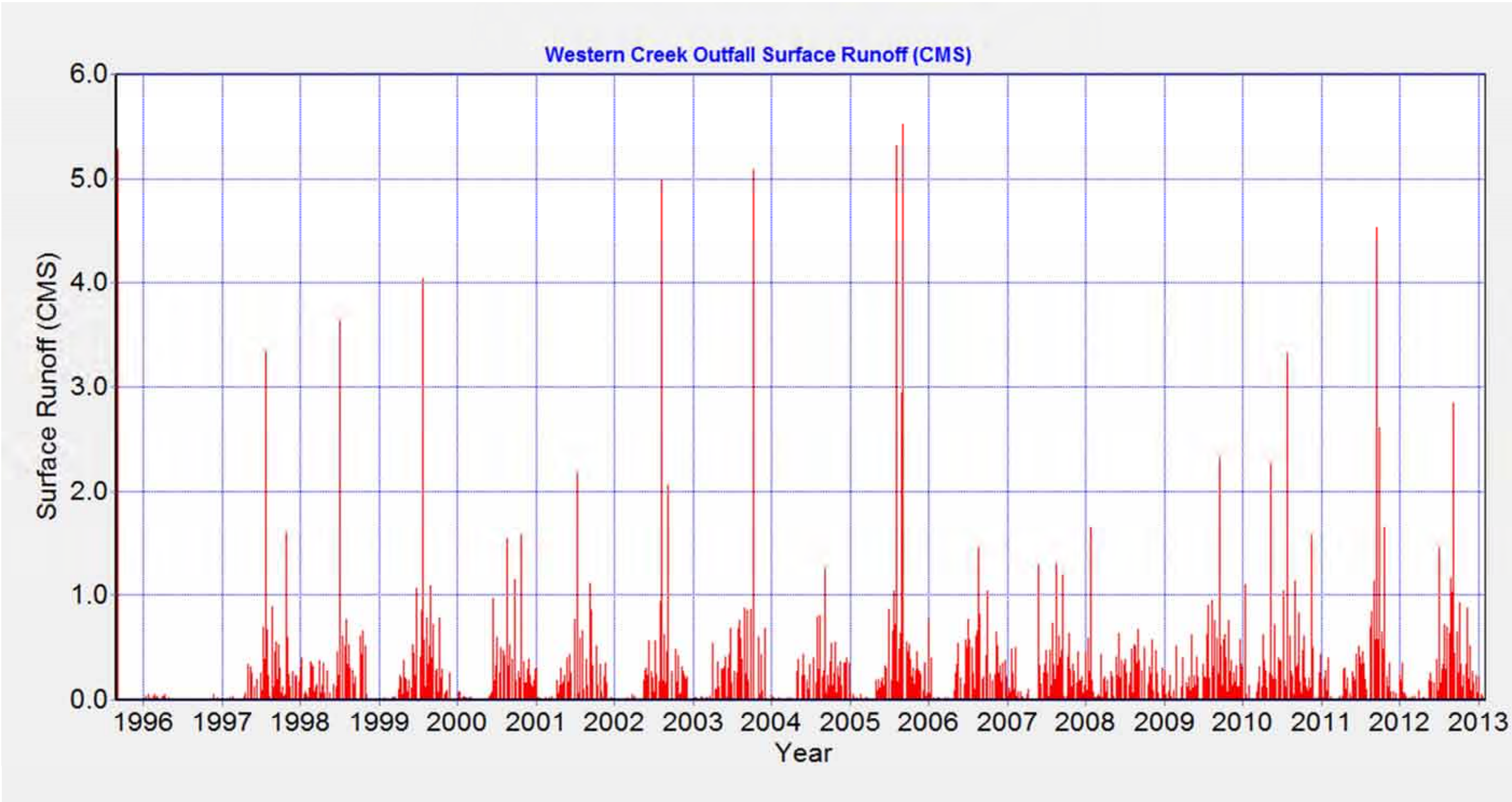


Figure 18: Surface Runoff Generated from Western Creek Subwatershed (m3/s) (1996-2013)

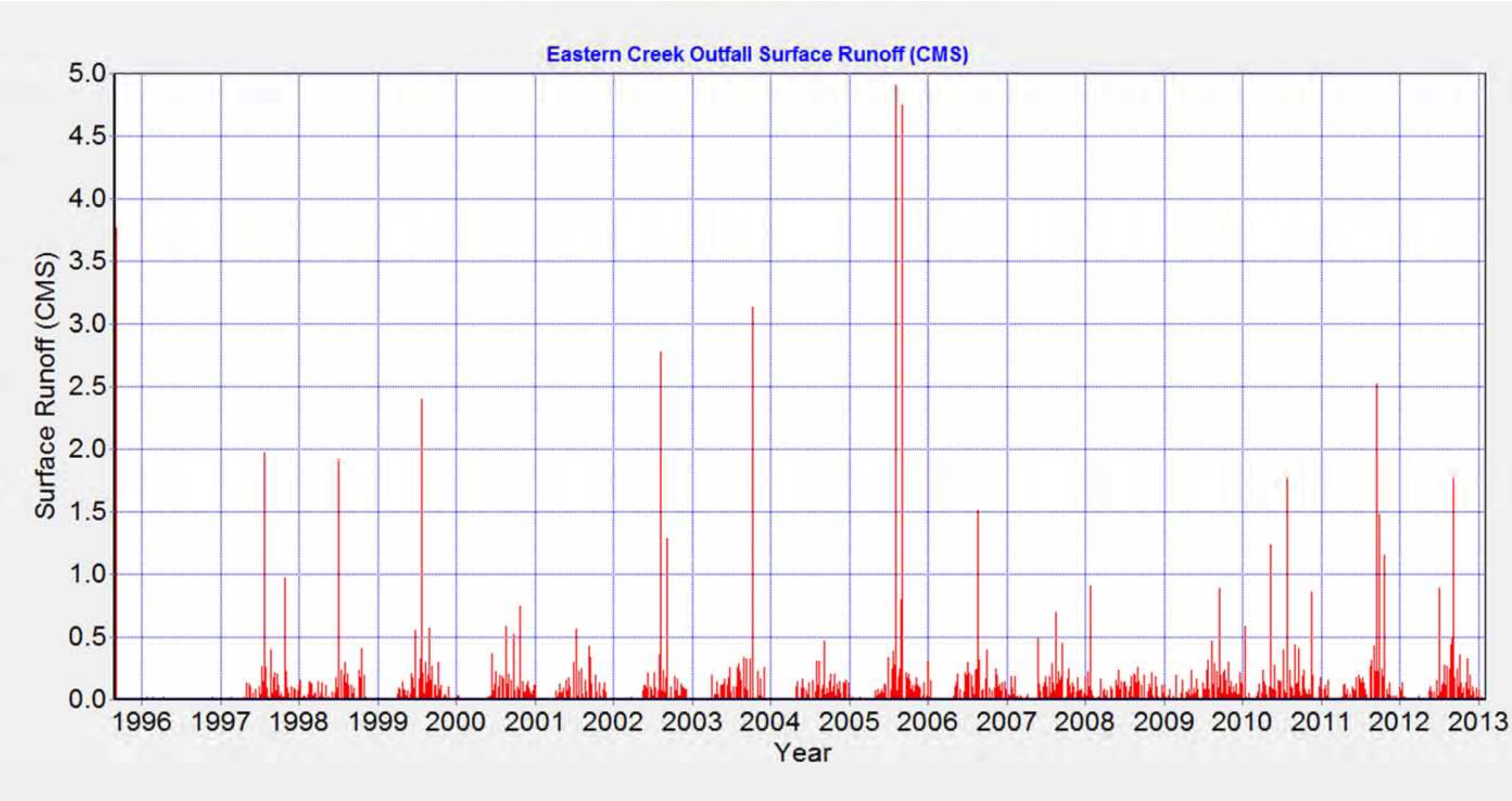


Figure 19: Surface Runoff Generated from Eastern Creek Subwatershed (m3/s) (1996-2013)

3.6 FLUVIAL GEOMORPHOLOGY

Studies were performed on the Eastern and Western Creeks in order to evaluate their fluvial geomorphological stability. Studies involved a visual assessment, a measurement of hydraulic geometry, and bed material core sampling and subsequent testing in the University of Ottawa Civil Engineering Hydraulics Laboratory (see Appendix B).

For the purpose of characterizing the fluvial geomorphology of the Eastern and Western Creek systems under existing conditions, the Fluvial Geomorphology Assessment study prepared by DST (2013) consulting engineers was reviewed. The overarching objective of the DST study was to evaluate the stability of the creeks under existing conditions and provide findings in regard to key issues and concern. Table 18 presents a summary of key findings and individual results are discussed in the subsequent sections.

Table 18: Findings of the Fluvial Geomorphology Assessment (DST, September 2013)

Geomorphologic Feature/Function	Western Creek	Eastern Creek
Channel Planform	Generally straight channel	Meandering, except for straight sub-reaches at the upstream and downstream ends.
Channel Slope	Overall slope is 1.9%. Very steep upstream of the Aviation Parkway (5%). Downstream ranges from 0.3% to 0.8%	Overall slope is 1.3%.
Dimensions	Width (2.7 - 7.05 m). Depth (0.16 – 0.41 m)	Width (2.8 – 3.2 m). Depth (0.33 – 0.44 m)
Bed Material	Median size is medium sand range with up to 21% of fine sediment.	Median size is medium to course sand range, including between 6% and 13% of fine sediment.
Bed Erodibility	Fine material within the stream bed (up to 21%) substantially reduces erosion thresholds. Critical shear stress ranges between 1.0 and 2.3 N/m^2	Fine material within the stream bed (up to 13%) reduces erosion thresholds. Critical shear stress ranges between 1.6 and 3.3 N/m^2
Impact from Existing Engineering Works	Sub-reach downstream of a culvert north of the Aviation Parkway presumably destabilized the downstream channel.	Most incised reach is located downstream of the Rockcliff Parkway culvert.
Channel Stability	Most of the sub-reaches are reasonably stable. Signs of instability between Havelock Rd and the Aviation Parkway.	Bed material is reasonably resistant to entrainment. Erosional processes are undergoing nonetheless.
Recommendations	1. It is imperative that any future stormwater detention pond designs minimize perturbation of the channel. a. Western – Report authors suggest 800 l/s will have no significant impact to the creek stability as the flow velocity and bed shear stress will not be affected b. Eastern - estimated bankfull discharge is 1500 l/s. 2. Implemented stormwater pond design should not change the input discharge or sediment load to either creek. 3. Channel Rehabilitation/ Maintenance to removed accumulated sediment/ blocked culverts: a. Western Creek - between Hemlock Rd and the Aviation Parkway & downstream of the culvert north of the Aviation Parkway	

Western Creek

The Western Creek was observed to be generally straight in its configuration. The creek is very steep upstream of the Aviation Parkway, with longitudinal bed slope of up to 5%. Downstream of the Parkway, the sub-reach slopes are reduced to less than 1%, including at least one sub-reach with bed slope of only 0.3%. Several distinct sub-reaches with different morphology are evident for the Western Creek. The most upstream channelized reach (upstream of Hemlock Road) is steep with cobble bed sediments and morphology approaching step-pool conditions. Downstream of the Aviation Parkway, conditions are relatively wide and shallow and have bed sediments with substantial portions (21%) of fine material, which greatly reduces the entrainability of these sediments. Sediments in the Western Creek generally have median grain sizes in the median sand range and include clay material, coarse cobble and muddy deposits containing a mixture of silt, clay and organic material.

The Western Creek was determined to be geomorphically stable, with most reaches lacking obvious signs of ongoing erosion. However, sub-reaches between Hemlock Road and the Aviation Parkway did show signs of instability, and a reach immediately downstream of a culvert north of the Aviation Parkway was deemed to be an eroding steep-walled gully with a very steep (5%) channel bed and incising into the consolidated clay base material. It is assumed that the channel was disturbed by flow from the existing culvert, leading to the destabilization of the downstream channel.

Eastern Creek

Several distinct sub-reaches with different morphology are also evident for the Eastern Creek. The Eastern Creek has straight sub-reaches at the upstream and downstream ends, but generally meanders, including a very tight meander bend near the downstream end. The channel hydraulics are slightly unusual because flow appears to infiltrate into the ground at several locations, including the channel terminus where the outlet to the Ottawa River does not exist. The overall bed slope is 1.3%. Sediments in the Eastern Creek have median grain sizes in the medium to coarse sand range, including bed clay and coarser gravel substrate. There are notable portions (between 6% and 13%) of fine sediments, which reduces their entrainability.

Unlike the Western Creek, the Eastern Creek has several sub-reaches that show signs of channel instability, including exposed roots on channel banks and over-sized channel dimensions. The most upstream reach appears to be the most destabilized, which is likely attributable to the culvert under the Rockcliffe Parkway and resultant potential augmentation and/or grade change.

In both creeks, it was observed that engineering works such as culverts have the potential to destabilize the channel. Therefore, it is crucial that any future stormwater detention pond designs minimize perturbation of the channel. Furthermore, due to the potential for destabilization, an implemented stormwater pond design should not change the input discharge or sediment load to either creek.

3.7 AQUATIC ECOLOGY

An aquatic habitat assessment was undertaken for the Western Creek and Eastern Creek downstream from the Former CFB Rockcliffe CDP site in order to assess potential impacts on aquatic habitat resulting from the redevelopment of the site. At the time of the assessment in 2013, the main concerns included the effects of the two (2) proposed stormwater quantity and quality management ponds located adjacent to the Western Creek in the vicinity southeast of the intersection of the Rockcliffe and Aviation Parkways and at the headwaters of the Eastern Creek at the Rockcliffe Parkway on baseflow and in-stream temperatures.

The assessment was undertaken using the Ontario Stream Assessment Protocol (OSAP) (Stanfield 2010) which includes modules for site identification and documentation, assessment of physical processes and channel structure/morphology, benthic macroinvertebrate community sampling, fish community sampling, and water temperature assessment. The ten (10) aquatic habitat assessment locations are illustrated on Figure 20 and summarized below. Source materials and the original Aquatic Habitat Assessment – Western and Eastern Creeks (DST, 2013) are included in Appendix D.

Assessment Locations

A description of each aquatic habitat assessment locations from upstream location to downstream location (Figure 20) for the Western Creek and Eastern Creek are presented below.

Western Creek

- WC5: The channel morphology site length was 40 m. WC5 was the upstream end of the Western Creek and is the first creek section where a defined channel is apparent. Further upstream water flows broadly over a wet vegetated area and the creek boundaries are poorly defined. At this location the creek in narrow and heavily overgrown with terrestrial vegetation, primarily grasses.

- WC4: The channel morphology site length was 44 m. At WC4 the creek boundaries are more clearly defined and the creek is less overgrown with terrestrial vegetation. Most of the site length is surrounded by grasses with some sections of secondary growth forest. Some areas of heavy erosion are apparent throughout this site.

- WC3: The channel morphology site length was 40 m. This site was located along the eastern tributary of the creek (the tributary flowing from the Former CFB Rockcliffe CDP site) just before it merges with the creek's western tributary. This section of the creek exhibits an increased water depth with pools formations evident in the upstream length. Most of the creek length is forested, but with significant light penetration. Significant undercutting of the banks and an enlarged channel cross-section are evident in this reach.

- WC2: The channel morphology site length was 40 m. This site was located downstream of where the confluence off the western and eastern tributaries. The banks in this reach are forested but there is extensive light penetration.