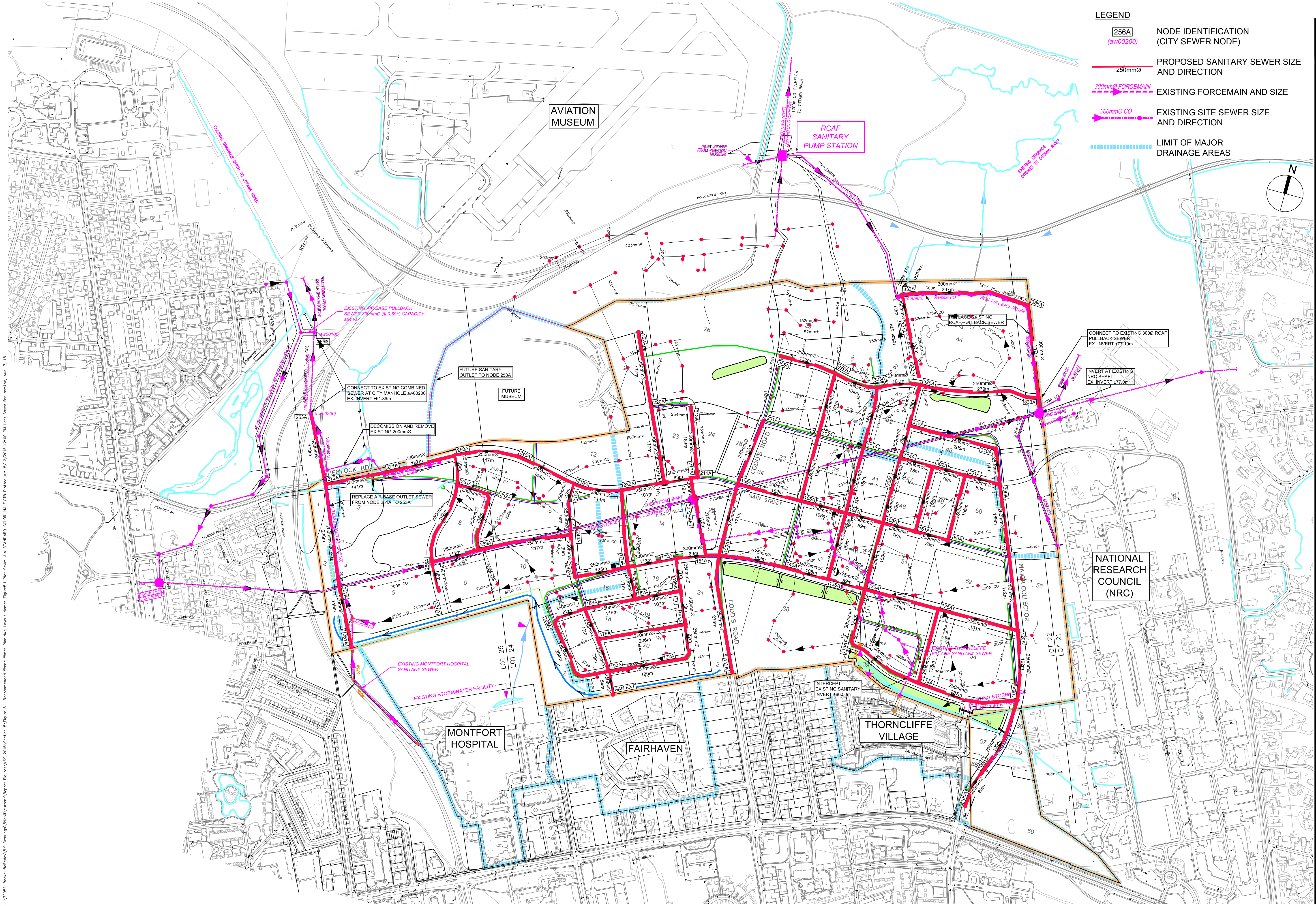


Scale

Project Title

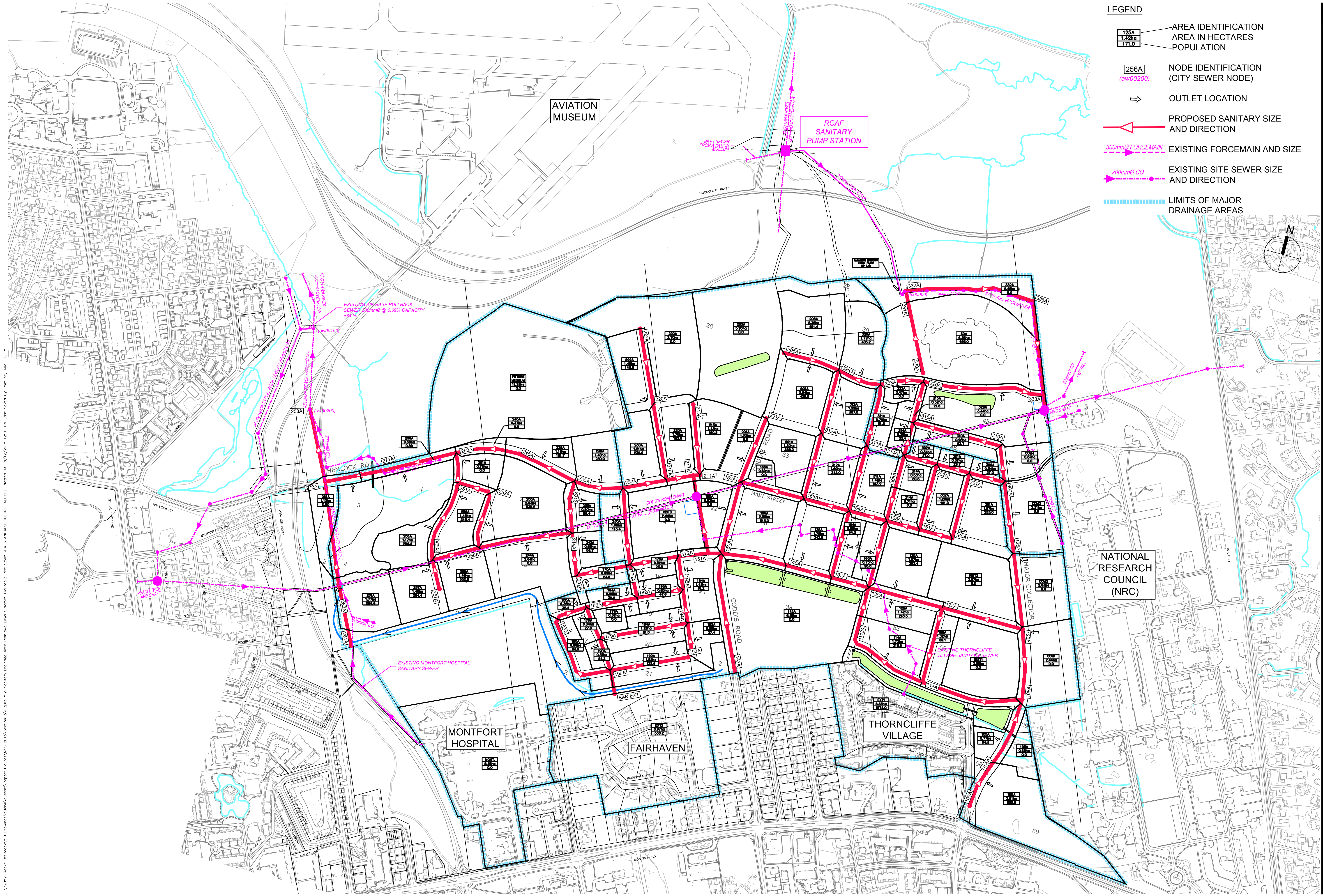
FORMER CFB ROCKCLIFFE MASTER SERVICING STUDY

Drawing Title

RECOMMENDED WASTEWATER PLAN

Sheet No.

FIGURE 5.1



Scale

Project Title

FORMER CFB ROCKCLIFFE
MASTER SERVICING STUDY

Drawing Title

SANITARY DRAINAGE
AREA PLAN

Sheet No.

FIGURE 5.2

terminating at the south limit of the former CFB Rockcliffe site. A wastewater allowance of about 11 l/s will be provided for flows from the Fairhaven development.

Wastewater flows from the Montfort Hospital are currently directed into the Airbase Outlet Sewer located near the west portion of the former CFB Rockcliffe site. It is proposed to continue to accept the hospital wastewater flows in a new sanitary sewer starting at node 261A. Based on an average institutional flow allowance of 50,000 l/day/gross ha, peaked at 1.5, an allowance of about 10 l/s is proposed to be provided for the hospital.

Future wastewater flows from the proposed 11.3 ha museum site located north of Hemlock Road are proposed to be routed to the existing 750 mm diameter Airbase Sewer by the site owner at node 253A (City sewer node aw 00200) where it can connect at the existing elevation of 61.69 m. The minimum sewer size from the museum site will be 250 mm diameter.

The Aviation Museum currently discharges wastewater flows into the RCAF Pump Station which is located north of the Aviation Parkway, and north of former CFB Rockcliffe. The pump station currently delivers flows through a 200 mm diameter forcemain to an existing combined sewer near node 332A (City sewer node rc00900) where it is then routed by gravity through the RCAF Pull-Back Sewer to the IOS trunk sewer at the NRC shaft (node 333A).

Although the RCAF Pump Station has a capacity of about 29 l/s, most of the contributing drainage areas on the subject site above the north escarpment will be redeveloped and future wastewater flows from this area are proposed to be re-routed to the RCAF Pull-Back Sewer. The City of Ottawa has estimated that the RCAF pump station is accepting only about 2 l/s from the Aviation Museum, it is however recommended that capacity in the proposed new site sewers continue to provide an allowance of 29 l/s for the pump station. If things change between the present and the time Phase 3 is designed then the question of the pump station allowance can be revisited in the future.

Combined sewage from the NRC Campus currently connects to the IOS sewer at the NRC Shaft. Combined flows from the NRC Campus are routed to an existing overflow chamber (City sewer node nr00100) through two sewers which are partially located on the former CFB Rockcliffe property. Most of the combined flow from the south portion of the NRC lands are routed to the overflow structure in the future commercial block 56 and the open space block 45 (refer to **Figure 1.3**) via an existing 750 mm diameter sewer. Combined flows from the northern portion of the NRC Campus are routed to the overflow structure in an existing 300 mm diameter sewer. The existing overflow structure is designed to pass dry weather flows directly to the NRC shaft structure (City sewer node oc00100) and wet weather flows to the Ottawa River via a 900 mm diameter overflow pipe.

It is proposed to leave the two existing NRC combined sewers intact until such time that the NRC completes its own separation plan. No allowance will be provided in the proposed site sewers for future wastewater flows from the NRC site. If and when the NRC completes a separation plan, it is assumed that plan will include dedicated sewers within its site including a new direct connection to the NRC Shaft without any impact upon the subject site. At that time, the existing NRC combined sewers, which are located on the subject site, can be decommissioned.

5.4 Proposed Outlets

As shown in **Figure 5.1**, there are three proposed sanitary sewer outlets for the former CFB Rockcliffe redevelopment. It is proposed to construct three separate sanitary sewer networks with connections to either the Airbase Outlet Sewer, the Codd’s Road Shaft or the NRC Shaft.

5.4.1 Airbase Outlet Sewer

The Airbase Outlet Sewer is shown on **Figure 2.13**. It consists of a 375 mm/750 mm diameter combined sewer located along the western edge of the former CFB Rockcliffe site. Besides providing an outlet for wastewater flow from the Montfort Hospital, it also collects combined sewage from the former Air Force base in areas west of Codd's Road. The existing sewer directs all flows to an existing overflow structure identified as node 255A (City sewer node aw00100) as indicated on **Figure 5.1**. The manhole structure at node 255A is designed to direct dry weather flows to the 300 mm diameter Airbase Pullback sewer and excess wet weather flows to a 900 mm diameter overflow pipe. Wet weather flows eventually outlet to the Ottawa River.

As part of the wastewater plan for the subject site, it is proposed to replace that portion of the existing Airbase Outlet Sewer that is located on site (i.e. between nodes 261A and 272A) as well as about another 120 m north of the subject site up to node 253A (City node aw00200)as indicated on **Figure 5.1**. The existing sewer between nodes 272A and 253A is at an elevation which is too high to permit a gravity outlet from the redeveloped Rockcliffe site without replacement.

It is proposed to continue to use the existing outlet sewer north of node 253A as shown on **Figure 5.1**. The existing invert at node 253A of the Airbase Outlet Sewer is about 61.89 m which is sufficiently deep to accommodate a gravity connection from the proposed site sanitary sewers. At node 253A, an existing 200 mm diameter combined sewer from Hemlock Road also connects to the Airbase Outlet Sewer. It is recommended that the 200 mm diameter pipe be disconnected and decommissioned from the existing structure. A future connection to node 253A is also proposed to carry wastewater from the 11.3 ha museum site. In a similar manner, it is recommended that the existing 900 mm diameter overflow pipe be disconnected from the node 255A structure and the manhole be re-benched. These latter works will ensure that only wastewater flows from the developed subject site will outlet to the Airbase Pull-Back Sewer.

5.4.2 Airbase Pullback Sewer

The existing Airbase Pullback Sewer is a 300 mm diameter pipe with an estimated full flow capacity of about 84 l/s. The sewer was reconstructed in about 2000 by the former Regional Municipality of Ottawa Carleton as part of the Alvin Heights Pull-Back Sewer Reconstruction. As per the Sanitary Sewer Design Sheet and the Sanitary Sewer Drainage Area Plan, both of which are included in **Appendix D**, an estimated peak wastewater flow of 62 l/s, which also includes an allowance for the future museum site, will be directed to the Airbase Pullback Sewer and eventually to the IOS trunk sewer.

5.4.3 Codd's Road Shaft

The Codd's Road Shaft location is indicated on **Figure 2.13**. This connection to the Interceptor Outfall Sewer was added in 1991 as part of the development of the Thorncliffe Village community. A dedicated sanitary sewer from that development collects wastewater and routes it directly to the IOS without any connections to the other existing sewers on the subject property. **Figure 2.14** provides a schematic of the existing Codd's Road Shaft details. Because the existing connection to this shaft delivers only wastewater flows and not combined sewage, there is no overflow at this shaft.

The connection includes a standard manhole with a 375 mm diameter inlet pipe. However, because the site is about 35 m above the Interceptor sewer, the manhole outlet includes a 300 mm diameter vertical steel casing sewer connecting directly to the Interceptor sewer. The existing manhole base elevation is approximately 83.00 m.

Based on the proposed wastewater plan, two connections to the existing manhole are proposed. However, these will both be lower than the existing manhole bottom so it is proposed to replace the manhole with a new structure. **Figure 5.3** provides some preliminary details of the proposed new connection at the Codd's Road Shaft. Essentially, the current connection will be lowered as needed to accommodate the two new inlet sewers. The elements of the vertical outlet pipe will remain unchanged but will be adjusted to a lower elevation.

The estimated peak wastewater flow proposed to be connected to the Codd's Road Shaft is about 156 l/s. Based on the Ontario Building Code (Section 7.5.8.4, 2012) the capacity of a 300 mm diameter vertical pipe is 2700 GPM or 170 l/s. The OBC assumes only 29% of the pipe area is used by wastewater. A copy of Table 7.5.8.4 and Figure 13-7 from the OBC is included in **Appendix D**.

5.4.4 NRC Shaft

The third proposed connection to the IOS sewer is at the NRC Shaft where combined flows from the NRC property; wastewater flows from the Aviation Museum and combined flows from the eastern portion of the former airbase site are presently connected. **Figures 2.15** and **2.16** show the existing connection details.

Flows from the NRC Campus enter an existing overflow manhole structure from two separate combined sewers. The overflow structure is designed to direct dry weather flow to the nearby NRC Shaft drop structure which in turn connects to the IOS sewer. The overflow structure has a 900 mm diameter overflow pipe which directs larger wet weather flows away from the site to the Ottawa River. Besides the dry weather flows from the NRC overflow chamber, the drop manhole also receives flows from the 300 mm diameter RCAF Pullback Sewer. The drop manhole outlets through a vertical drop pipe directly to the Interceptor sewer located about 35 m below the drop manhole.

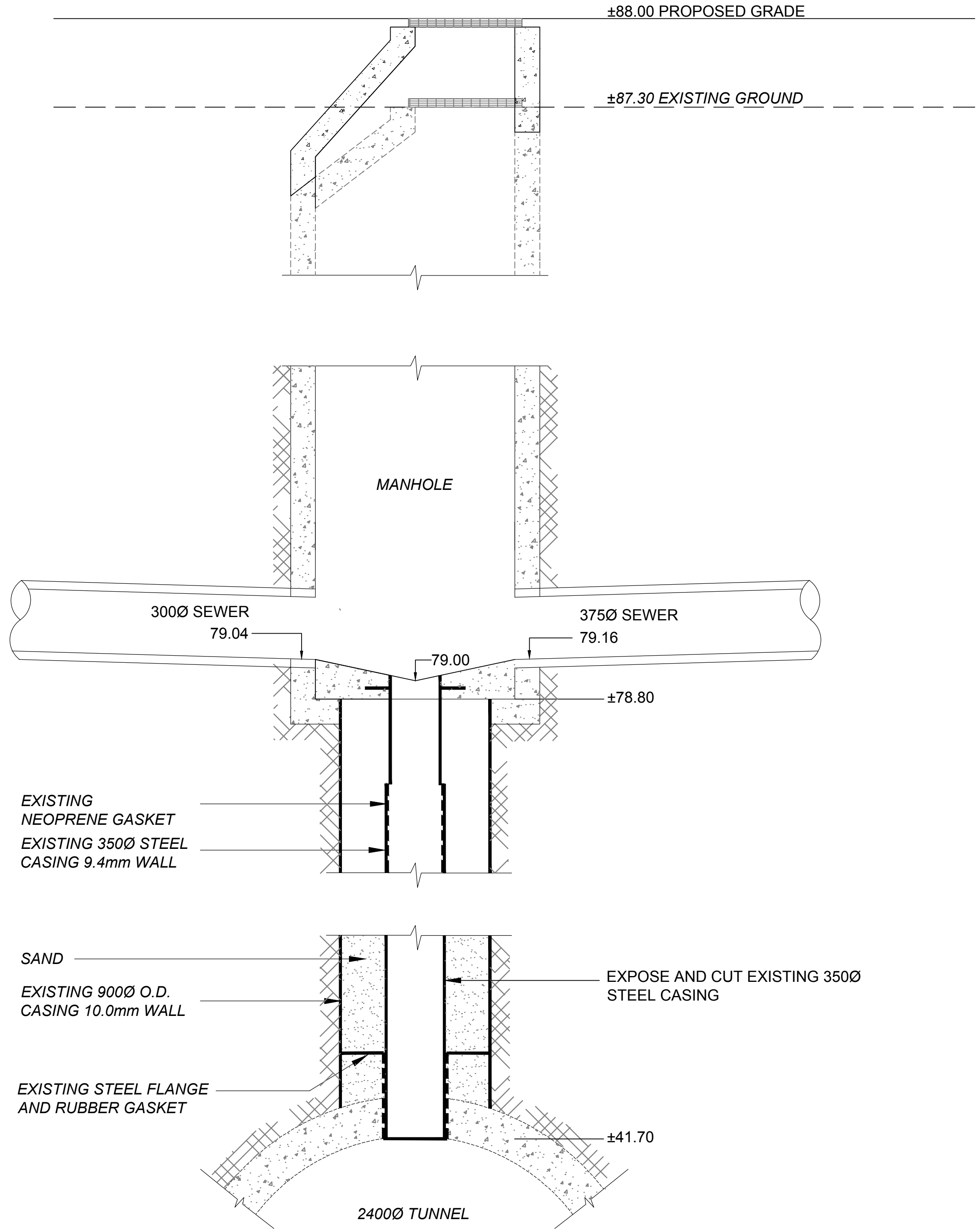
Both the NRC overflow chamber and adjacent incoming and outgoing overflow pipes are proposed not to be impacted by the proposed site development. The drop manhole is also proposed to remain as-is. Future separated flows from Phase 3 and the RCAF Pump Station are proposed to connect directly to the existing 300 mm diameter RCAF Pullback Sewer at a location about 40 m upstream of the NRC Shaft. The balance of the RCAF Pullback Sewer is proposed to be replaced as part of the future design of Block 44, which is considered the Special Design Area (Block 44, refer to **Figure 1.3**). The pullback sewer is connected to the NRC Shaft at an elevation of about 77.0 m. Existing ground is close to the 84.0 m contour, so the existing infrastructure is sufficiently deep to accept a new connection from the proposed site sanitary sewers.

The City has commented that the existing pullback sewer is located in a forested area. It is likely that every reasonable attempt will be made to retain existing significant vegetation. In that event the pullback sewer, as well as incoming pipes such as the pump station forcemain, may be relocated. Those decisions will be made at the time of final concepts and future design the Special Design Area.

The estimated peak wastewater flow proposed to be connected to the RCAF Pullback Sewer is 49 l/s. This includes an allowance of 29 l/s for the RCAF Pump Station. Based on a measured slope of 0.43%, the existing sewer has a capacity of about 63 l/s.

5.5 Phasing Requirements

Figure 1.6 shows the proposed phasing plan for development of former CFB Rockcliffe. Phase 1 will include about 82 ha of the central and eastern portions of the site and will be sub-phased into phases 1A and 1B. Phase 1A will include Codd's Road up to the Town Centre and most of the low density residential development areas west of Codd's Road; a school site and park. The



IBI

Project Title	Drawing Title	Sheet No.
FORMER CFB ROCKCLIFFE MASTER SERVICING STUDY	PROPOSED CONNECTION CODD'S ROAD SHAFT	FIGURE 5.3

Phase 1 wastewater sewers will also be sized to accept flows from about 14 ha of the western portion of Phase 3. Wastewater flow from Blocks 29 to 32 is proposed to be collected and directed towards the Codd's Road Shaft.

Because of the need for multiple access points for watermains, vehicles and utility looping, Phase 1A will also include the extension of both Main Street and Burma Road. However, the latter streets are included in Phase 1A only for infrastructure redundancy. No developments along these streets are proposed until Phase 1B is completed. Phase 1A will also include capacity for potential future urbanization of the Fairhaven community as well as interception and collection of existing sanitary flows from Thorncliffe Village. All wastewater from Phase 1, and a portion of Phase 3, will be directed to the proposed reconstructed Codd's Road Shaft. **Figure 5.4**, which is included in **Appendix D**, shows the proposed wastewater plan for Phase 1A. For quick reference, a reduced copy of this figure is included herein.

The ultimate wastewater servicing plan for the subject site includes sanitary sewers in Burma Road. As part of the Phase 1A development, it is proposed to construct Burma Road from Montreal Road to Hemlock Road, including all proposed infrastructure. The Burma Road sanitary sewers in that portion of Burma Road are proposed to connect at node 108A and ultimately outlet westward to node 114A. However, the single loaded local road containing the 108A to 114A sanitary sewer is not planned for Phase 1A construction. Therefore, until Phase 1B is completed, the Burma Road Phase 1A sanitary sewer will remain out of service. Because this sanitary sewer will not be connected to any developments and therefore will not receive wastewater flows until Phase 1B is constructed, it is probable that it could fill with groundwater. To ensure water levels within this sanitary sewer do not surface, it is proposed to cap the sanitary sewer at the west leg (outlet side) of node 108A and construct a temporary drainage pipe designed to outlet to the new Burma Road SWM Facility. The "dry" sanitary sewers in Main Street will naturally outlet to the Codd's Road sewers at node 155A, thus ensuring they will not flood.

Phase 2 will include development of approximately 24 ha west of Phase 1. Wastewater flows from this phase will be directed to a new outlet sewer which will replace most of the existing 750 mm diameter Airbase Outlet Sewer located near the northwest of the subject site. Sanitary sewers in Phase 2 will also intercept and collect existing flows from the Montfort Hospital. The federal government may develop the museum site. Wastewater flows from that site can be collected and directed to the existing Airbase Outlet Sewer at node 253A as indicated on **Figure 5.1**.

Phase 3 will include development of the north and northeast portions of the site. Part of the wastewater in Phase 3 will be directed to the IOS via the new RCAF Pullback Sewer and the balance towards the Codd's Road Shaft via Phase 1 sewers. Also as stated earlier, a wastewater allowance will also be included for the RCAF Pump Station.

Development of Phase 1A will impact the existing Thorncliffe Village sanitary sewer. It is proposed to intercept that sewer at the new Codd's Road alignment near node 150A and convey flows in the new Phase 1A sanitary sewer to the IOS. The balance of the Thorncliffe Village sanitary sewer can remain active until Phase 1B is developed.

5.6 Ottawa Interceptor Sewer

The Ottawa Interceptor Sewer (IOS) is a 2.4 m diameter trunk sewer which carries combined sewage from a large portion of the City of Ottawa to the sewage treatment facility located several kilometres east of the subject site. The sewer bisects the subject site about 35 m below grade. The sewer has been in operation for about 50 years and the City is presently considering twinning the sewer to provide additional capacity and operational redundancy.

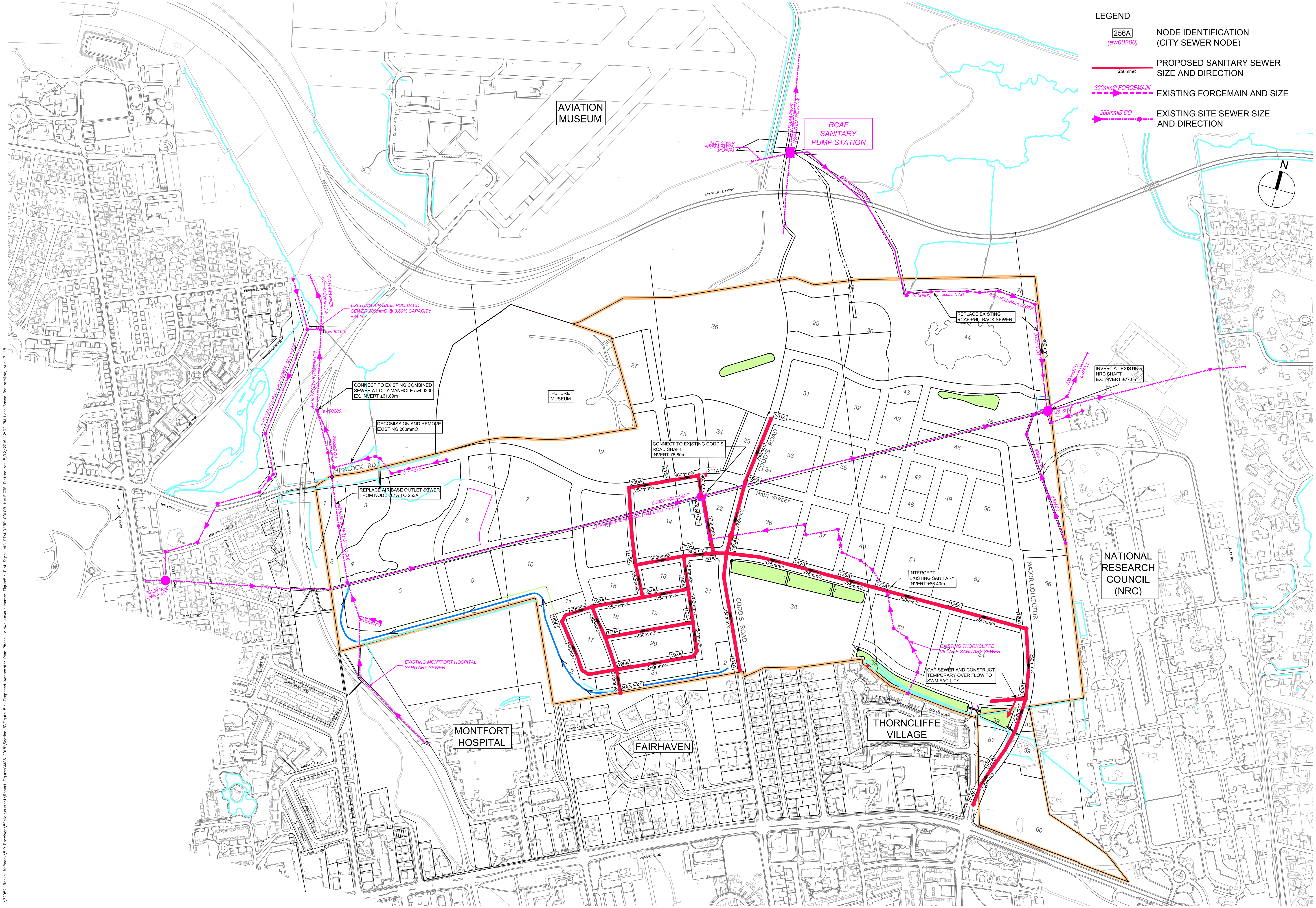


FIGURE 5.4

Sheet No.

RECOMMENDED PHASE 1A
WASTEWATER PLAN

Drawing Title

FORMER CFB ROCKCLIFFE
MASTER SERVICING STUDY

Project Title

Scale

The City has not yet completed an Environmental Assessment for the proposed new trunk sewer. Present options are only preliminary and currently there is no work underway to develop and evaluate potential options. Twinning of the IOS is likely a generation away. Until the City advances the twinning project, it is unknown if the new trunk sewer will be located below the subject site. The sewer could also be located in corridors either north or south of the former CFB Rockcliffe site. Even if the new sewer is ultimately constructed below the subject site, it will most likely be completed by trenchless technologies at similar elevations as the existing pipe.

The proposed development will commence as soon as the CDP, and supporting technical documents such as this report, are reviewed and approved. It is conceivable that the first development applications will be submitted in mid 2015 with construction of Phase 1A starting in 2015.

Through the development review process, a strata easement will be secured to protect the City's ability to access and maintain the existing IOS tunnel. As the CDP accommodates the existing shafts to the IOS within future City parks, full and final easements will be protected. A new permanent IOS shaft is also needed and is anticipated to be built within one of the proposed parks. This has been taken into account through the strategic location of parks where the additional shaft and associated maintenance staging areas would be required.

From an infrastructure perspective, the preferred location for the new IOS shaft has been identified as Parkette #5 – Centre Parkette (block 22), adjacent to the existing drop shaft. Other possible locations include the eastern edge of Park #4 – East Neighbourhood Park (block 45), west of the existing shaft, and the north-west corner of Park #3 – West Neighbourhood Park (block 10). Construction of the new shaft would last a few months and would require 2,000 – 3000 m^2 of staging area. The staging area would revert to park use upon completion of the shaft construction. Criteria for locating the new shaft will be based on consultation with affected stakeholders. If the new shaft is not located on City owned land, a full and final easement will be required for access to the new shaft.

The existing IOS tunnel will also be twinned with a second sewer tunnel in the future. Subject to the ultimate routing selected for the second tunnel and its associated shaft, a second subsurface strata easement may be required to protect for the possible routing of this second tunnel and for its associated access shaft. It is also anticipated that construction of the second sewer tunnel would take place after development and occupancy of surrounding land. Accordingly, the City will engage with affected stakeholders.

5.7 Cost Estimates

As stated earlier in this report, the proposed wastewater plan for the former CFB Rockcliffe site will make use of three different outlets. This means that the proposed plan will require only nominal size sanitary sewers with no sewer being larger than 375 mm diameter. Therefore, based on the current City of Ottawa's Development Charge By-Law, which only starts to share wastewater sewer costs for sewers sized 450 mm diameter and larger, no proposed sewers will be cost shared by the City. Accordingly, no cost estimates of the proposed wastewater system are provided in this report.

5.8 Conclusions

It is recommended that as part of the development of the former CFB Rockcliffe site that all existing combined sewers be replaced with dedicated wastewater and storm sewers. The new wastewater sewers will be designed using the criteria recommended by the City of Ottawa as noted in its Ottawa Design Guidelines for Sewers.

All wastewater flows from the former CFB Rockcliffe redevelopment will be directed to the Ottawa Interceptor Outfall Sewer (IOS), which is located on the site. Wastewater flows are proposed to be routed to one of the three existing connection shafts located either adjacent to or on the site. Flows from the western portion of the site are recommended to be directed to a new sanitary sewer along the western edge of the site which will connect to the Airbase Pullback Sewer and eventually reach the IOS at the Peach Tree Road Shaft via the existing Alvin Heights Pullback Sewer.

The proposed wastewater system is also recommended to be sized to accept existing wastewater flows from the Montfort Hospital. Wastewater flow from the future museum can be directed to the existing Airbase Pullback Sewer at node 253A by the museum site owners.

Separated wastewater flows from the central and southwestern portion of the site are recommended to be routed to the existing Codd’s Road Shaft located near the centre of the site. The new wastewater system for Phase 1 will also be sized to accept future flows from the Fairhaven Community in the event the existing private septic systems are replaced with a central piped wastewater system. The Phase 1 sewer will also intercept existing flows from the Thorncliffe Village sanitary sewer system.

Wastewater flows from the northeast portion of the site are recommended to be routed to the IOS at the existing NRC Shaft. It is also recommended that most of the existing RCAF Pullback Sewer be replaced leaving only about 40 m of the lower section, which connects to the drop structure, in place. The new sewer will also be sized to continue to accept flows from the RCAF pump station. Because Block 44, on which the existing RCAF Pullback Sewer is located, is a Special Design Area, details of the development of that area, including new infrastructure, will be reviewed in the future as part of the final plan and design of the block.

It is also recommended that the new site development not impact the two existing NRC combined sewers which are located on the subject site. These sewers will continue to function until such time that the NRC elects to complete its own separation program, at which time the existing sewers can be decommissioned. No provision is proposed to be made in the proposed new wastewater sewer for flows from the NRC Campus.

6 Stormwater Management System

6.1 Introduction

It has been concluded that the existing storm sewer system within former CFB Rockcliffe has reached its useful life and the site's redevelopment should include the construction of a new separated storm sewer system including a dual drainage network and end-of-pipe SWM facilities. The proposed storm sewer system is designed to not only convey runoff from the former CFB Rockcliffe site but to also convey runoff from several external areas. Capacity for stormwater runoff from the NRC Campus, Thorncliffe, Foxview, and Fairhaven communities, as well as the Montfort Hospital, is provided in the new stormwater system. Regard has also been given to the long term potential for a new museum to be built in the northwest corner of the proposed development plan.

6.2 LID Stormwater Pilot Project

As noted in **Sections 1.5** and **1.6**, the City of Ottawa and CLC have agreed to pursue phased stormwater management demonstration projects for former CFB Rockcliffe using LID Best Management Practices (BMPs). Aquafor Beech was retained by CLC to prepare the "Former CFB Rockcliffe LID Stormwater Management Pilot Study Project" (May 2015). The report is being read concurrently with this report.

The MSS has been completed using sound engineering principles in the development of the preferred stormwater solution applying conventional stormwater practices including, but not limited to, piped stormwater infrastructure and stormwater management facilities. The preferred stormwater solution using conventional stormwater practices has been developed in accordance with regulatory requirements to service the proposed development as a stand-alone system and to accommodate the potential LID practices identified in the second independent study detailed below.

The 'Former CFB Rockcliffe LID Stormwater (SWM) Pilot Project Study' being prepared by Aquafor Beech is intended to permit the implementation, monitoring, and evaluation of alternative stormwater management systems based upon the principles of low impact development. The work program for the LID Stormwater Pilot Project Study was developed in consultation with the City of Ottawa and will provide direction for the implementation of LID controls in parallel with the conventional storm servicing presented in the MSS.

A SWM Working Group has been formed, consisting of key members of the City of Ottawa, CLC staff, and consultants engaged by CLC. The SWM Working Group will:

- Make recommendations to the City of Ottawa staff working on former CFB Rockcliffe
- Determine the information required to design the integrated LID SWM system
- Determine the information to be collected during the monitoring programs
- Review the collected monitoring data from each successive phase of development
- Determine how and to what extent LID measures will be implemented beyond the initial phase based upon monitoring and whether accounting for some or all LID benefits in the design of the conventional storm servicing can be supported.

The proposed LID implementation process for former CFB Rockcliffe was developed using a phased Adaptive Management System (AMS) approach whereby a science-based methodology is developed and applied to understand and quantify the function, potential benefits and drawbacks from the proposed LID approaches. In this regard, each phase of potential LID implementation corresponding to the phases of development will involve the completion of six steps:

1. Planning (subject of the LID SWM Pilot Project Study)
2. Construction
3. Monitoring
4. Reporting
5. Quantification of benefits
6. Refinement of the LID approach prior to the subsequent phase of LID implementation

Steps 5 and 6 (Quantification of benefits and Refinement) are critical process elements, allowing ‘real-world’ results to be communicated to and vetted by City of Ottawa staff and agencies and subsequently translated into direction for both the refinement of future LID implementation and refinement of the conventional storm servicing presented in the MSS. In this manner, it is only the quantifiable benefits that will influence stormwater servicing of former CFB Rockcliffe.

As discussed in the previous section, the LID SWM Pilot Project Study report should be read in parallel with this MSS document as it relates to storm sewer servicing. Furthermore, forthcoming monitoring reports which quantify the performance of LID practices as it relates to water quality, water balance, volume, and peak flows should also be considered and related to the aforementioned two independent documents. The implementation of stormwater servicing for redevelopment of former CFB Rockcliffe will be flexible and realizes that stormwater BMPs, techniques and approaches will change as the knowledge base advances, that future phases and associated monitoring will refine the findings from the MSS and LID SWM Pilot Project Study and that this overall stormwater servicing approach is not static.

An overall study process is detailed in **Figure 1.5**. It summarizes the MSS, CDP, LID SWM Pilot Project Study and the proposed LID implementation process for former CFB Rockcliffe and uses the AMS approach.

6.3 Stormwater Management Criteria

6.3.1 Regulatory Agencies

The following agencies are involved in the review of the SWM servicing for former CFB Rockcliffe:

- City of Ottawa
- Rideau Valley Conservation Authority
- Ontario Ministry of the Environment
- National Capital Commission

6.3.2 Reference Documentation

The following documents were referenced as part of the establishment of the SWM criteria:

- Stormwater Management Planning and Design Manual (Ontario Ministry of the Environment, March 2003);
- City of Ottawa Sewer Design Guidelines (November 2004 and September 2008 Section 8.0), henceforth referred to as the OSDG;

- City of Ottawa Technical Bulletin ISDTB-2012-1 (January 31, 2012), henceforth referred to as Technical Bulletin 2012-1;
- City of Ottawa Technical Bulletin ISDTB-2012-4 (June 20, 2012), henceforth referred to as Technical Bulletin 2012-4; and,
- City of Ottawa Technical Bulletin ISDTB-2014-1 (February 5, 2014), henceforth referred to as Technical Bulletin 2014-1.

6.3.3 Regulatory Requirements

6.3.3.1 Water Quantity Control

As previously noted in **Section 2.4.3**, runoff from the site is currently conveyed to two creeks via two existing culvert crossings of the Aviation and Rockcliffe Parkways.

A preliminary analysis has been completed based on available data for the culvert crossings. Culvert design nomographs for the existing eastern culvert (Rockcliffe Parkway) and normal depth of flow within the existing ditch downstream of the culvert were analyzed to determine maximum capacity of the existing eastern outlet system. The existing western culvert (Aviation Parkway) was modeled in XPSWMM to define the quantity of the flow conveyed through the culvert under the post development condition (further details are provided in **Section 6.6.1)**.

Based on the analysis, runoff from the eastern limit of the site is currently constrained by the capacity of the existing eastern culvert crossing. Modeling results of the western limit of the site shows that runoff from the site will be conveyed through the existing western culvert with no overflow. A summary of the eastern and western culvert analysis is provided in **Table 6.1**. Culvert design nomographs and modeling files are enclosed in **Appendix E.**

- Eastern (Rockcliffe Parkway) Outlet

  The results of the analysis indicate that the culvert will overflow at an elevation of approximately 54.80 m. Based on this condition, the theoretical conveyance of the culvert with no overflow is approximately 3.5 cms.

- Western (Aviation Parkway) Outlet

  The results of the modeling indicate that the culvert will overflow at an approximate elevation of 62.01 m. Based on this condition, the conveyance of the culvert with no overflow is approximately 1.35 cms.

Table 6.1 Summary of Culvert Analysis

OUTLET	CAPACITY (CMS)
Eastern (Rockcliffe Parkway)	3.50
Western (Aviation Parkway)	1.35

As previously discussed, the preferred SWM alternative is comprised of two end-of-pipe SWM facilities with outlets to the Ottawa River. Construction of the Eastern SWM Facility, adjacent to the Rockcliffe Parkway, will include a new pipe conveying outflow from the facility to the Ottawa River. Due to the direct connection to the Ottawa River, the Eastern SWM Facility is not required to provide water quantity control. Additional field work and consultation with the NCC and RVCA will be required as part of detailed design.

The Western SWM Facility is tributary to the Aviation Parkway culvert crossing. Enhancement to remove debris from the Aviation Parkway culvert, remove silt build up in the creek at the outlet of the Aviation Parkway culvert, and remove the blockage in a culvert within the RCMP campus is required to restore a positive outlet.

6.3.3.2 Water Quality Control

The SWM facilities servicing the site will provide an Enhanced Level of Protection, which corresponds to 80% TSS removal as per the Stormwater Management Planning and Design Manual (Ontario Ministry of the Environment, March 2003).

6.3.3.3 Erosion Control

It is recommended that the future SWM facility designs minimize perturbation of the creek channels. The Eastern and Western Creeks are characterized in the Fluvial Geomorphology report.

The Eastern Creek is characterized by eroding, incising reaches separated by depositional wet grassy areas. The estimated bankfull discharge is between 0.5 cms and 2.5 cms. Since the proposed stormwater management approach calls for dedicated storm sewers which will outlet to either one of two central SWM facilities, potentially less water from the subject site will discharge to the exiting Eastern Creek. All excess flows are proposed to be directed away from the Creek. From the fluvial geomorphology perspective, the estimated flow generated by the contributing drainage area during the 25 mm storm event is 0.56 cms, which is at the lower end of the bankfull estimates. As such, exacerbated erosion is not anticipated. Additionally, runoff reduction by LIDs has not been considered. Therefore, no erosion control or any remediation works are currently proposed for the Eastern Creek.

It is proposed to conduct a 3 year monitoring program within the Eastern Creek starting in spring 2016. The monitoring program will include annual inspection with photo documentations for evidence of erosion, souring or deposition of sediment. In addition, it is proposed to identify two sections, which are most susceptible to erosion, and to conduct yearly topographical surveys after the spring freshet. The locations of monitoring stations will be identified at the detailed design stage. The final monitoring program should confirm if any remedial/mitigation work should be adopted that the Creek is not deteriorating as a result of the proposed development.

The Western Creek is generally stable except a reach immediately downstream of a culvert north of the Hemlock Road. The estimated bankfull discharge is between 0.7 cms and 1.4 cms downstream of the Aviation Parkway and upstream of the confluence with the western tributary, as discussed earlier in **Section 2.3.2.** As stated previously, the SWM approach for the Western Creek is to discharge only that volume of treated flow that matches the capacity of the Aviation Parkway culvert. Flows in excess of that capacity will be diverted away from the Western Creek and routed directly to the Ottawa River via a new pipe. Further discussion is provided within the **Section 6.5.4.4**.

Sections of the Western Creek need to be cleaned out to re-establish its maximum capacity. For a short distance below the Aviation Parkway culvert, the existing Creek is full of silt and farther downstream a Creek crossing is plugged. Part of the SWM plan for the Western SWM facility includes the cleaning of the Creek to restore its maximum capacity. **Figure 2.18** identifies some of the creek features discussed above. It should be noted that cleaning of the Western Creek is proposed to be completed concurrent with construction of the Western Pond during the Phase 2 development.

6.3.3.4 Temperature

With respect to temperature, the Aquatic Habitat Assessment indicates that temperature mitigation measures should not be required for the outflows from future SWM facilities.

6.3.3.5 Water Budget

6.3.3.5.1 Background

A water budget analysis has been completed by Aquafor Beech for existing conditions. It 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 the water budget analysis is to analyze pre-development 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
    - Field confirmation of watercourse mapping
    - Evaluation of Infiltration capacity using the Guelph Permeameter
- Technical reports covering the study area, including
    - “Fluvial Geomorphology Assessment Study,” DST (September 2013)
    - “Preliminary Geotechnical Investigation Study Report Infiltration Ponds,” DST (November 2013)
    - “Hydrogeological Report Stormwater Management Support Study,” DST (June 2014)
    - “Stormwater Management Existing Conditions & LID Pilot Project Scoping,” Aquafor Beech (May 2015)
- Historical metrological data (Environment Canada and City of Ottawa Weather Stations), including
    - Hourly precipitation values
    - Air temperature
- Soils data, including
    - “Stormwater Management Existing Conditions & LID Pilot Project Scoping,” Aquafor Beech (May 2015)
    - Field Observations

6.3.3.5.2 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.

6.3.3.5.3 Climate Data

Precipitation data from four rain gauges belonging to the City of Ottawa were reviewed. These stations included Lemieux, Hawthorne, Lee's, and ROPEC. The 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 6.1**).

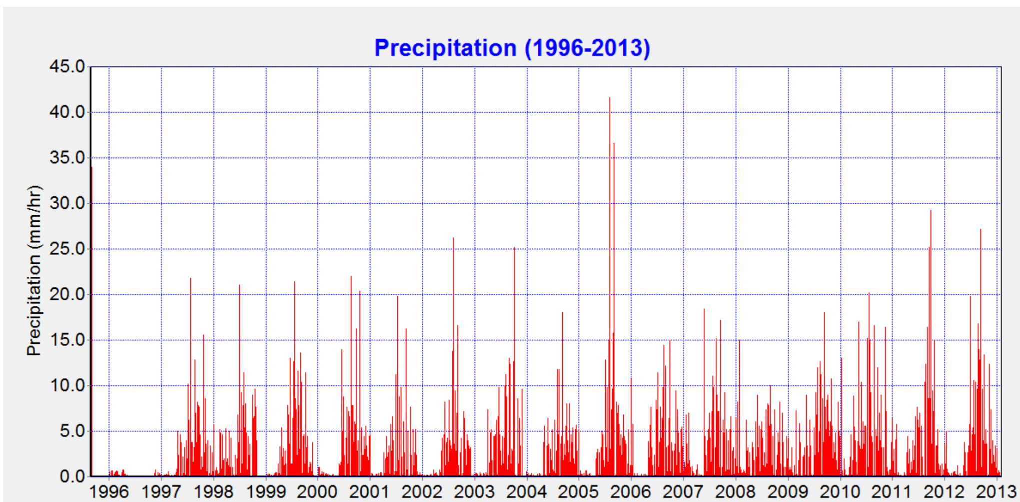


Figure 6.1 Precipitation Record (mm/hr) from 1996 to 2013

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 6.2**). 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.

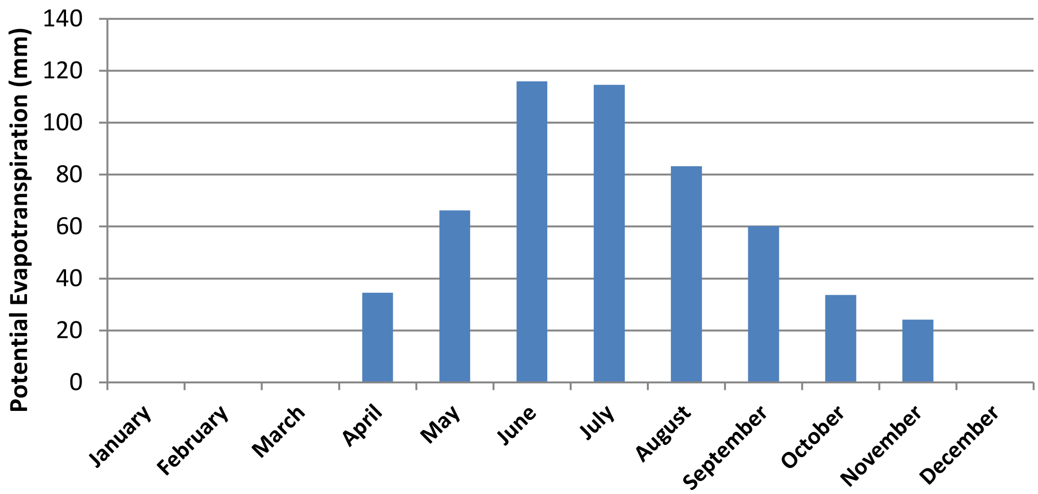


Figure 6.2 Average Potential Evapotranspiration Values between 1996 and 2013

6.3.3.5.4 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 6.3** shows a schematic of Western Creek and Eastern Creek subcatchments.

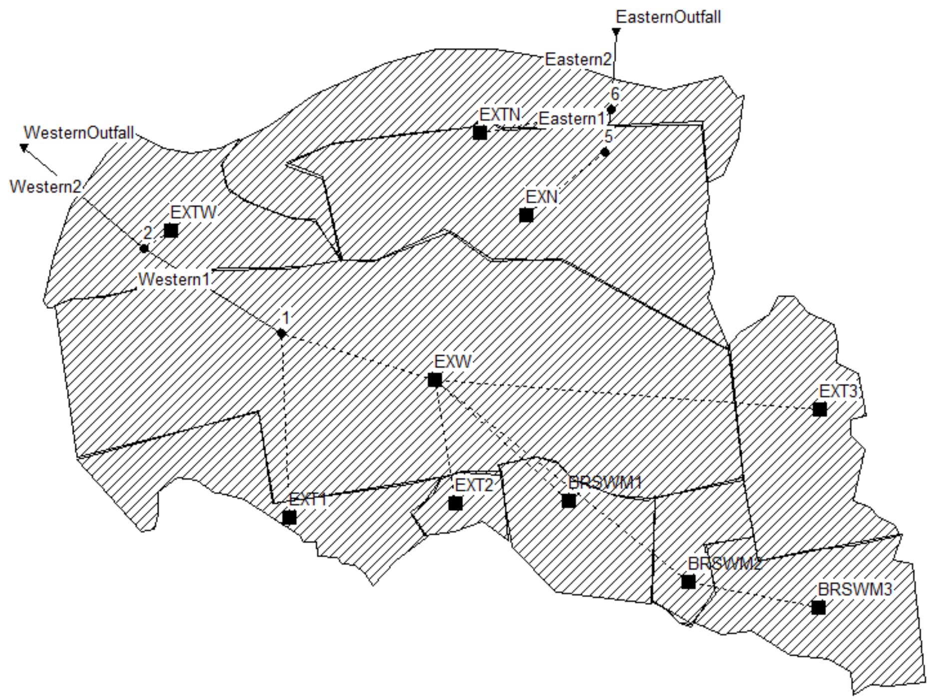


Figure 6.3 The EPA SWMM Model Schematic and Subcatchment Delineation

Imperviousness values and depression storage assumptions were added to the model using the values shown in **Table 6.2**. 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).

Table 6.2 Subcatchment Characteristics in the Study Area

SUBWATERSHED	SUBCATCHMENT	AREA (HA)	IMPERVIOUSNESS (%)	DEPRESSION STORAGE (MM) (PERVIOUS)	DEPRESSION STORAGE (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

** Location of proposed development*

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 2015, 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 6.3**).

Table 6.3 Infiltration Parameters for the Soils Encountered in the Study Area

SUBWATERSHED	SUCTION HEAD (MM)	CONDUCTIVITY (MM/HR)	INITIAL DEFICIT (FRACTION)
Western Creek	133.9	9.5	0.180
Eastern Creek	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 follows, in **Table 6.4**.

Table 6.4 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 6.4 shows how evapotranspiration and infiltration represent a major part of the annual water budget for the study area. For demonstration purposes, **Figure 6.4** shows how losses (infiltration and evapotranspiration) represented a major part of the water budget in the study area in 2013.

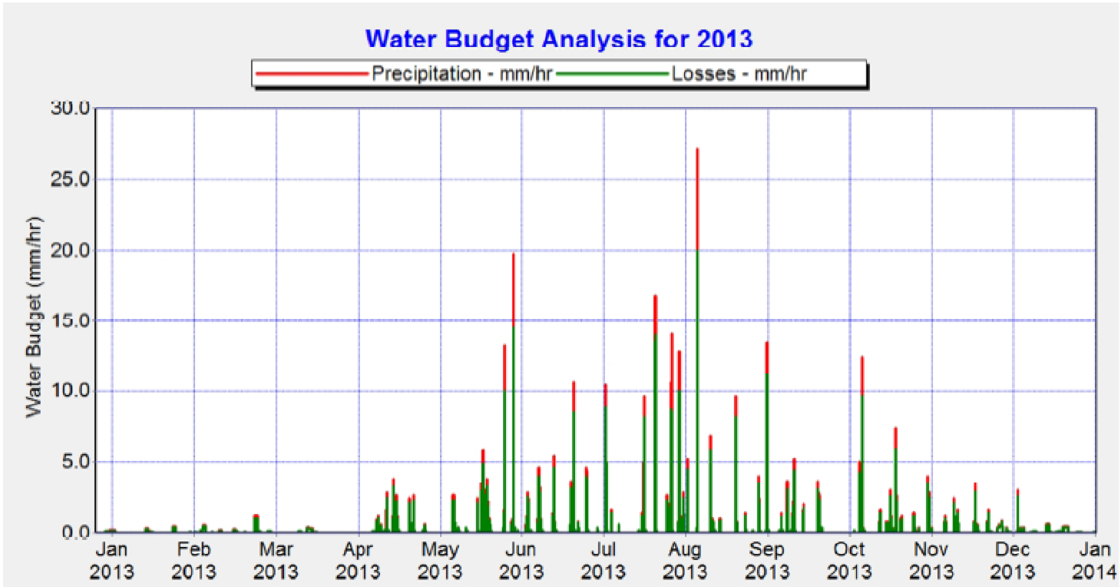


Figure 6.4 Water Budget Analysis Results for the months of 2013

Surface Runoff Regime at Outfalls

The results in **Table 6.5** show annual surface runoff volumes and Mean Annual Flows (MAF) for the two creeks at their respective outfalls (Western Outfall and Eastern Outfall; **Figure 6.3**). Observed baseflow values were added to **Table 6.5** to provide a context for the streamflow regime at the outfalls of both creeks.

Table 6.5 Surface Runoff Volumes and Rates at the Outfalls of Western and Eastern Creeks

SUBWATERSHED	RUNOFF VOLUME/YEAR (X 1000 M^3)	MEAN ANNUAL FLOW (CMS)	MAX ANNUAL FLOW (CMS)	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

Figures 6.5 and 6.6 show the surface runoff hydrographs for Western Creek and Eastern Creek outfalls, respectively.

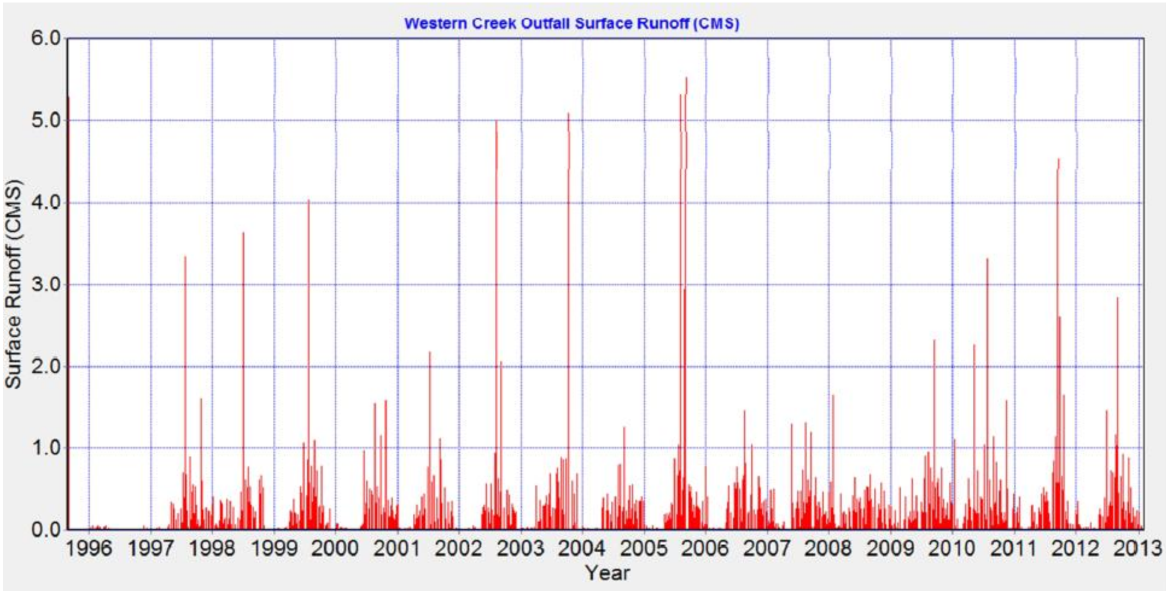


Figure 6.5 Surface Runoff Generated from Western Creek Subwatershed (cms) (1996-2013)

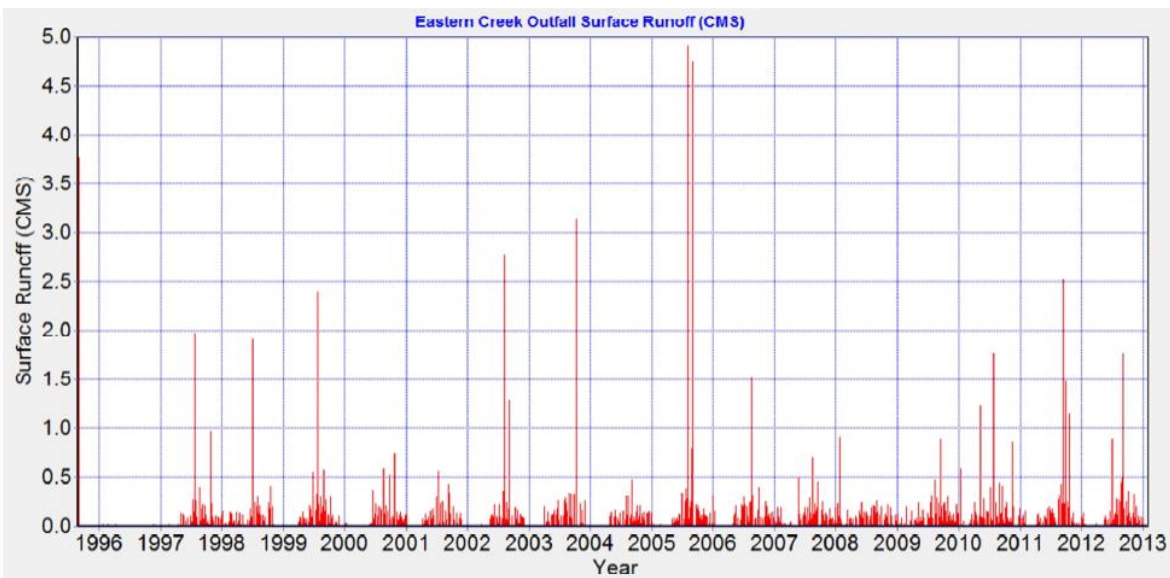


Figure 6.6 Surface Runoff Generated from Eastern Creek Subwatershed (cms) (1996-2013)

6.3.3.6 Storm Events

The following storm events have been simulated:

- 25 mm 4 hour Chicago storm with 12 minute time step (for water quality simulation of the SWM facilities and bankfull flow conditions of the creeks);
- 2, 5 and 100 year 24 hour SCS Type II storms with 12 minute time steps;

- 5 year 3 hour Chicago storm with 10 minute time step (for dual drainage evaluation, specifically minor system capture);
- 100 year 3 hour Chicago storm with 10 minute time step (for dual drainage evaluation, specifically major flow conveyance);
- 100 year 3 hour Chicago storm + 20% increase in intensity with 10 minute time step (applied as a stress test as per Technical Bulletin 2012-4);
- July 1, 1979 historical storm with 5 minute time step (per OSDG);
- August 1988 storm with 5 minute time step (per OSDG); and,
- August 1996 storm with 5 minute time step (per OSDG).

6.3.3.7 Level of Service

The level of service for future development is to be based on the latest City of Ottawa guidelines. The key recommendations of Technical Bulletin 2012-4 are summarized as follows:

- An inlet time of 10 minutes is to be used for all land uses and lot grading configurations. For the sizing of sewer segments (leads) connecting back yard catchbasins to the street sewer the inlet time is 15 minutes.
- In the absence of detail lot configuration and building size and position, the total imperviousness of external future urban areas is to be used for detailed design of infrastructure for the subject phase of development. The imperviousness ratio must be consistent with the runoff coefficient.
- Designs of sewer systems in Greenfield developments are not to be completed on the basis of unit ICD flow rates. ICD flow rates are to be calculated for each drainage area to ensure that the stormwater management (SWM) objectives are satisfied.
- The maximum flow depth on streets under either static or dynamic conditions shall be 300 mm. Flow depth of runoff on street rights-of-way is not to extend onto private property.
- In addition to the use of Historical storms, drainage systems are to be stress tested using design storms calculated on the basis of a 20 % increase of the City's IDF curves rainfall values. Modifications to the drainage system would be required if severe flooding of properties is identified.

6.4 System Concept

The proposed redevelopment of former CFB Rockcliffe will be comprised of low, medium and high density residential, commercial, institutional and park space. As part of the development, the existing combined sewer and storm sewer systems will be eliminated, with installation of new independent storm and sanitary services. The proposed storm system will be designed to provide capacity for the existing external flows that are currently conveyed through former CFB Rockcliffe, including outflows from the northeastern portion of the Montfort Hospital, the Burma Road SWM Facility, and surface flows from the external lands south and east of the site. Further, there is the long term potential for a new museum to be built northwest of the study area.

The conceptual stormwater management system incorporates standard urban drainage design and stormwater management features that can be summarized as follows:

- a dual drainage concept; and,

- two end-of-pipe stormwater management facilities.

The system concept is discussed in the subsequent sections and has been developed based on the reference documentation outlined in **Section 6.3.2**. As discussed in **Section 1.6**, LID techniques are being considered for the site. For the evaluation contained within this report, the stormwater analysis considers the above-noted standard urban drainage features. The LID features will be evaluated in a parallel process. It should be noted that the system presented in this report is designed with the flexibility to incorporate LID techniques.

It should be noted that as part of the redevelopment of the former CFB Rockcliffe, the Burma Road SWM Facility will be rehabilitated. The facility was designed in the early 1990s as an on-line facility providing water quantity control for the proposed adjacent residential development, as well as a portion of the NRC Campus via a 600 mm diameter CSP culvert just north of the Burma Road cul-de-sac. It is proposed to retrofit the facility to increase the available storage, which will aid in reducing storm sewer sizes in the study area. The outflow from the facility will connect to the new storm sewer system. A new culvert will be installed at the realigned Burma Road and the existing culvert will also be upgraded. Further discussions on Burma Road SWM Facility and culvert capacities are provided in **Section 6.5.4.2.1**.

6.4.1 Dual Drainage Design

Dual drainage design features a combination of on-site detention (surface ponding) with inlet control devices (ICDs) and direct conveyance with no ponding. It accommodates both minor (pipe) and major (surface) stormwater runoff. During frequent storms, the effective runoff collected by catchment areas is directly released via catchbasin inlets into the network of storm sewers, called the minor system. During less frequent storms, the balance of the flow (in excess of the minor flow) is accommodated by a system of rear yard swales and street segments called the major system. The main advantage of this arrangement is its ability to adjust the rate of total inflow into the minor system to satisfy the required level of service. The required total inflow is typically maintained by the restriction of the capacity and the density of the inlets directly connected into this system. As noted, during less frequent storms, the balance of the flow is accommodated by the major system. Typically, this accommodation is achieved by the attenuation on catchment surfaces called on-site detention and/or direct conveyance of the flow to a recipient. For former CFB Rockcliffe, no on-site storage on street segments has been applied to the site.

The dual drainage system was evaluated using the SWMHYMO hydrological model, discussed in **Section 6.5**.

6.4.1.1 Minor System

The minor system storm sewers will be sized based on the rational method, applying standards of both the City of Ottawa and MOE. Some of the key criteria proposed for this site include the following:

- Sewer Sizing: Rational Method
- Design Return Period: 1:5 year (local and collector streets)
  1:10 year (arterial roads)
- Initial Time of Concentration: 10 minutes
- Manning's: 0.013
- Velocity: Minimum 0.80 m/s
  Maximum 3.00 m/s

- Minimum Slope: Refer to below **Table 6.6**

Table 6.6 Minimum Pipe Slopes per OSDG

PIPE DIAMETER (MM)	SLOPE (%)
250	0.432
300	0.340
375	0.250
450	0.195
525	0.160
600	0.132
675	0.113
750 and larger	0.100

With respect to runoff coefficients, this report assumes the values presented in **Table 6.7**.

Table 6.7 Typical Runoff Coefficients

DEVELOPMENT	RESIDENTIAL DEVELOPMENT	RUNOFF COEFFICIENT, C	RUNOFF COEFFICIENT INCREASED BY 25%
Residential	Single Family	0.70	0.87
	Town Houses	0.70-0.80	0.87-1
	High Rise Apartment	0.80	1
Institutional, Commercial and Industrial		0.80	1
Park		0.20	0.25

Where possible, roads are designed to accommodate on-site storage. Inlet control devices (ICDs) will be utilized to control the surcharge in the minor system during infrequent storm events and maximize use of available on-site storage. ICDs will be sized at the detailed design stage.

The recommended minor storm sewer plan is presented on **Figure 6.7** and profiles of the main trunk sewers are presented on **Figure 6.8** (both enclosed in **Appendix E**). The associated storm sewer design spreadsheets are also enclosed in **Appendix E**, along with the corresponding drainage area plan (**Figure 6.9**). All minor storm sewer sizes will be reviewed and confirmed at the time of detailed design.

6.4.2 End-of-Pipe SWM Facilities

To address the water quantity and quality requirements of the redeveloped former CFB Rockcliffe site, it has been concluded that two end-of-pipe SWM facilities will be provided. The locations of the facilities are indicated on **Figure 1.3**. Minor and major flow from the majority of the study area will be conveyed to the facilities for treatment, prior to being released to the Ottawa River.

With respect to water quality, the facilities will be designed to provide an Enhanced Level of Protection. According to the MOE Stormwater Management Planning and Design Manual (March 2003), treatment volume is a function of drainage area, type of pond, urban imperviousness ratio, and Level of Protection. The Enhanced Level of Protection corresponds to end-of-pipe storage volumes required for the long-term average removal of 80% of total suspended solids. The storage requirements are based on the MOE Manual and those provided are summarized in the following **Table 6.8**. Supporting calculations are provided in **Appendix E**. It should be noted that in addition to the Rockcliffe development, the Eastern SWM Facility provides water quality treatment for the lands tributary to the Burma Road SWM Facility (a water quantity facility only) and for the future museum site. The Western SWM Facility provides water quality treatment for the portion of the Montfort Hospital site tributary to the Montfort SWM Facility (a water quantity facility only).

Table 6.8 Water Quality Volumes

SWM FACILITY	LEVEL OF PROTECTION	URBAN DRAINAGE AREA, TYPE OF FACILITY, % IMP	PERMANENT STORAGE (HA-M)		EXTENDED STORAGE (HA-M)		TOTAL STORAGE (HA-M)	
			Req.	Prov.	Req.	Prov.	Req.	Prov.
Eastern	Enhanced	160.58 Wet Pond 63%	2.72	3.59	0.64	1.81	3.36	5.40
Western	Enhanced	44.80 Wet Pond 35%	0.45	1.90	0.18	0.44	0.63	2.34

The proposed Eastern SWM Facility is comprised of 3.59 ha-m of permanent storage and 1.81 ha-m of extended storage based on a 25 mm storm event, exceeding the MOE requirements. The proposed Western SWM Facility is comprised of 1.90 ha-m of permanent storage and 0.44 ha-m of extended storage based on the 25 mm storm event, exceeding the MOE requirements. The area tributary to Eastern and Western SWM Facilities for water quality treatment are indicated on **Figure 6.10.** Relevant calculations are presented in **Appendix E**.

It should be noted that runoff from the Special Design Area (Block 44, refer to **Figure 1.3**) is to be provided with its own stormwater management (for example, potentially an oil-grit separator), outletting to the Eastern Creek. The conceptual location of the on-site water quality treatment system is indicated on **Figure 6.10**. The details of the preferred stormwater management approach for this area will be determined in the future during the Phase 3 design which is presently estimated to be in 2024. The use of LID elements will also be considered for this area.

Similar to the Special Design Area, runoff from the small area west of the Western SWM Facility (**Figure 6.10**) is to be provided with its own stormwater management system (for example, potentially a vortechs system) outletting to Western Creek. The conceptual location of the on-site water quality treatment system is shown in **Figure 6.10**. Details will be provided in the future during the Phase 2 Design.

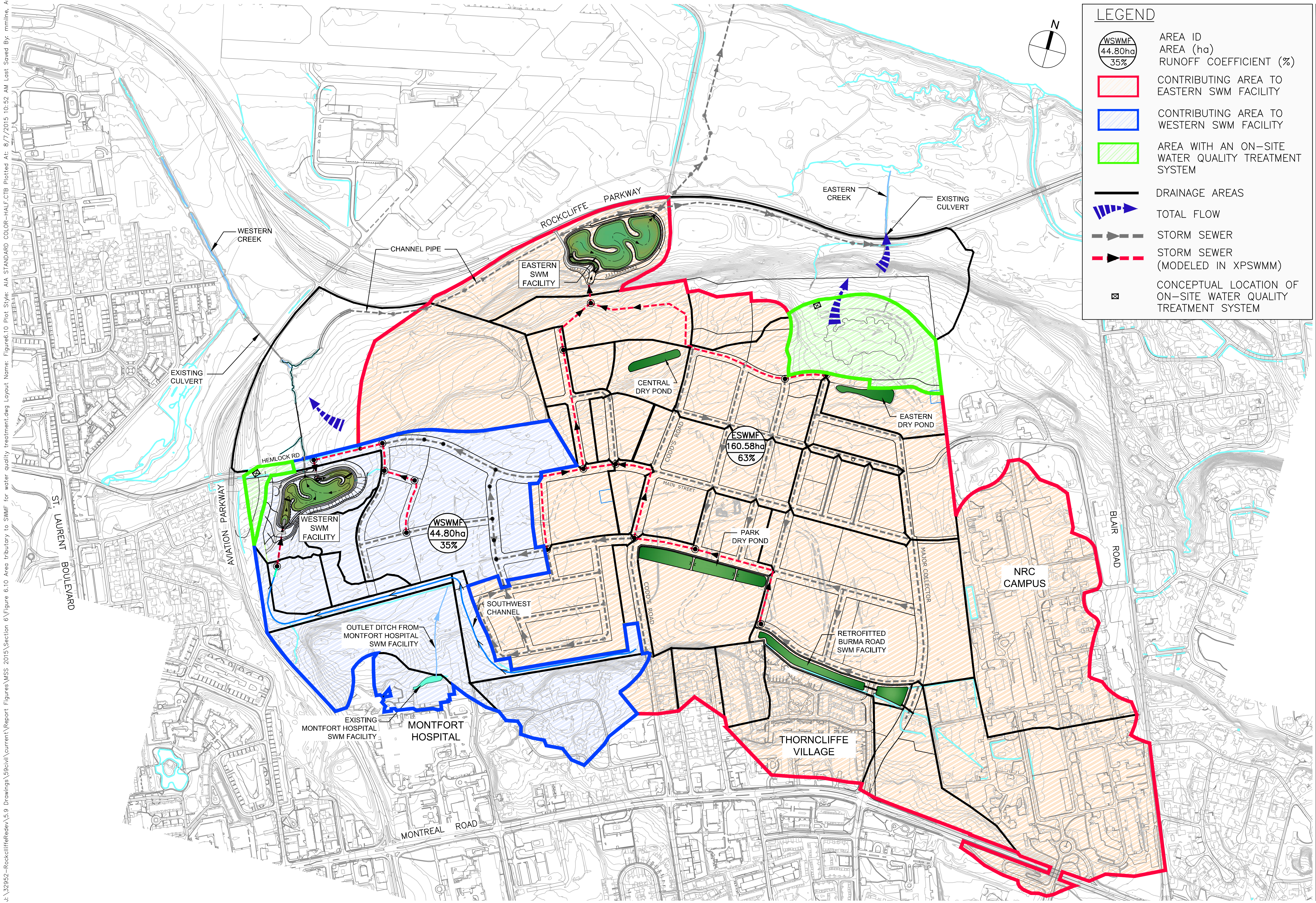
6.5 Hydrological Evaluation

A SWMHYMO model has been developed to represent former CFB Rockcliffe, as well as the external drainage areas. The SWMHYMO model technique offers single storm event flow generation and routing.

Simulations have been completed for both existing and post-development conditions, each discussed in the following sections.

6.5.1 Existing Conditions Evaluation

Land use, selected modeling routines, and input parameters applied to the existing conditions simulation are discussed below. A drainage area plan is provided on **Figure 2.17**. The corresponding SWMHYMO schematic is provided on **Figure 6.11** (enclosed in **Appendix** E). Model files are also located in **Appendix E**.



[B]

Scale

Project Title
FORMER CFB ROCKCLIFFE
MASTER SERVICING STUDY

Drawing Title
AREA TRIBUTARY TO
SWM FACILITIES
FOR WATER QUALITY TREATMENT

Sheet No.
FIGURE 6.10

6.5.1.1 Land Use

An aerial photograph of the study area is presented in **Figure 1.1**. The official closure of former CFB Rockcliffe was in 2009, and since that time all of the buildings and houses on the site have been demolished. The road network and residential driveways are still in place. Otherwise the majority of the site has grass cover, with individual trees and some tree groupings.

As noted in **Section 1.3**, areas external to the study area are being considered. Specifically related to existing stormwater conditions, the following are relevant: East of the study area, the NRC Campus is a mix of low-density low- and medium-rise buildings. The existing Thorncliffe Village, Foxview and Fairhaven residential developments are located south of the study area. Thorncliffe and Foxview are considered medium density, while Fairhaven is a low-density rural estates development. The Montfort Hospital campus, situated south of the study area, is comprised of medium-rise buildings and surface parking, north of which are the Montfort Hospital Woods, considered an Urban Natural Area. The NCC lands located between the study area and the Rockcliffe Parkway are undeveloped, with a combination of grass and shrub and tree cover.

6.5.1.2 Drainage Area Parameters

The main hydrology parameters are summarized below and in **Table 6.10** and **Table 6.11**.

Area: Catchment areas are based on review of the topography of the site and external areas. Please refer to **Figure 2.17** for delineation of the drainage boundaries.

Infiltration: A composite CN value was applied to the developed portion of former CFB Rockcliffe. 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 in the following **Table 6.9** for reference.

Table 6.9 Composite CN Value

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
EXE	36.34	9.09	99	27.25	76	82

Initial Abstraction (Depression Storage): Depression storage depths of 0.8 mm and 1.5 mm were used for impervious and pervious areas, respectively. These values are more conservative than those in the OSDG.

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 24 hour SCS Type II design storm events were used in the evaluation of the site.

Table 6.10 Existing Conditions Hydrological Parameters – Study Area

AREA ID	AREA (HA)	CN	TIME TO PEAK (H)
EXW	86.06	83	0.80
EXE	36.34	82	0.13

Table 6.11 Existing Conditions Hydrological Parameters – External Areas

AREA ID	DESCRIPTION	AREA (HA)	CN	IMP (%)		LENGTH, LGI (M)	TIME TO PEAK (H)
				TOTAL	DIRECTLY		
EXT1	Residential	2.95	N/A	61	61	140	N/A
EXT2	Open space, rural estate	6.18	80	N/A	N/A	N/A	0.16
EXT3	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 Campus	18.39	N/A	71	71	450	N/A
EXTW	Vacant	17.52	72	N/A	N/A	N/A	0.22
EXTE	Vacant	24.68	71	N/A	N/A	N/A	0.41
EXM	Montfort Hospital	2.02	N/A	31	31	114	N/A

6.5.2 Existing Conditions Results

Results of the SWMHYMO computer simulation are summarized in the below **Table 6.12**.

Table 6.12 Summary of Existing Conditions Results

DESIGN STORM	PEAK FLOW TO EASTERN OUTLET (CMS)	PEAK FLOW TO WESTERN OUTLET (CMS)
2 year 24 hour SCS Type II	2.60	3.71
5 year 24 hour SCS Type II	3.89	5.51
100 year 24 hour SCS Type II	8.06	11.13

The above results and **Section 6.3.3.1** indicate that both culverts will theoretically overflow under existing conditions.

6.5.3 Post-development Conditions Evaluation

Land use, selected modeling routines, and input parameters applied to the post-development conditions simulation are discussed below. A drainage area plan is provided on **Figure 6.13** (indicating minor system network) and on **Figure 6.14** (indicating major flow routing). The conceptual end-of-pipe SWM facilities have been included in the SWMHYMO simulation. The corresponding SWMHYMO schematic is provided on **Figure 6.12** (enclosed in **Appendix E**).

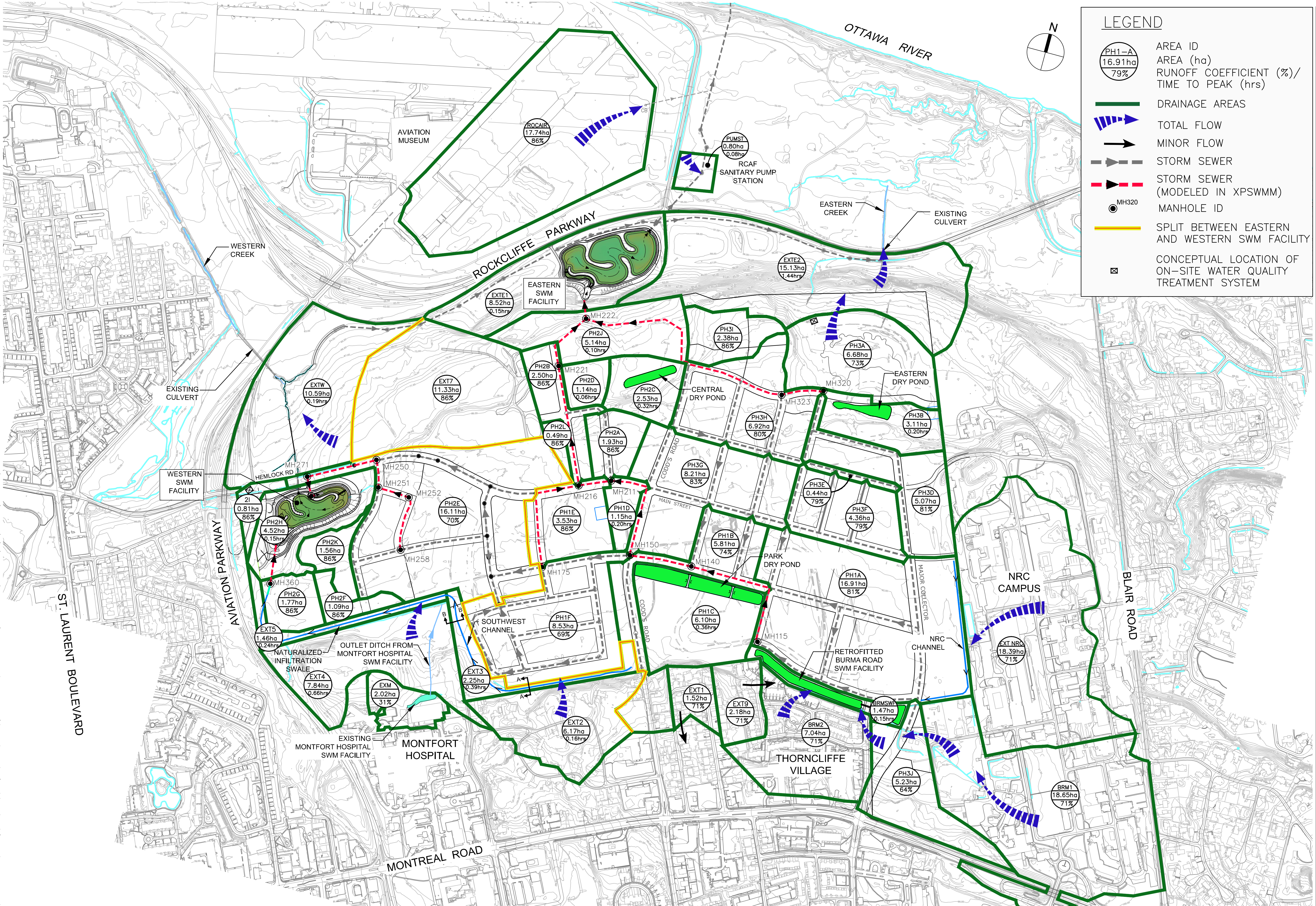
6.5.3.1 Land Use

The study area will be developed as a mixed-use community featuring residential (low- and mid-rise), retail, mixed-use, and employment land use, as well as school sites, parks, and natural areas. Further, there is the long term long term potential for a new museum to be built northwest of the study area.

6.5.3.2 Drainage Area Parameters

The main hydrology parameters are summarized below and in **Table 6.13** and **Table 6.14**.

Area: The various phases of development have been divided into semi-lumped areas based on the minor and major system networks. With respect to external rural areas, the delineation is based on servicing and topography. Refer to **Figures 6.13** and **6.14**.



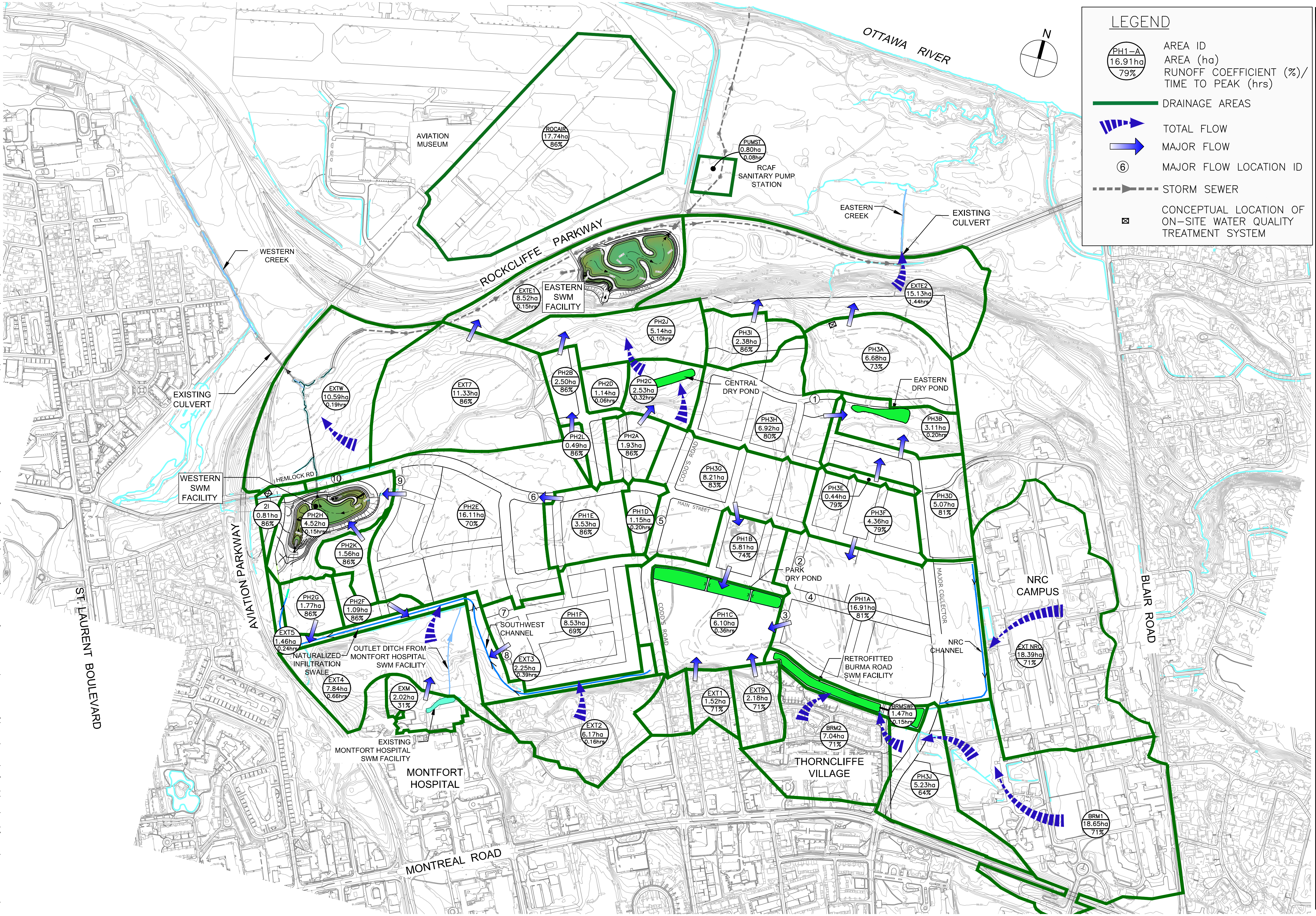
[B]

Scale

Project Title
FORMER CFB ROCKCLIFFE
MASTER SERVICING STUDY

Drawing Title
POST-DEVELOPMENT CONDITIONS
DRAINAGE AREA PLAN
MINOR SYSTEM CONNECTIVITY

Sheet No.
FIGURE 6.13



[B]

Scale

Project Title
FORMER CFB ROCKCLIFFE
MASTER SERVICING STUDY

Drawing Title
POST-DEVELOPMENT CONDITIONS
DRAINAGE AREA PLAN
MAJOR SYSTEM CONNECTIVITY

Sheet No.
FIGURE 6.14

Imperviousness: In general, total and directly connected imperviousness ratios based on typical runoff coefficients have been applied. The runoff coefficients have been used in similar, recent developments in Ottawa.

Time to Peak: Time to peak for the rural areas was established using the Uplands Method, which accounts for elevation, distance and land use within the drainage area to estimate velocity of runoff. The time of concentration (Tc) is then calculated based on velocity, from which time to peak is determined applying the formula 0.6Tc (based on the SWMHYMO User's Manual). It is the time to peak that is applied in the model.

Minor system capture:

- **Internal** The minor system capture applied across the study area is based on the 5 year simulated flow, specifically the 5 year 3 hour Chicago storm event. Major system analysis indicated that the inflow rates at two drainage areas (PH1A and PH3G, refer to **Figure 6.13**) required increasing to confine the maximum ponding in street segments to 0.3 m. Minor system inflow rates are summarized in **Table 6.13**.

  For those catchment areas where total flow is captured, flow values are representative of the 100 year 3 hour Chicago storm event.

- **External**
  - Total flow from the NRC Campus (Area EXTNRC, refer to **Figure 6.13**) will be conveyed to the Burma Road SWM Facility through a temparary swale as part of Phase 1A. The proposed swale will be constructed on CLC property.
  - Total flow from the majority of Thorncliffe Village (Area BRM2, refer to **Figure 6.13**) and minor flow from the remainder of the development (Area EXT9, refer to **Figure 6.13**) is conveyed to the facility via the existing storm sewer. The minor system inflow rate for Area EXT9 is based on the 5 year 3 hour Chicago storm event.
  - As noted in **Section 2.4.3**, major system flow from the Foxview community (Area EXT1, refer to **Figure 6.13**) is tributary to the Rockcliffe development. The minor system inflow rate for Area EXT1 is based on the 5 year 3 hour Chicago storm event.
  - Total flow from the Fairhaven community (Area EXT2, refer to **Figure 6.13**) is directed to the proposed Southwest Channel as part of Phase 1A.
  - Outflow from the existing Montfort Hospital SWM Facility, major flow from the northeastern portion of the hospital site (Area EXM, refer to **Figure 6.13**), and total flow from the Montfort Hospital Woods (Area EXT4, refer to **Figure 6.13**) is directed to the proposed Southwest Channel as part of Phase 2.
  - Total flow from part of the Rockcliffe Airport/Aviation Museum Area (Area ROCAIR, refer to **Figure 6.13**) and total tributary flow from the small area close to the Pump Station, north of the Eastern SWM facility (Area PUMST, refer to **Figure 6.13**) is proposed to be conveyed to the Ottawa River via a new outlet storm sewer as part of the Phase 1A.
  - Water quality treatment of runoff from the future museum site (Area EXT7, refer to **Figure 6.13**) has been accounted for in the Eastern SWM Facility. The proposed pipe servicing the site is discussed in **Section 6.5.4.3**. Minor system inflow rates are summarized in **Table 6.13**. For those catchment areas where total flow is captured, flow values are representative of the 100 year 3 hour Chicago storm event.

Surface storage and major flow routing:

- As noted in **Section 6.4.1**, no on-site storage on street segments was applied.
- Major flow from the western part of the site is directed to the Western SWM Facility (refer to **Figure 6.14**).
- Several major flow storage and routing features are provided across the site and are summarized below. The features aid in reducing surface flow to meet City of Ottawa criteria and reduce pipe sizes within the development.
    - The retrofitted Burma Road SWM Facility, a dry pond, provides water quantity control for approximately 50 ha of development. Total tributary flow from the NRC Campus east of Burma Road will be conveyed via the new culvert at the realigned Burma Road to the facility. Runoff from the realigned Burma Road north of Montreal Road will also be conveyed to the facility. Total flow from the majority of Thorncliffe Village (Area BRM2) and minor flow from the remainder of the development (Area EXT9) will be conveyed to the facility via the existing storm sewer.
    - Three dry ponds are proposed across the site. Major flow from portions of the study area will be directed to the dry ponds for attenuation prior to release. The potential infiltration component of these ponds will be evaluated as part of the parallel LID work; as part of the MSS, however, the storage volume designated for both of the ponds is for major system storage only. The functional design for the major system storage provides flexibility in terms of the potential future use for LID techniques.

      A dry pond is proposed in the park block identified as Block 38 in the concept plan (refer to **Figure 1.3**). Two other dry ponds are proposed, referred to as the eastern and central dry ponds.
    - There is a channel proposed around the south and west of the study area, henceforth referred to as the Southwest Channel. It will extend west from Codd's Road, between the redevelopment and the Montfort Hospital Woods, from where it will extend north prior to releasing to the Western SWM Facility (refer to **Figure 6.14**). Major flow runoff from a portion of the study area will be directed to the channel, as will runoff from Fairhaven, the Montfort Hospital Woods, outflow from the Montfort Hospital SWM Facility, and major flow from the northeastern portion of the hospital site. The infiltration potential for this channel will be evaluated as part of the parallel LID work; as part of the MSS, however, the storage volume designated for the pond is for major system storage only. The functional design for the major system storage provides flexibility in terms of the potential future use for LID techniques. Development setbacks are not recommended for this drainage feature.
    - NRC Runoff presently drains westward across the subject site especially over the employment block 56 (refer to **Figure 1.3**). A swale is proposed along the eastern and southern edge of block 56 to capture and convey NRC surface runoff and direct it to the improved Burma Pond during Phase 2 construction (refer to **Figure 6.14**). The existing ditch will be modified if and as needed to accommodate the NRC runoff.
- Conceptual major flow routing is presented on **Figure 6.14**. The Macro Grading Plan is provided on **Figure 6.15** which is included in **Appendix E**.

Infiltration: Infiltration losses were selected to be consistent with OSDG. Specifically, Horton's values applied are f_0 = 76.2 mm, f_c = 13.2 mm/h and k = 0.00115 s^{-1}. For simulation of undeveloped or low density areas, the SCS methodology was applied. CN values ranging between 71 and 80 were applied, based on available soils information.

Length: The impervious length applied is based on an average measured length through the catchment and the calculated length based on the SWMHYMO User's Manual. The pervious length is based on average lot depth. This approach is consistent with OSDG.

Initial Abstraction (Depression Storage): Depression storage depths of 0.8 mm and 1.5 mm were used for impervious and pervious areas, respectively. These values are more conservative than those in the OSDG.

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

Slope: A slope of 0.5% was used for impervious surfaces and a slope of 2% was used for pervious areas (lot grading).

Table 6.13 Post-Development Conditions Hydrological Parameters – Study Area

OUTLET	AREA ID	AREA (HA)	IMP (%)		TIME TO PEAK (H)	CN	LGI (M)	MINOR SYSTEM CAPTURE	
			TOTAL	DIRECTLY				UNIT (L/S/HA)	TOTAL (L/S)
Eastern SWM Facility	PH3B	3.11	N/A	N/A	0.20	76	N/A	N/A	364*
	PH3D	5.07	81	81	N/A	N/A	262	194	985
	PH3E	0.44	79	79	N/A	N/A	207	198	87
	PH3F	4.36	79	79	N/A	N/A	235	197	858
	PH3G	8.21	83	83	N/A	N/A	279	196	1610
	PH3H	6.92	80	80	N/A	N/A	278	190	1317
	PH3I	2.38	86	86	N/A	N/A	203	216	515
	PH3J	5.23	64	64	N/A	N/A	243	N/A	1778*
	PH2A	1.93	86	86	N/A	N/A	139	232	447
	PH2B	2.50	86	86	N/A	N/A	238	211	527
	PH2C	2.53	N/A	N/A	0.32	76	N/A	N/A	221*
	PH2D	1.14	N/A	N/A	0.06	76	N/A	N/A	246*
	PH2J	5.14	N/A	N/A	0.1	76	N/A	N/A	882*
	PH2L	0.49	86	86	N/A	N/A	49	253	124
	PH1A	16.91	81	81	N/A	N/A	618	156	2645
	PH1B	5.81	74	74	N/A	N/A	293	177	1031
	PH1C	6.10	N/A	N/A	0.36	76	N/A	N/A	493*
	PH1D	1.15	N/A	N/A	0.2	76	N/A	N/A	135*
	PH1E	3.53	86	86	N/A	N/A	216	214	756
	PH1F	8.53	69	69	N/A	N/A	257	172	1469
Eastern Creek via Rockcliffe Parkway Culvert	PH3A	6.68	73	73	N/A	N/A	247	152	1017
Western SWM Facility	PH2E	16.10	70	70	N/A	N/A	361	160	2583
	PH2F	1.09	86	86	N/A	N/A	99	243	265
	PH2G	1.77	86	86	N/A	N/A	124	234	415
	PH2H (Pond Block)	4.52	N/A	N/A	0.15	95	N/A	N/A	1240*
	PH2K	1.57	81	81	N/A	N/A	176	224	351
Western Creek via Aviation Parkway Culvert	PH2I	0.81	86	86	N/A	N/A	132	N/A	352*

** Total flow capture during the 100 year 3 hour Chicago storm event.*

Table 6.14 Post-Development Conditions Hydrological Parameters – External Areas

OUTLET	LOCATION, AREA ID		AREA (HA)	IMP (%)		TIME TO PEAK (H)	CN	LGI	MINOR CAPTURE	SYSTEM
				TOTAL	DIRECTLY				UNIT (L/S/HA)	TOTAL (L/S)
Eastern SWM Facility	Undeveloped, includes Eastern SWM Facility	EXT E1	8.82	N/A	N/A	0.15	95	N/A	N/A	2338**
	Foxview	EXT1	1.52	71	71	N/A	N/A	110	N/A	*Major flow only*
	Museum	EXT7	11.33	86	86	N/A	N/A	342	194	2200
Eastern SWM Facility via Burma*	NRC Campus	EXT-NRC	18.39	71	71	N/A	N/A	450	N/A	5703†
	Thorncliffe Village	BRM1	18.65	71	71	N/A	N/A	541	N/A	5618†
		BRM2	7.04	71	71	N/A	N/A	N/A	N/A	2532†
		BRM SWF	1.47	N/A	N/A	0.15	95	N/A	N/A	374**
		EXT9	7.04	71	71	N/A	N/A	125	62	435
Eastern Creek via Rockcliffe Parkway Culvert	Undeveloped	EXT E2	15.13	N/A	N/A	1.44	71	N/A	N/A	614**
Ottawa River via a New Storm Sewer	Airport/ Museum	ROC AIR	17.74	86	86	N/A	N/A	660	N/A	5.55†
	Undeveloped	PUM ST	0.8	N/A	N/A	0.08	71	N/A	N/A	0.13†
Western SWM Facility via Southwest Channel	Fairhaven	EXT2	6.17	N/A	N/A	0.16	80	N/A	N/A	934†
		EXT3	2.25	N/A	N/A	0.39	80	N/A	N/A	196†
	Southwest Channel, Montfort Hospital and Woods	EXT4	7.84	N/A	N/A	0.66	80	N/A	N/A	470†
		EXT5	1.46	N/A	N/A	0.24	80	N/A	N/A	175†
		EXM	2.02	31	31	N/A	N/A	114	N/A	569**€
Western Creek via Aviation Parkway Culvert	Undeveloped	EXT W	10.59	N/A	N/A	0.19	72	N/A	N/A	1120†

** Water quantity control only is provided by Burma Road SWM Facility; water quality control is provided in the Eastern SWM Facility*
*** Total flow generated by catchment during the 100 year 3 hour Chicago event*
† Total flow capture during the 100 year 3 hour Chicago event
€ Water quantity control only is provided by Montfort Hospital SWM Facility; water quality control is provided in the Western SWM Facility

6.5.4 Post-Development Conditions Results

6.5.4.1 Major Flow Routing on Street Segments

The maximum overland flow on streets has been reviewed at critical downstream locations and is summarized in **Table 6.15**

Table 6.15. The overland flow was evaluated based on proposed grades. Using the channel routing routine in SWMHYMO, maximum normal depth and velocity of flow have been quantified and results are presented below for the 100 year 3 hour Chicago storm event, which is

commonly used to evaluate the urban component of dual drainage. Major flow outlets locations at which flow was evaluated are indicated on **Figure 6.14**. Flow location IDs are also summarized in **Table 6.15**. Model files and supporting calculations are enclosed in **Appendix E**.

It should be noted that non-standard cross-sections are proposed for major roads across the site, and were accounted for in the major system analysis. However, the analysis considered two exceptions where street sawtoothing is expected (identified as flow locations 3 and 5). For these two locations, standard road cross-sections were applied to simplify the analysis.

Table 6.15 Summary of Major Flow at Critical Downstream Street Segments (SWMHYMO file 32952VXD.out)

FLOW LOCATION	STREET SEGMENT LOCATION: *DRAINAGE AREA ID (MH LOCATION, REFER TO FIGURE 6.7*, *APPENDIX E)*	MAX. CUMULATIVE FLOW (CMS)	DEPTH (M)	VELOCITY (M/S)	D X V (M^2/S)
100 year 3 hour Chicago Storm					
1	PH3H (MH 323 to 320)	1.24	0.15	1.75	0.26
2	PH1A (MH 163 to 130)	0.81	0.15	1.06	0.16
3	PH1A (MH 130 to 115)	1.29	0.23	1.29	0.30
4	PH1A (MH 125 to 130)	1.30	0.20	1.07	0.21
5	PH3G (MH 155 to 150)	0.50	0.17	0.73	0.12
6	PH2E (MH 230 to 235)	0.67	0.15	0.88	0.13
7	PH1F (MH 183 to 185)	0.89	0.13	1.59	0.21
8	PH1F (MH 190 to 185)	0.60	0.12	1.25	0.15
9	PH2E (MH 251 to 250)	2.29	0.20	1.63	0.33
10	PH2E (MH 250 to 271)	3.95	0.23	2.46	0.56

At all locations, the depth of ponding is less than the City guideline of 0.3 m. Further, the depth by velocity does not exceed the City guideline of 0.6 m^2/s.

6.5.4.2 Surface Storage and Major Flow Routing Features

As noted in **Section 6.5.3**, major flow from across the site is routed to various major system features. Each feature is listed in the below **Table 6.16**, with a summary of total flow contributing to the feature during the 100 year 3 hour Chicago storm event. The features are discussed in greater detail below.

Table 6.16 Summary of Flow to Major Flow Features (SWMHYMO file 32952PD.out)

MAJOR SYSTEM OUTLET	FLOW (CMS)	CONTRIBUTING DRAINAGE AREAS (LOCATION, AREA ID)	
Burma Road SWM Facility	14.9	NRC Campus	EXT-NRC
		Thorncliffe Village	BRM1
			BRMSWF
			BRM2
		Study Area	PH3J
Park Dry Pond	4.2	Thorncliffe Village	EXT9
			EXT1
		Study Area	PH3F
			PH1A
			PH3G
			PH1B
			PH1C
Eastern Dry Pond	2.5	Study Area	PH3D
			PH3E
			PH3B
			PH3H
Central Dry Pond	0.5	Study Area	PH2A
			PH2C

MAJOR SYSTEM OUTLET	FLOW (CMS)	CONTRIBUTING DRAINAGE AREAS (LOCATION, AREA ID)	
Southwest Channel	1.9	Fairhaven	EXT2
			EXT3
			EXT4
		Montfort	EXT5
			EXM
			PH1F
			PH2F
			PH2G

6.5.4.2.1 Retrofitted Burma Road SWM Facility

The Burma Road SWM Facility is located at the northern boundary of Thorncliffe Village, west of Burma Road. It is recommended that the existing facility be expanded to provide water quantity control for approximately 50 ha of development to reduce pipe sizes in the study area. The retrofitted facility will remain a dry pond. The retrofit will involve a new culvert crossing the realigned Burma Road, the addition of an upstream cell, upgrading the existing culvert crossing just north of the existing Burma Road cul-de-sac, expansion of the existing linear facility, and a new outlet structure.

Total tributary flow from the NRC Campus east of Burma Road will be conveyed via the new culvert at the realigned Burma Road to the facility. Runoff from the realigned Burma Road north of Montreal Road will be conveyed to the facility. Total flow from the majority of Thorncliffe Village (Area BRM2) and minor flow from the remainder of the development (Area EXT9) will be conveyed to the facility via existing storm sewers. There are no proposed changes to the minor or major connectivity from Thorncliffe Village to the SWM facility.

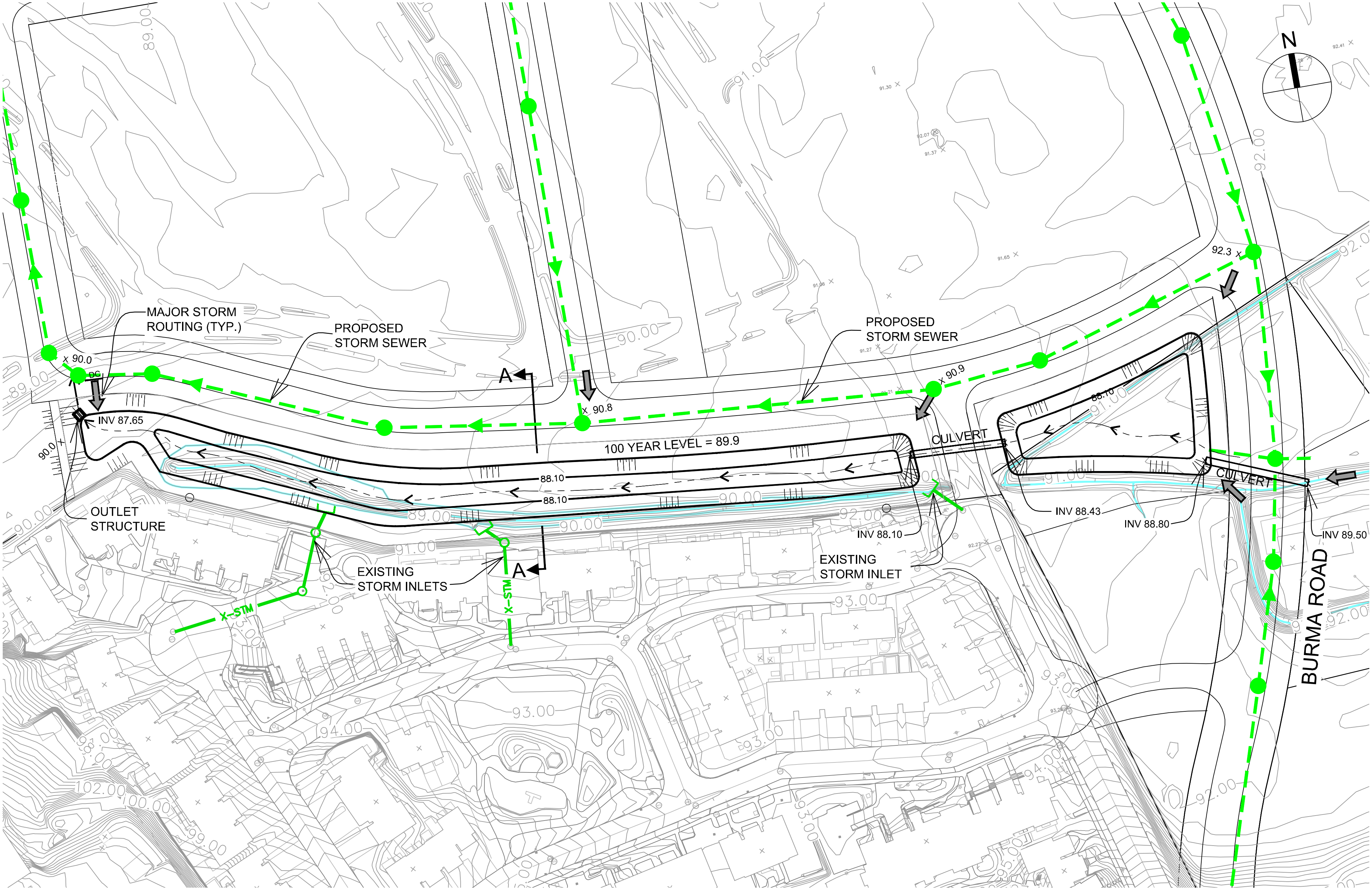
The conceptual plan and cross-section of the Burma Road SWM Facility are presented on **Figures 6.17** and **6.18**. As noted above, it is proposed that a new culvert be installed across the realigned Burma Road to convey flow from the east to the upstream cell of the facility. The new culvert is proposed to be a 900 mm x 3000 mm rectangular box with 2% longitudinal slope. The invert of the culvert is proposed to be set at 89.50m, corresponding to the bottom of the existing ditch upstream of the culvert. The existing culvert crossing just north of the existing Burma Road cul-de-sac will be upgraded to convey flow to the downstream cell of the facility. The upgraded culvert is proposed to be a 1500 mm x 3600 mm box with 1% longitudinal slope. The invert of the culvert is proposed to be set at 88.43m (refer to **Figure 6.17** and **6.18**).

Each cell is proposed to be provided with a low flow channel at approximately 0.15% longitudinal slope. Above the elevation of the low flow channel, the cell can be designed with a 1% cross-slope. The side slopes of the facility are proposed to be 3H:1V, tying into existing ground on the south and the proposed development to the north. The outlet control structure, located at the western-most point of the facility, will convey flow via a new 30 m – 1500 mm diameter storm sewer to the future storm sewer to be located in the adjacent street.

The elevation of the outlet storm sewer corresponds to the invert of the facility at the downstream end (87.65 m), resulting in there being no permanent pool in the facility. The outflow from the facility is restricted to 8.15 cms. During the 100 year storm event, the water level will reach 89.90 m, matching the 100 year water level of the existing facility and to an average depth of 1.80 m. The total expected volume to be utilized is 1.00 ha-m.

6.5.4.2.2 Dry Ponds

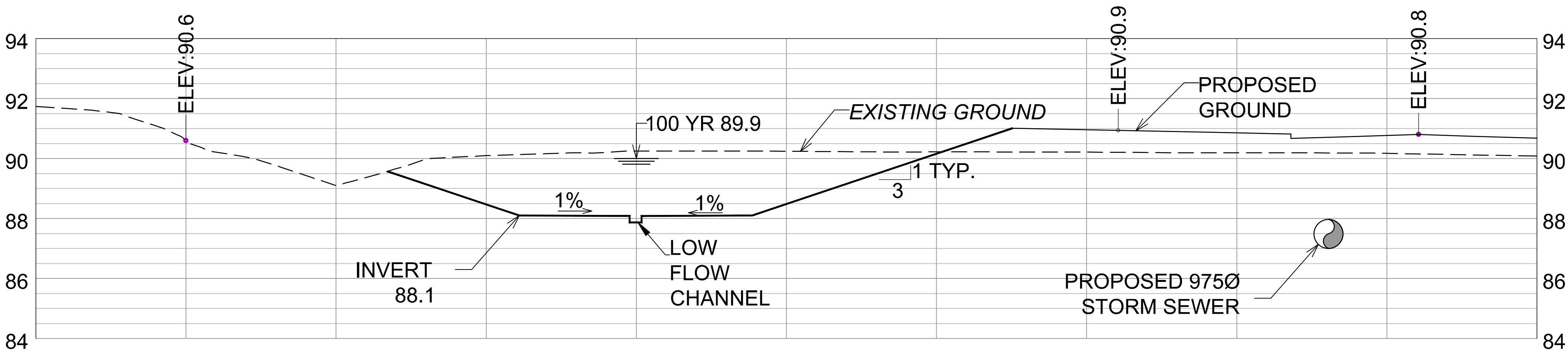
As noted in **Section 6.5.3.2**, three dry ponds are proposed across the study area. Major flow from across a portion of the study area will be routed to one of those three ponds for attenuation prior to release. As previously noted, the infiltration component of these ponds will be evaluated as part of the parallel LID work; as part of the MSS, however, the storage volume designated for



J:\32952-RockcliffeRedev\5.9 Drawings\59civil\current\Report Figures\MSS 2015\Section 6\Figure 6.17 Burma Road Pond.dwg Layout Name: Figure 6.17

IBI

Scale	Project Title	Drawing Title	Sheet No.
NTS	FORMER CFB ROCKCLIFFE MASTER SERVICING STUDY	RETROFITTED BURMA ROAD SWM FACILITY	FIGURE 6.17



IBI

Scale	Project Title	Drawing Title	Sheet No.
NTS	FORMER CFB ROCKCLIFFE MASTER SERVICING STUDY	RETROFITTED BURMA ROAD SWM FACILITY SECTION A-A	FIGURE 6.18

three ponds is for major system storage only. The functional design provides flexibility in terms of the potential future use for LID techniques.

One dry pond is proposed in the park Block 38 (refer to **Figure 1.3**). Total flow from Foxview (Area EXT1), major flow from a portion of Thorncliffe Village (Area EXT9), and major flow from the southeast of the study area will be conveyed overland to the park dry pond for attenuation, prior to being released to the minor system. The areas contributing major flow to the park dry pond are indicated on **Figure 6.14**.

The conceptual plan and cross-section of the park dry pond are presented on **Figures 6.19** and **6.20**. The facility is divided into three cells, separated by berms which will be fitted with culverts designed to maintain a maximum water depth in each cell of about 0.60 m. The berms have been incorporated into the design to allow for pedestrian access to the rest of the park south of the dry pond. The three cells are designed with flat bottoms, ranging from 88.5 m at the upstream, 87.7 m at the intermediate cell, and 87.5 m at the downstream cell. This will provide flexibility at the detailed design stage, should the dry pond also be used for infiltration purposes. Otherwise, a longitudinal slope can be introduced across each of the cells. Details on how the conveying system functions, such as the connecting culverts capacity and relationship of water levels in these three cells will be provided during the detailed design stage.

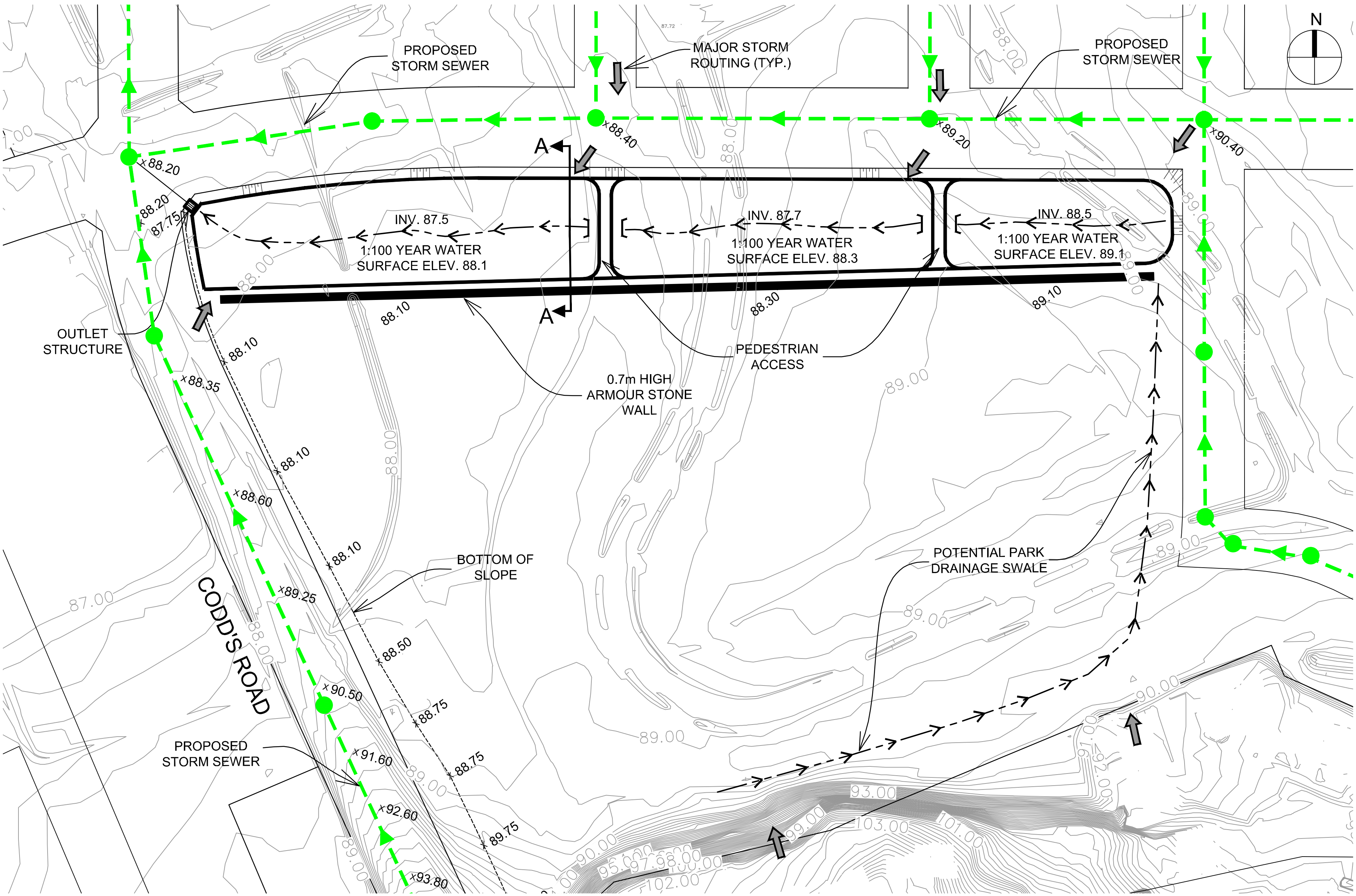
On the north, the side slopes of the facility are proposed at 3H:1V. On the south, a 0.7 m armour stone wall is proposed. As with any proposed wall terracing can also be substituted. The outlet control structure, located at the western-most point of the facility, will convey flow to the storm sewer located in the adjacent street. The elevation of the outlet control structure corresponds to the invert of the facility (87.5 m), resulting in there being no permanent pool in the facility. Outflow from the facility is restricted to 160 l/s. During the 100 year storm event, it is anticipated the average depth of ponding will be 0.6 m. The total volume utilized is 3,000 cu-m.

In addition to the park dry pond on Block 38, two additional dry ponds are proposed, and there are referred to as the eastern and central dry ponds (indicated on **Figure 6.14**).

The eastern dry pond is located in the northeast of the site. The bottom of the pond is proposed to be flat, with an invert of about 85.5 m. The side slopes of the pond are proposed to be 3H:1V. The outlet control structure, located at the western-most point of the pond, could convey flow to the storm sewer located in the adjacent street. The elevation of the outlet control structure is suggested to be set at 85.3 m, slightly below the invert of the pond, resulting in there being no permanent pool in the pond. Outflow from the facility is restricted to 147 l/s. During the 100 year storm event, it is estimated the water level will reach 86.1 m, corresponding to an average depth of 0.6 m. The total expected volume to be utilized is 1610 cu-m. The conceptual plan and cross-section of the pond are presented on **Figures 6.21** and **6.22**.

The central dry pond is proposed to be located in the future part Block 26 (refer to **Figure 1.3**). The bottom of the pond is proposed to be flat, with an invert of 82.3 m. The side slopes of the pond are proposed to be 3H:1V. Outflow from the pond will be conveyed from the western-most point of the pond by either pipe or swale. The outflow will be conveyed to the Eastern SWM Facility. The elevation of the outlet corresponds to the invert of the pond (82.3 m), resulting in there being no permanent pool in the pond. Outflow from the pond is restricted to 98 l/s. During the 100 year storm event, it is estimated the water level will reach 82.5 m, corresponding to an average depth of 0.3 m. The expected total volume to be utilized is 560 cu-m. The conceptual plan and cross-section of the dry pond are presented on **Figures 6.23** and **6.24**.

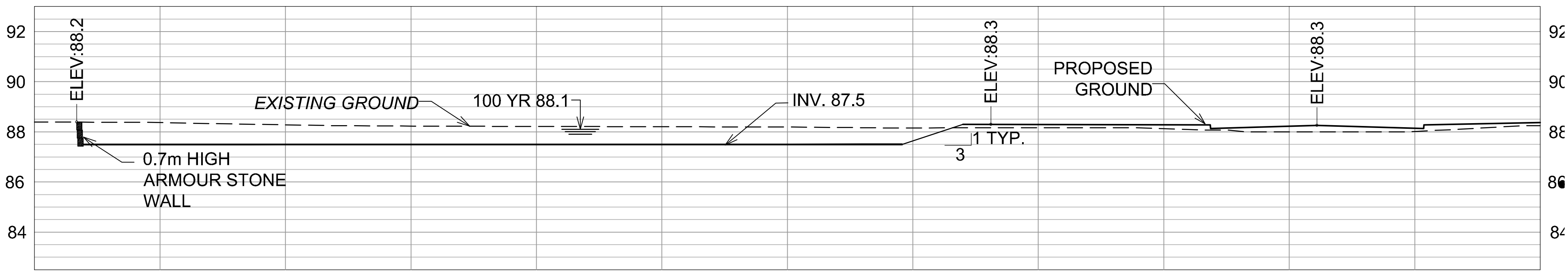
It should be noted that the performance of the proposed three dry pond features will be evaluated after these facilities have been in service. These features are the subject of the parallel report “Former CFB Rockcliffe LID Stormwater (SWM) Pilot Project Study” by Aquafor Beech. It is possible these ponds may provide some stormwater management “credit” and that “credit” could be quantified and incorporated in future SWM pond designs. However at this time,



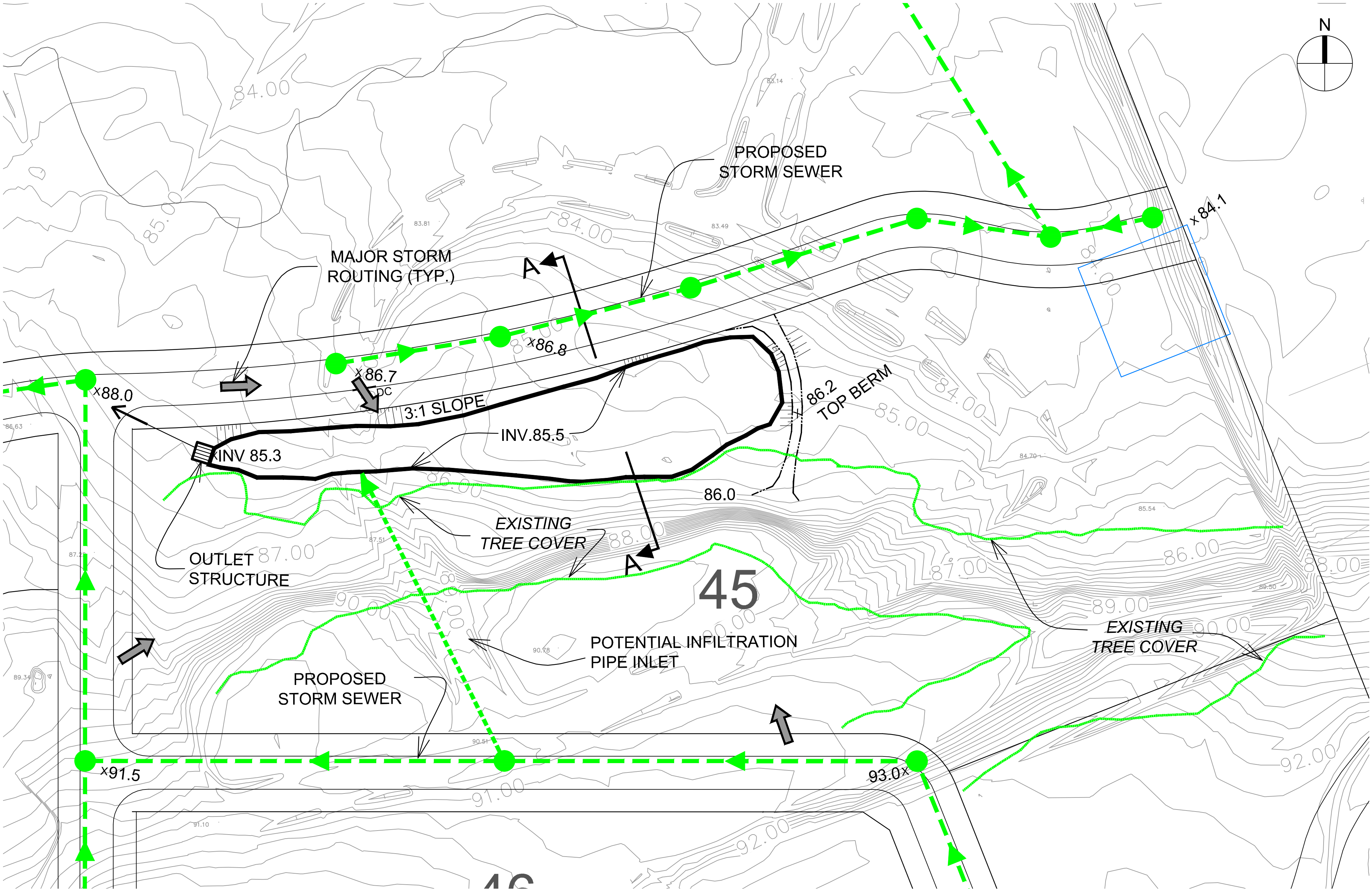
Scale	Project Title	Drawing Title	Sheet No.
NTS	FORMER CFB ROCKCLIFFE MASTER SERVICING STUDY	PARK DRY POND	FIGURE 6.19

IBI

J:\32952-RockcliffeRedev\5.9 Drawings\59civil\current\Report Figures\MSS 2015\Section 6\Figure 6.19 Park Dry Pond.dwg Layout Name: Figure6.19

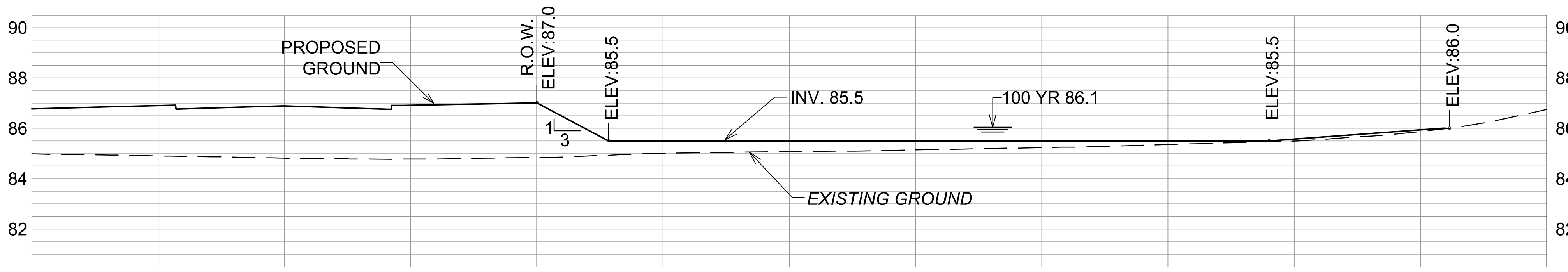


Scale	Project Title	Drawing Title	Sheet No.
NTS	FORMER CFB ROCKCLIFFE MASTER SERVICING STUDY	PARK DRY POND SECTION A-A	FIGURE 6.20



Scale	Project Title	Drawing Title	Sheet No.
NTS	FORMER CFB ROCKCLIFFE MASTER SERVICING STUDY	EASTERN DRY POND	FIGURE 6.21

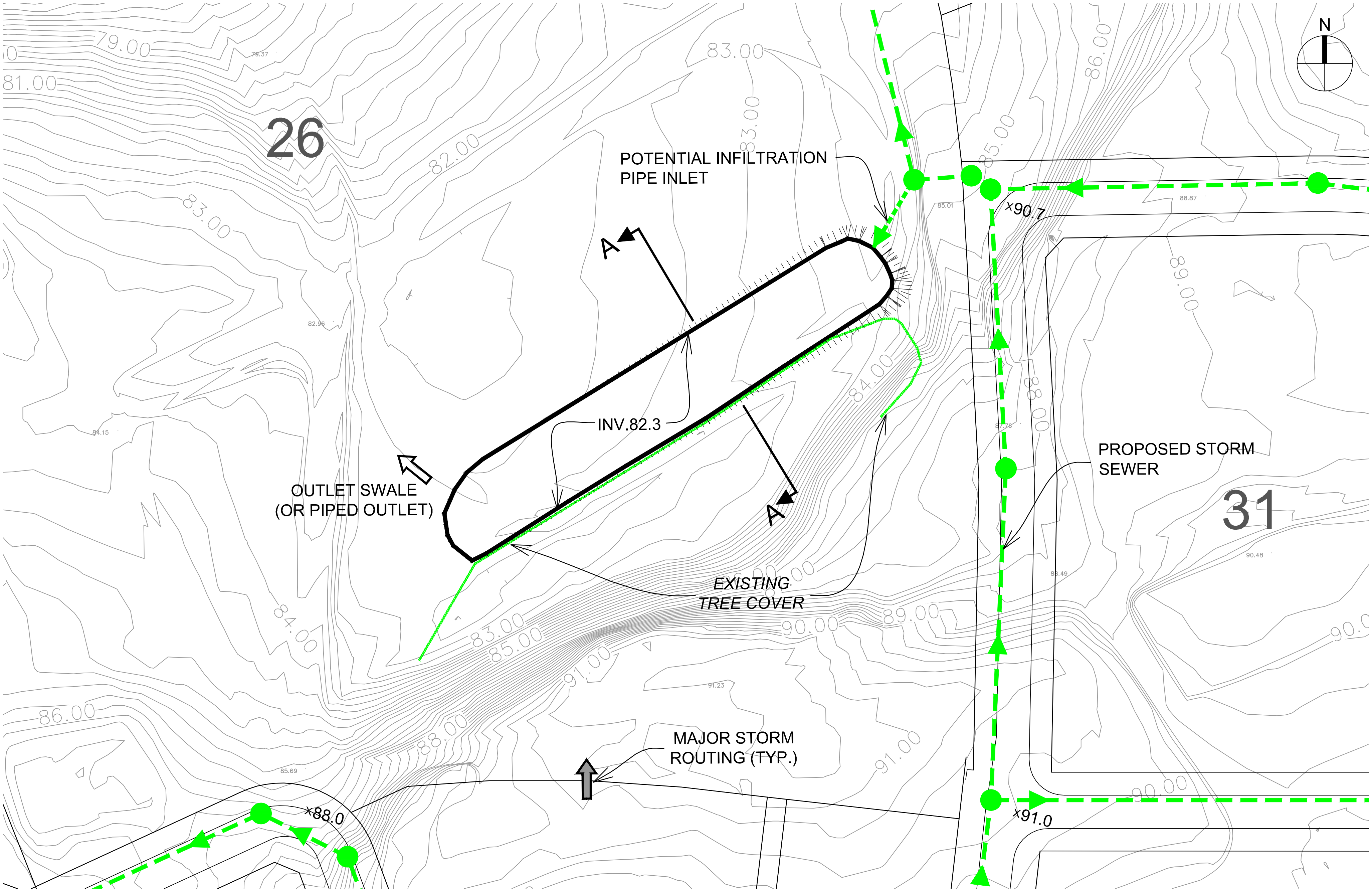
IBI



J:\32952-RockcliffeRedev\5.9 Drawings\59civil\current\Report Figures\MSS 2015\Section 6\Figure 6.22 Eastern Dry Pond Section A-A.dwg Layout Name: Figure6.22

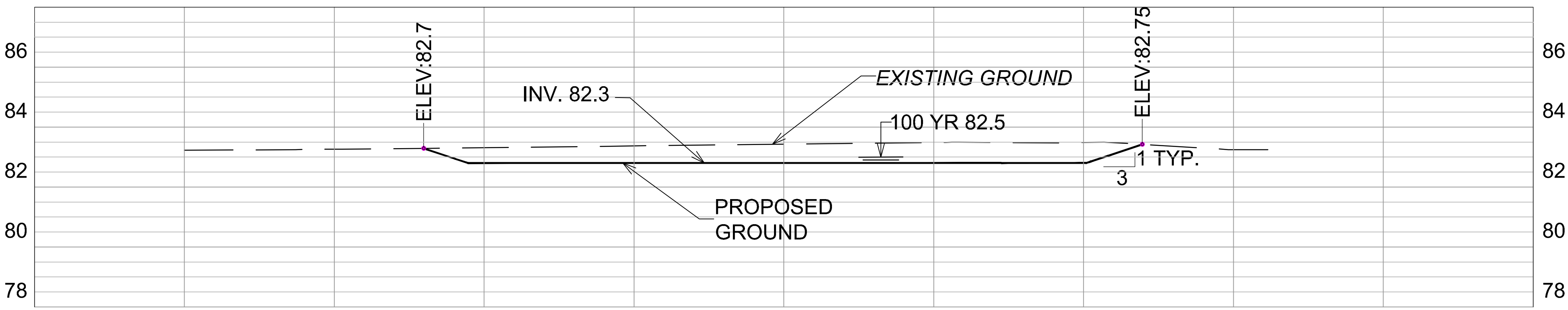
Scale	Project Title	Drawing Title	Sheet No.
NTS	FORMER CFB ROCKCLIFFE MASTER SERVICING STUDY	EASTERN DRY POND SECTION A-A	FIGURE 6.22

J:\32952-RockcliffeRedev\5.9 Drawings\59civil\current\Report Figures\MSS 2015\Section 6\Figure 6.23 Central Dry Pond.dwg Layout Name: Figure6.23



Scale	Project Title	Drawing Title	Sheet No.
NTS	FORMER CFB ROCKCLIFFE MASTER SERVICING STUDY	CENTRAL DRY POND	FIGURE 6.23

IBI



J:\32952-RockcliffeRedev\5.9 Drawings\59civil\current\Report Figures\MSS 2015\Section 6\Figure 6.24 Central Dry Pond Section A-A.dwg Layout Name: Figure6.24

IBI

Scale	Project Title	Drawing Title	Sheet No.
NTS	FORMER CFB ROCKCLIFFE MASTER SERVICING STUDY	CENTRAL DRY POND SECTION A-A	FIGURE 6.24

since any dry pond feature "credit" is unknown, the stormwater management concept for the subject site has been developed as standard SWM facilities, assuming all development areas are tributary to the end-of-pipe facilities.

6.5.4.2.3 Southwest Channel

As noted in **Section 6.5.3**, there is a channel proposed to be constructed around the south and west of the study area. It will extend west from Codd's Road, between the redevelopment and the Montfort Hospital Woods, from which it will extend north prior to discharging to the Western SWM Facility (refer to **Figure 6.14**). Major flow from a portion of the study area will be directed to the channel, as will total flow from Fairhaven and the Montfort Hospital Woods, as well as the outflow from the Montfort Hospital SWM Facility.

The proposed channel will consist of a trapezoidal cross-section with a bottom ranging in width from 1-3 m, and 3H:1V side slopes that tie into existing grades to the south and west and the proposed development to the north and east. The proposed longitudinal slope of the channel ranges from between 0.5 and 1.7%. At its downstream end, the total estimated flow to be conveyed to the channel is 2.9 cms. The estimated depth of flow in the channel during the 100 year event is 0.6 m. Two conceptual cross-sections are indicated on **Figure 6.25**. Capacity calculations for four reaches of the proposed feature are included in **Appendix E**. The channel design will be further refined to encourage infiltration, as part of Aquafor Beech's proposed work plan for the LID pilot project.

6.5.4.2.4 NRC Swale

As noted in **Section 6.5.3,** NRC runoff presently drains westward across the subject site especially over the employment Block 56 (refer to **Figure 1.3**). A swale is proposed along the eastern and southern edge of block 56 to capture and convey NRC surface runoff and direct it to the improved Burma Pond during Phase 2 construction (refer to **Figure 6.14**). The existing ditch will be modified if and as needed to accommodate the NRC runoff.

The proposed swale will include a trapezoidal cross-section with a bottom width of 3 m, and 2.5H:1V side slopes that tie into existing grades to the south and east and the proposed development to the north and west. The average longitudinal slope of the swale will be about 0.65%. The total expected flow to be conveyed to the swale is 5.7 cms. The estimated depth of flow in the channel during the 100 year event is 0.93 m. A capacity calculation for the proposed swale is included in **Appendix E**.

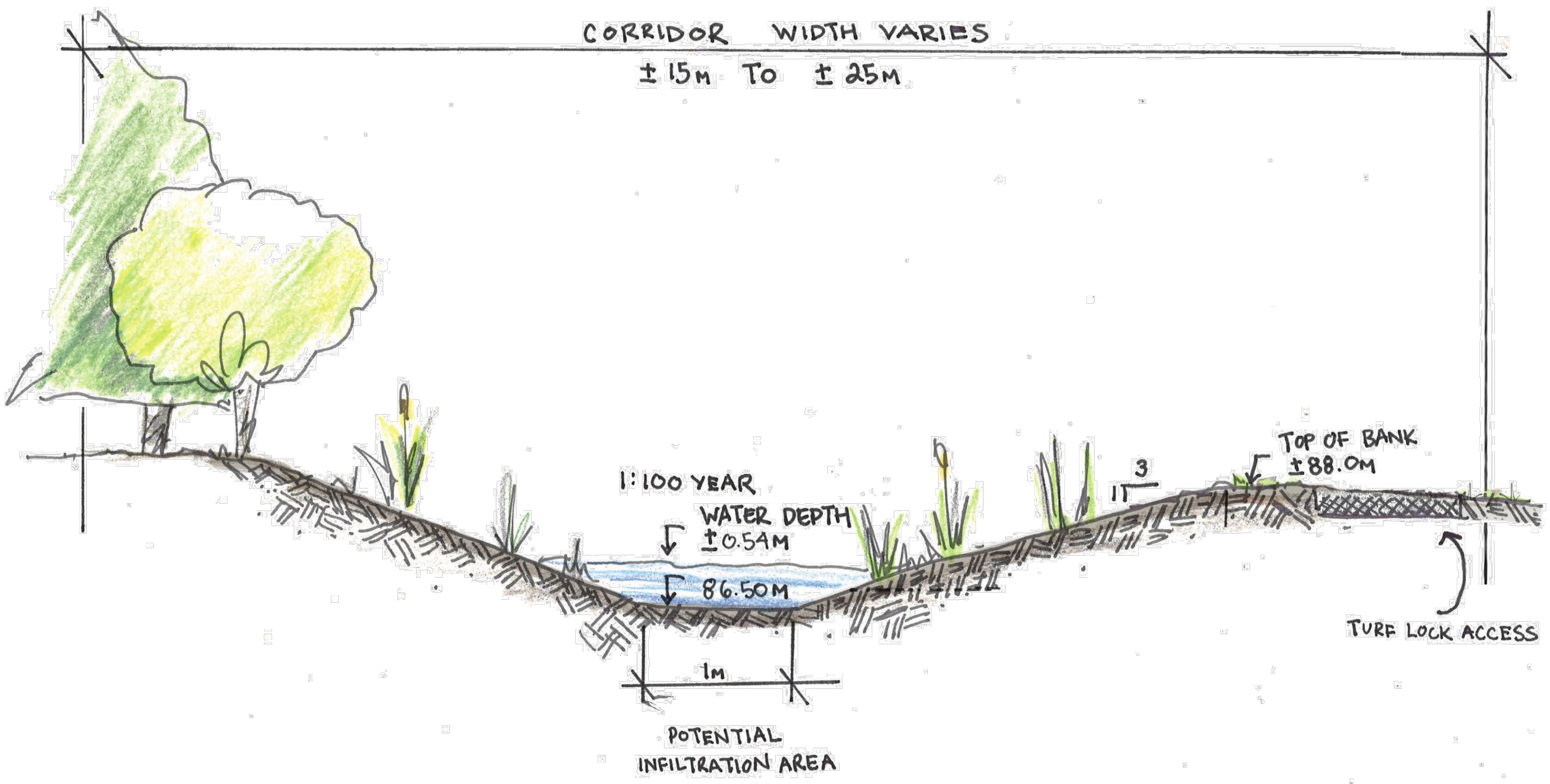
6.5.4.3 Storm Sewer Servicing of Future Museum Site

The Eastern SWM facility will be sized to provide quality treatment of minor storm runoff from the future museum site. The design of the SWM facility will include an inflow pipe element designed for the anticipated museum site. The future design of the museum site will include the minor storm sewer pipe from the site to the pond facility.

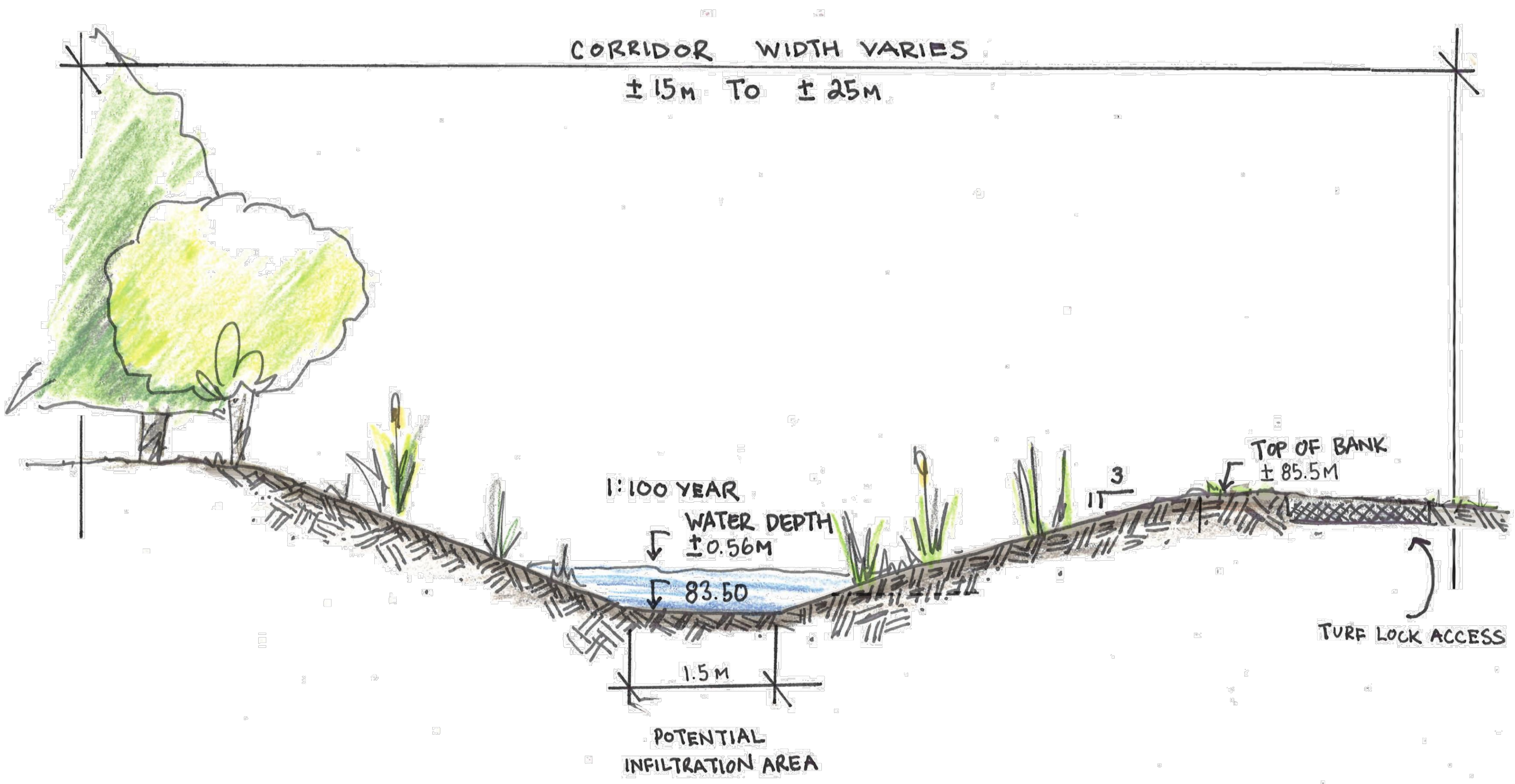
6.5.4.4 Channel-pipe Configuration

Treated runoff from the western SWM facility will be routed to the Ottawa River via two conduits. Some flows will be routed to the River through the Western Creek starting at the existing 900 mm diameter culvert under the Aviation Parkway. Flows in excess of the culvert capacity will be routed eastward through a combination channel and pipe conduit, which will be designed to bypass the Eastern facility and outlet directly to the Ottawa River.

At the upstream end of the Aviation Parkway culvert, a short trapezoidal channel is proposed to convey flow in excess of the Aviation Parkway culvert capacity. The upstream channel invert is proposed to match that of the culvert obvert (61.2 m) thereby ensuring the culvert capacity is not



SECTION A-A
(SEE FIGURE 6-13)



SECTION B-B
(SEE FIGURE 6-13)

Sheet No.	Drawing Title	Project Title	Scale
FIGURE 6.25	SOUTHWEST CHANNEL TYPICAL SECTIONS	FORMER CFB ROCKCLIFFE MASTER SERVICING STUDY	N.T.S.

IBI

exceeded. The channel bottom width is proposed to be about 3 m with 3H:1V side slopes. The channel is proposed to be about 120 m long and be constructed at an average slope of 0.50% and eventually outlet into a 800 m long 1350 mm diameter pipe.

The 1350 mm diameter pipe will be designed to convey the Western facility excess flow around the Eastern SWM Facility, tying into the 2400 mm diameter Eastern SWM Facility outlet sewer, where it will be released to the Ottawa River (refer to **Section 6.5.4.5.1**).

Minor storm runoff from the future museum site development will connect independently to the Eastern facility. A future inlet to the Eastern SWM Facility will need to be constructed to accommodate the museum site runoff. The potential new inlet structure and forebay are schematically noted as “Potential Sediment Forebay” on **Figure 6.26**.

6.5.4.5 Conceptual Design of End-of-Pipe SWM Facilities

The two end-of-pipe SWM facilities were included in the XPSWMM simulation to confirm design and performance. Each facility is discussed below. As noted in **Section 6.4.2**, at this stage of project, it is assumed that all development area, with the exception of the Special Design Area and the small area west of the Western SWM Facility (Area PH3A and Area PH2I respectively, refer to **Figures 6.13 and 6.14**), is tributary to one of the two end-of-pipe SWM facilities.

6.5.4.5.1 Eastern SWM Facility

As indicated on **Figure 6.13**, the future trunk storm sewer servicing the eastern portion of the study area is proposed to extend north from the development towards the escarpment bordering the study area. It is proposed that the 3000 mm x 3600 mm trunk storm sewer will terminate at the top of the escarpment and the runoff will cascade to the SWM facility below via a waterfall. At the detailed design stage, additional geotechnical field work will be completed to accurately delineate the rock and waterfall profile. Further, the MSS has built in flexibility in the waterfall location to accommodate species at risk (discussed in **Section 2.3.4**).

As noted in **Section 6.5.4.3**, a future storm sewer north of the museum site, extending east towards the SWM facility will convey the minor storm runoff from the museum to the Eastern SWM Facility for water quality treatment. This second inlet to the SWM facility (refer to **Figures 6.13 and 6.14**) will be constructed only at the time the museum site is developed.

The conceptual Eastern SWM Facility, designed as a wet pond (refer to **Section 6.4.2**), is comprised of a stilling basin, and a wet cell, with an outlet structure to a new storm sewer to the Ottawa River. At the upstream end of the facility, the stilling basin will provide energy dissipation for the proposed waterfall. Runoff will then flow into the sediment forebay, prior to the downstream wet cell. Outflow from the facility is proposed to discharge via the outlet structure to a new storm sewer which will convey runoff from the pond directly to the Ottawa River.

The presence of birds frequenting the stormwater facility is a concern of the nearby Rockcliffe Airport. Among other features, floating islands can act as a bird deterrent. However, the City has indicated that it does not want to endorse floating islands without additional information. It is therefore recommended that as per the “Thermal Mitigation for End-of-the-Pipe Ponds” section in the May 2015 “Former CFB Rockcliffe Redevelopment – Stormwater Management Existing Conditions and LID Demonstration Project Scoping Document” by Aquafor Beech, that floating islands be considered as a LID initiative.

In addition to floating islands, the pond can also be designed with smaller open water surfaces which is considered less desirable to birds. As a further deterrent to birds, the pond edges can be raised with small retaining walls and/or heavily planted. Birds like to travel from water to land or vice versa without navigating a vertical feature and are also suspicious of vegetation in which predatory animals can seek shelter. Therefore, a combination of smaller water surfaces, pond edge treatment and potentially floating islands are recommended elements to discourage the presence of birds.

A conceptual plan, profile and typical cross section of the facility are presented on **Figures 6.26**, **6.27**, and **6.31** respectively**.** The bottom of the facility is proposed to be set at 53.70 m and the permanent water level is 2.3 m higher (elevation 56.00 m) resulting in an estimated permanent volume of 3.59 ha-m. The performance of the conceptual Eastern SWM Facility is summarized in **Table 6.17**.

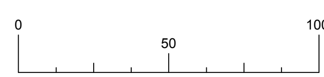
Water quality was simulated using the 25 mm Chicago storm event. During this storm event, the water quality extended storage is 1.81 ha-m with a depth of 0.88 m. The outflow from the facility during the 25 mm event is 0.97 cms. The outflow hydrograph from the facility is provided in **Appendix E**. The results indicate that during the water quality storm event the outflow would be released over 24 hours, as per MOE requirements.

During the 100 year 24 hour SCS Type II event, considered the design storm for the pond, 4.56 ha-m of active storage is expected to be utilized at an estimated depth of 2 m (elevation 58.00 m). The 12.21 cms outflow from the SWM facility, during the 100 year storm event, will be controlled by a 1300 mm diameter orifice (invert 56.00 m) and 4.80 m weir (elevation 57.00 m, see **Figure 6.27**). During less frequent storm events, flow from the facility will be conveyed through the water quality orifice only. The outflow from the facility will be conveyed to the Ottawa River via a new 2400 mm diameter storm sewer. A preliminary eastern SWM facility outlet pipe profile is provided in **Figure 6.28** (enclosed in **Appendix E**)**.** The existing storm sewers servicing the Area ROCAIR and Area PUMST (refer to **Figure 6.13**) will be tied into the new outlet storm sewer. The total tributary flow from these areas will be conveyed to the Ottawa River through the new 2400 mm diameter storm sewer. In a case where the outlet structure becomes fully blocked and the SWM facility continues to receive inflow, runoff from the eastern SWM facility will discharge into Eastern Creek as shown in **Figure 6.26**. An emergency overflow route from the facility into Eastern Creek is provided via a berm located immediately downstream of the structure (**Figure 6.26**). The berm has been graded conceptually 0.05 m below the rest of the berm, at elevation 58.25 m, for approximately 10 m section of berm. If the water level in the facility rises to 58.25 m, then the water will overtop the berm and discharge into Eastern Creek as shown in **Figure 6.26**.

Total flow from Area EXTE2 and Area PH3A (the Special Design Area) and major flow from Area PH3I (refer to **Figures 6.13 and 6.14**) are tributary to the Eastern Creek. The areas could generate almost 2.89 cms during the 100 year 3 hour Chicago event (the more critical storm event). This value is less than the maximum capacity calculated for the existing culvert (estimated at 3.5 cms, refer to **Section 6.3.3.1**).

Bankfull flow conditions in the Eastern Creek were simulated using the 25 mm storm event. Statistically, the 25 mm precipitation corresponds to a storm event with approximately a 1:5 year return period. The estimated bankfull flows could range between 0.50 cms and 2.5 cms (DST, September 2013). From a fluvial geomorphology perspective the flow generated by the above-noted drainage areas during the 25 mm storm event is 0.56 cms, which is at the lower end of the bankfull estimates.

It is proposed to provide baseflow augmentation to the Eastern Creek from the Eastern SWM Facility by means of a small diameter pipe. Based on a volumetric calculation, 51,081 cu-m/year is to be conveyed from the pond to the creek (refer to supporting calculations in **Appendix E**). It is recommended that this be confirmed at the detailed design stage. It should be noted that the required baseflow augmentation will be based on the future analysis of the Eastern Creek.



Project Title	Drawing Title	Sheet No.
FORMER CFB ROCKCLIFFE MASTER SERVICING STUDY	EASTERN SWM FACILITY PLAN VIEW	FIGURE 6.26