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File: 2160

FUNCTIONAL SERVICING REPORT

Westwood Estates (Phase 3)
City of Port Colborne
December 2022

INTRODUCTION

This report is to address the servicing needs for the proposed residential development of Westwood Estates (Phase 3) located within the last remaining lands of the Westwood Park Secondary Plan, south of Stanley Street, east of Cement Road, west of Olga Drive, and north of the Eagle Marsh Drain in the City of Port Colborne.

The 30.55 hectare property shall consist of a mix of single detached dwellings, street townhouse dwellings, and a future apartment Block (Block 178). The site will include associated asphalt parking lot, concrete curb, catch basins, storm sewers, sanitary sewers, and watermain.

The objectives of this study are as follows:

1. Identify domestic and fire protection water service needs for the site;
2. Identify sanitary servicing needs for the site; and,
3. Identify stormwater management needs for the site.

WATER SERVICING

There is an existing 200mm diameter Municipal watermain located on Sugarloaf Street as well as a 150mm diameter Municipal watermain located on Lancaster Drive. It is proposed to connect to the existing municipal watermain on Sugarloaf Street and extend a new 200mm diameter municipal watermain within the subject lands to Lancaster Drive to provide looped watermain system for domestic water supply and fire protection. The remaining local streets will be serviced with local 150mm diameter watermains.

Proposed hydrants located within the development will provide adequate fire protection. The spacing and location of the proposed fire hydrants will be identified as part of the detailed engineering design.

SANITARY SERVICING

There is an existing 350mm diameter sanitary sewer on Sugarloaf Street as well as an existing 300mm diameter sanitary sewer on Lancaster Drive. It is proposed to convey the proposed sanitary flows from the subject lands to the existing 350mm diameter sanitary sewer on Sugarloaf Street. Due to potential grading constraints within the subject lands, the existing 350mm diameter sanitary sewer on Sugarloaf may need to be reconstructed and lowered to Schofield Avenue. The extent of the reconstruction works will be determined as part of the detailed engineering design.

The subject lands consist of a total sanitary drainage area of approximately 15.81 hectares and a corresponding population of approximately 945 persons, including Block 178. The peak sanitary flow from the subject lands is approximately 18.60 L/s which corresponds to approximately 21.6% of the total capacity within the existing 350mm diameter sanitary sewer on Sugarloaf Avenue. Therefore, there is expected to be adequate capacity within the existing sanitary sewers to service the subject lands.

As part of the pre-consultation process for the subject lands, a wet and dry weather flow analysis was requested by the Region of Niagara to ensure the receiving system has adequate capacity throughout the sanitary sewer's lifecycle. Table 1 shows the corresponding wet and dry weather sanitary flows generated from the site.

Table 1. Wet and Dry Weather Flow Analysis	
Residential Dry Weather Flow	
275 L/cap/day - *945 persons*	259,875 L/day
Allowable Initial Leakage per OPSS.MUNI 410	
0.075 L/mm diameter/100m of sewer/hour - *250 mm dia, 2400m total sewer length*	10,800 L/day
Maximum End of Life Infiltration Allowance	
0.286 L/s/ha – *15.81 ha*	390,671 L/day

STORMWATER MANAGEMENT PLAN

A separate Stormwater Management Plan has been prepared by Upper Canada Consultants (UCC) and has been enclosed in Appendix A for reference. The following shall provide a summary of the enclosed Stormwater Management Plan.

The site discharges peak stormwater flows to the Eagle Marsh Drain, which is ultimately discharges into Lake Erie. As per the recommendations by the Niagara Region, the SWM facilities have been designed to Enhanced Level Protection (80% TSS Removal).

The subject lands are located immediately upstream of the Eagle Marsh Drain's ultimate outlet to Lake Erie. Therefore, stormwater management quantity controls are not required from the subject lands.

A permanent water elevation is present the Eagle Marsh Drain, which is maintained by the water elevation in Lake Erie. Therefore, downstream erosion effects are not anticipated in the Eagle Marsh Drain due to uncontrolled stormwater flows discharging from the subject lands in frequent storm events and it is not considered necessary to provide downstream erosion protection from proposed stormwater management facilities within the subject lands.

It is proposed to construct two stormwater management wet pond facilities (A1 and A2) to provide only stormwater quality controls for the subject lands prior to discharging to the Eagle Marsh Drain. The following tables summarize the MECP design criteria for the required quality controls and the proposed SWM Facility characteristics designed to achieve the criteria.

Table 2. SWM Facility 'A1' – MECP Quality Requirements Comparison

SWM Facility Characteristic	MECP Requirement	Provided by SWM Facility
Permanent Pool Volume (m^3) - *minimum*	1,043	1,282
Extended Detention Volume (m^3) – *minimum*	241	1,435
Total Quality + Detention Storage (m^3) – *minimum*	1,284	2,717
Facility Drawdown Time (hours) – *minimum*	24	25
Forebay Length (m) – *minimum*	21.60	30.00
Forebay Width (m) – *minimum*	2.70	3.00
Average Forebay Velocity (m/s) – *maximum*	0.15	0.06
Cleanout Frequency (years) - *minimum*	10	11

Table 3. SWM Facility 'A2' Characteristics

Design Storm (Return Period)	Peak Flows (m3/s)		Maximum Elevation (m)	Maximum Volume (m3)
	Inflow	Outflow		
25 mm	0.670	0.022	175.91	1,405
5 Year	1.346	0.122	176.40	3,170

Table 3. SWM Facility 'A2' – MECP Quality Requirements Comparison		
SWM Facility Characteristic	**MECP Requirement**	**Provided by SWM Facility**
Permanent Pool Volume (m^3) - *minimum*	2,157	2,421
Extended Detention Volume (m^3) – *minimum*	764	2,557
Total Quality + Detention Storage (m^3) – *minimum*	2,921	4,978
Facility Drawdown Time (hours) – *minimum*	24	32
Forebay Length (m) – *minimum*	21.54	24.50
Forebay Width (m) – *minimum*	2.69	4.50
Average Forebay Velocity (m/s) – *maximum*	0.15	0.09
Cleanout Frequency (years) - *minimum*	10	10

Table 4. SWM Facility 'A2' Characteristics				
Design Storm (Return Period)	**Peak Flows (m3/s)**		**Maximum Elevation (m)**	**Maximum Volume (m3)**
	Inflow	**Outflow**		
25 mm	0.670	0.022	175.91	1,405
5 Year	1.346	0.122	176.40	3,170

The minor stormwater flows shall be conveyed through the proposed storm sewer system to the proposed stormwater management facilities and to the Eagle Marsh Drain. Major overland flows will be primarily conveyed to the wet pond facilities, which further directs the flows overland to the Eagle Marsh Drain.

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CONCLUSIONS AND RECOMMENDATIONS

Therefore, based on the above comments and design calculations provided for this site, the following summarizes the servicing for this site.

1. The existing 200mm diameter municipal watermain on Sugarloaf Street will have sufficient capacity to provide both domestic and fire protection water supply.

2. The existing 350mm diameter municipal AC sanitary sewer on Sugarloaf Street will have adequate capacity for the proposed development. The existing 350mm diameter sanitary sewer on Sugarloaf may need to be reconstructed and lowered to Schofield Avenue at an extent to be determined as part of detailed engineering design.

3. Stormwater quantity controls and erosion protection are not considered necessary for the subject lands.

4. Stormwater quality protection is being provided by the two wet pond facilities up to Enhanced (80% TSS) Level Protection as per the recommendation of the Region of Niagara.

Based on the above and the accompanying General Servicing Plan, and Drainage Area Plans, there exists adequate municipal servicing for this development. We trust the above comments and enclosed calculations are satisfactory for approval. If you have any questions or require additional information, please do not hesitate to contact our office.

Respectfully Submitted,

B. Kapteyn

LICENSED PROFESSIONAL ENGINEER
B. J. KAPTEYN
100509155
PROVINCE OF ONTARIO

Brendan Kapteyn, P.Eng.

Encl.

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APPENDICES

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APPENDIX A

Westwood Estates (Phase 3) Stormwater Management Plan

STORMWATER MANAGEMENT PLAN

WESTWOOD ESTATES (PHASE 3)

CITY OF PORT COLBORNE

Prepared by:

Upper Canada Consultants
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L2W 1A3

December 2022

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Appendix E Future HEC-RAS Cross Sections (with Levee)

REFERENCES

1. Stormwater Management Planning and Design Manual
   Ontario Ministry of Environment (March 2003)

2. Soils of the Regional Municipality of Niagara Soil Survey Report No. 60 of the Ontario Institute of Pedology. (1989)

STORMWATER MANAGEMENT PLAN

WESTWOOD ESTATES (PHASE 3)

CITY OF PORT COLBORNE

1.0 INTRODUCTION

1.1 Study Area

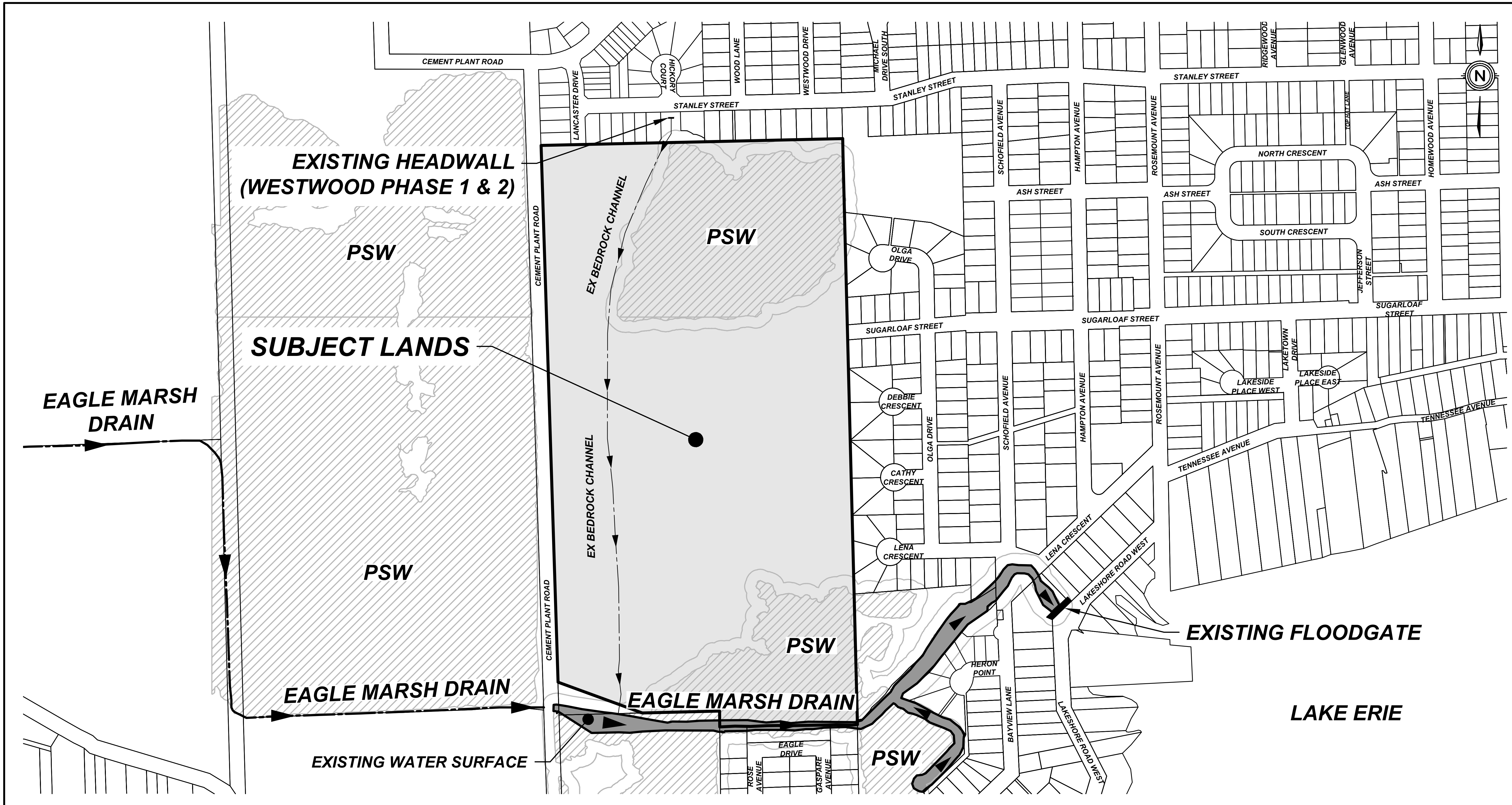
The proposed residential development of Westwood Estates (Phase 3), is located within the remaining lands of the Westwood Estates Park Secondary Plan in City of Port Colborne. As shown on the enclosed Site Location Plan (Figure 1), the subject property is situated south of Stanley Street, east of Cement Road and west of Olga Drive, and north of the Eagle Marsh Drain in the City of Port Colborne.

The study area is approximately 30.55 hectares and shall consist of a mix of single detached dwellings, street townhouse dwellings, and a future apartment Block (Block 178). The site will include associated asphalt parking lot, concrete curb, catch basins, storm sewers, sanitary sewers, and watermain.

1.2 Objectives

The objectives of this study are as follows:

1. Establish specific criteria for the management of stormwater from this site.
2. Determine the impact of development on the stormwater peak flow & volume of stormwater from the drainage area.
3. Investigate alternatives for controlling the quality of stormwater discharging from the site.
4. Establish the property requirements to construct a stormwater management facility for the Draft Plan of Subdivision.



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WESTWOOD ESTATES (PHASE 3)

CITY OF PORT COLBORNE
SITE LOCATION PLAN

DATE	2022–12–08
SCALE	1:5000 m
REF No.	2160
DWG No.	FIGURE 1

1.3 Existing & Proposed Conditions

a) Existing Conditions

The site has been partially used as an agriculture land and remaining portion is undeveloped open space with two Provincially Significant Wetlands (PSW) located in the north-east and on the south-east portions of the site.

The topography of the site is relatively flat with a general southerly slope towards the Eagle Marsh Drain. There is an existing drainage channel within the middle of the site, flowing from north to south providing a stormwater outlet for the previously constructed Phases of the Westwood Estates Subdivision (Phases 1 and 2). This drainage channel was constructed within the existing shallow bedrock present within the subject lands.

The soils within the subject lands, according to the Ontario Institute of Pedology, predominantly consist of Brooke soils, with 50-100 cm of variable textures over bedrock and an infiltration rate classified as “Poorly Drained”.

b) Proposed Conditions

The development area is approximately 30.55 hectares and will consist of a mix of single family residential dwellings, street town residential dwellings and a future apartment block (Block 178). The site shall be provided with full municipal services including sanitary sewers, storm sewers and watermain with asphalt pavement, concrete curbs and gutters.

2.0 STORMWATER MANAGEMENT CRITERIA

New developments are required to provide stormwater management in accordance with provincial and municipal policies including:

- Stormwater Quality Guidelines for New Development (MECP/MNRF, May 1991)
- Stormwater Management Planning and Design Manual (MECP, March 2003)

Based on the comments and outstanding policies from the City of Port Colborne, Regional Municipality of Niagara, Niagara Peninsula Conservation Authority (NPCA), and the Ministry of the Environment, Conservation and Parks (MECP), the following site-specific considerations were identified:

- Stormwater runoff from the development shall be collected and treated to an Enhanced (80% TSS removal) standard prior to discharge to the receiving watercourse (Eagle Marsh Drain); and,
- The subject lands are located immediately upstream of the Eagle Marsh Drain’s ultimate outlet to Lake Erie. Detaining future peak stormwater flows on site will result in increasing the greater peak stormwater flows from the upstream lands within the Eagle Marsh Drain watershed.

- The Regional Municipality of Niagara has requested that downstream erosion protection be provided prior to discharging to the Eagle Marsh Drain.

Based on the above and a review of the site-specific considerations, the following stormwater management criteria have been established for this site:

- Stormwater **quality** controls are to be provided to provide Enhanced Protection (80% TSS removal) in accordance with MECP guidelines prior to outletting to the Eagle Marsh Drain;

- Stormwater **quantity** controls are not required for stormwater flows discharging from the subject lands; and,

- A permanent water elevation is present the Eagle Marsh Drain, which is maintained by the water elevation in Lake Erie. Therefore, downstream erosion effects are not anticipated in the Eagle Marsh Drain due to uncontrolled stormwater flows discharging from the subject lands in frequent storm events and it is not considered necessary to provide **downstream erosion protection** from proposed stormwater management facilities within the subject lands.

3.0 STORMWATER ANALYSIS

Since stormwater quantity controls are not required for the subject lands, future stormwater flows were modelled using the MIDUSS computer modelling program for the purposes of sizing sediment forebays and determining stormwater quality volumes **only**.

This program was selected because it is applicable to an urban drainage area like the study area, it is relatively easy to use and modify for the proposed drainage conditions and control facilities, and it readily allows for the use of design storm hyetographs for the various return periods being investigated.

3.1 Design Storms

The 5 year design storm hyetograph was developed using a Chicago distribution based on City of Welland Intensity-Duration-Frequency (IDF) curves in accordance with City of Port Colborne standards. The 25mm design storm IDF curve parameters were derived using a 4-hour Chicago distribution. Table 1 summarizes the rainfall data.

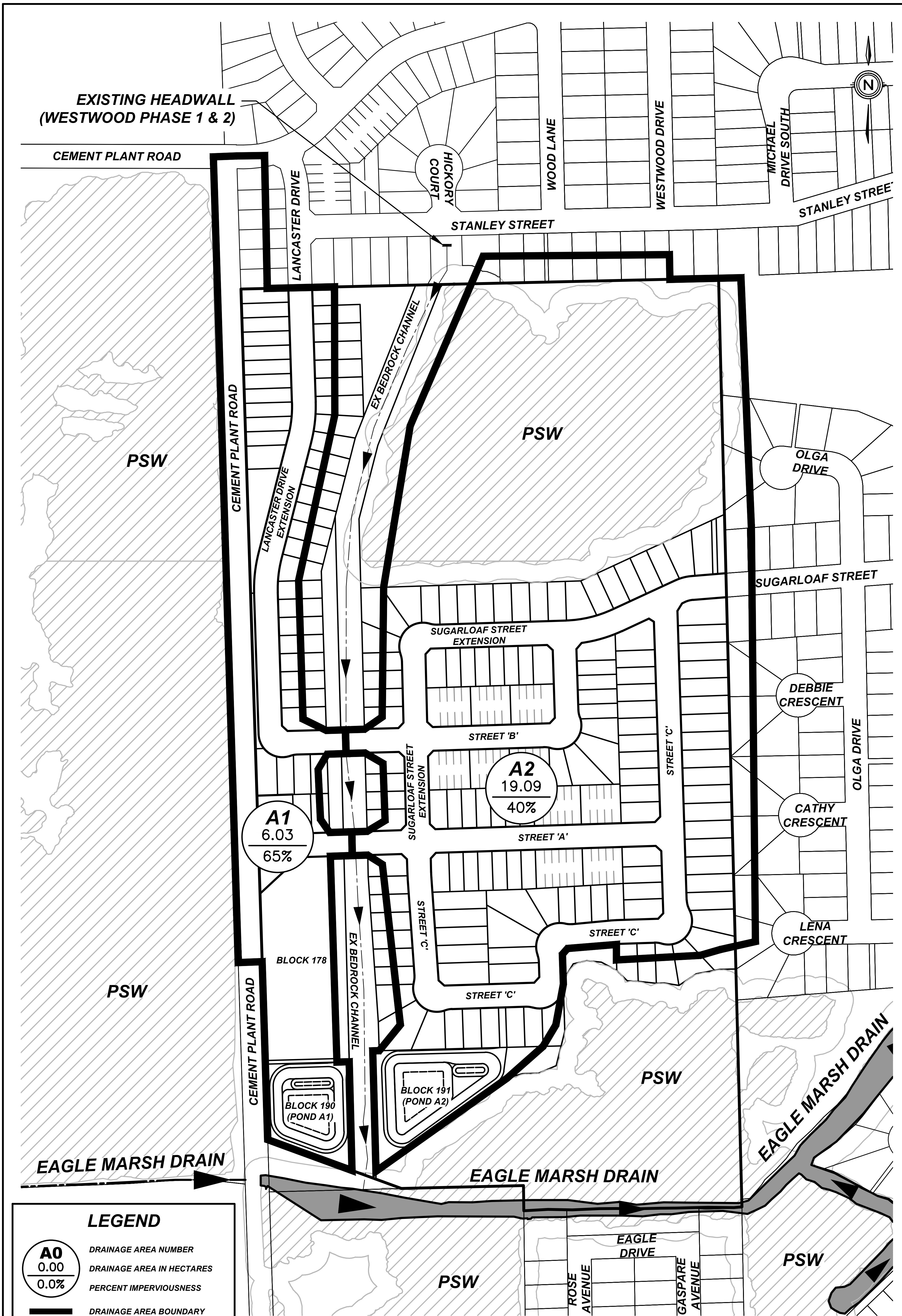
Table 1. Rainfall Data			
Design Storm (Return Period)	**Chicago Distribution Parameters**		
	a	**b**	**c**
25mm	512.0	6.00	0.800
5 Year	830.0	7.30	0.777
$\text{Intensity}\ (mm/hr) = \dfrac{a}{(t_d + b)^c}$			

3.2 Proposed Conditions

The future drainage areas for the proposed development, shown in Figure 2, were modelled to establish the stormwater peak flows and volumes once development has been completed at the proposed site for the purposes of sediment forebay sizing and determining stormwater quality control volumes **only**. Input parameters for the computer model are shown in Table 2.

Table 2. Hydrologic Parameters for Future Conditions								
Area No.	**Area (ha)**	**Length (m)**	**Slope (%)**	**Manning – "n"**		**Soil Type**	**SCS CN**	**Percent Impervious**
				Perv.	**Imperv.**			
A1	6.03	200	1.0	0.25	0.015	C	77	65%
A2	19.09	357	1.0	0.25	0.015	C	77	40%
	25.12	**Total Area (ha)**						

The detailed MIDUSS modelling output files have been enclosed in Appendix C for reference.



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WESTWOOD ESTATES (PHASE 3)

CITY OF PORT COLBORNE
FUTURE STORM DRAINAGE AREAS

DATE	2022-12-08
SCALE	1:3000 m
REF No.	2160
DWG No.	FIGURE 2

4.0 STORMWATER MANAGEMENT ALTERNATIVES

4.1 Screening of Stormwater Management Alternatives

A variety of stormwater management alternatives are available to control the quality of stormwater, most of which are described in the Stormwater Management Planning and Design Manual (MECP, March 2003). Alternatives for the proposed and ultimate developments were considered in the following broad categories: lot level, vegetative, infiltration, and end-of-pipe controls. General comments on each category are provided below. Individual alternatives for the proposed development are listed in Table 3 with comments on their effectiveness and applicability to the proposed outlet.

a) Lot Level Controls

Lot level controls are not generally suitable as the primary control facility for quality control. They are generally used to enhance stormwater quality in conjunction with other types of control facilities.

b) Vegetative Alternatives

Vegetative stormwater management practices are not generally suitable as the primary control facility for quality control. They are generally used to enhance stormwater quality in conjunction with other types of control facilities.

c) Infiltration Alternatives

Where soils are suitable, infiltration techniques can be very effective in providing quantity and quality control. However, the very small amount of surface area on this site dedicated to permeable surfaces such as greenspace and landscaping make this an impractical option. Therefore, infiltration techniques will not be considered for this development.

d) End-of-Pipe Alternatives

Surface storage techniques can be very effective in providing quality and quantity control. Wet facilities are effective practices for stormwater quality control for large drainage areas (>5 ha).

Table 3. Evaluation of Stormwater Management Practices

Westwood Estates (Phase 3)	Criteria for Implementation of Stormwater Management Practices (SWMP)							
	Topography	**Soils**	**Bedrock**	**Groundwater**	**Area**	**Technical Effectiveness (10 high)**	**Recommend Implementation Yes / No**	**Comments**
Site Conditions	***Flat ±1%***	***Variable ±15 mm/hr***	***Shallow***	***At Considerable Depth***	***± 25.1ha***			
Lot Level Controls								
Lot Grading	<5%	nlc	nlc	nlc	nlc	2	Yes	Quality/quantity benefits
Roof Leaders to Surface	nlc	nlc	nlc	nlc	nlc	2	Yes	Quality/quantity benefits
Roof Ldrs.to Soakaway Pits	nlc	loam, infiltr. > 15 mm/hr	>1m Below Bottom	>1m Below Bottom	< 0.5 ha	6	No	Unsuitable site conditions
Sump Pump Fdtn. Drains	nlc	nlc	nlc	nlc	nlc	2	Yes	Suitable site conditions
Vegetative								
Grassed Swales	< 5 %	nlc	nlc	nlc	nlc	7	Yes	Quality/quantity benefits
Filter Strips(Veg. Buffer)	< 10 %	nlc	nlc	>.5m Below Bottom	< 2 ha	5	No	Unsuitable site conditions
Infiltration								
Infiltration Basins	nlc	loam, infiltr. > 15 mm/hr	>1m Below Bottom	>1m Below Bottom	< 5 ha	2	No	Unsuitable site conditions
Infiltration Trench	nlc	loam, infiltr. > 15 mm/hr	>1m Below Bottom	>1m Below Bottom	< 2 ha	4	No	Unsuitable site conditions
Rear Yard Infiltration	< 2.0 %	loam, infiltr. > 15 mm/hr	>1m Below Bottom	>1m Below Bottom	< 0.5 ha	7	No	Unsuitable site conditions
Perforated Pipes	nlc	loam, infiltr. > 15 mm/hr	>1m Below Bottom	>1m Below Bottom	nlc	4	No	Unsuitable site conditions
Pervious Catch basins	nlc	loam, infiltr. > 15 mm/hr	>1m Below Bottom	>1m Below Bottom	nlc	3	No	Unsuitable site conditions
Sand Filters	nlc	nlc	nlc	>.5m Below Bottom	< 5 ha	5	No	High maintenance/poor aesthetics
Surface Storage								
Dry Ponds	nlc	nlc	nlc	nlc	> 5 ha	7	No	No quality control
Wet Ponds	nlc	nlc	nlc	nlc	> 5 ha	9	Yes	Very effective quality control
Wetlands	nlc	nlc	nlc	nlc	> 5 ha	6	No	Very effective quality control
Other								
Oil/Grit Separator	nlc	nlc	nlc	nlc	<2 ha	3	No	Limited benefit/area too large

Reference: Stormwater Management Practices Planning and Design Manual - 2003
nlc - No Limiting Criteria

4.2 Selection of Stormwater Management Alternatives

Stormwater management alternatives were screened based on technical effectiveness, physical suitability for this site, and their ability to meet the stormwater management criteria established for proposed and future development areas. The following stormwater management alternatives are recommended for implementation on the proposed development:

- **Lot grading** to be kept as flat as practical in order to slow down stormwater and encourage infiltration.

- **Roof leaders to be discharged to the ground surface** in order to slow down stormwater and encourage infiltration.

- **Grassed swales** to be used to collect rear lot drainage. Grassed swales tend to filter sediments and slow down the rate of stormwater.

- Two **wet pond facilities** to be constructed to provide stormwater quality enhancement.

5.0 STORMWATER MANAGEMENT PLAN

A MIDUSS model was created to assess future peak flows and stormwater volumes generated within the site. The proposed stormwater management facilities shall provide quality controls for future drainage areas ‘A1’ and ‘A2’.

It is proposed to construct two stormwater management wet pond facilities (‘A1’ and ‘A2’) which will provide stormwater management quality controls to MECP Enhanced levels (80% TSS Removal) prior to discharging to the Eagle Marsh Drain. The proposed wet ponds will collect major and minor stormwater flows from their respective drainage areas.

5.1 Proposed SWM Facility ‘A1’

5.1.1 Stormwater Quality Control

Based on Table 3.2 of SWMP & Design Manual, the water quality storage requirement is approximately 213 m^3/ha for *Enhanced* protection for developments with 65% impervious areas. The drainage area contributing peak stormwater flows to facility A1 is 6.03 hectares. The storage volumes required for the proposed quality controls are shown in Table 4.

Table 4. SWM Facility 'A1' - Stormwater Quality Volume Calculations	
Total Water Quality Volume = 6.03 ha x 213 m^3/ha = 1,284 m^3	Reference: Table 3.2, SWMP & Design Manual (MECP 2003)
Permanent Pool Volume = 6.03 ha x 173 m^3/ha = 1,043 m^3	**Extended Detention Volume** = 6.03 ha x 40 m^3/ha = 241 m^3

5.1.2 Stormwater Management Facility Configuration

As shown in Figure 3, it is proposed to construct a two-stage control outlet for the proposed stormwater management facility. The first stage of control consists of a reverse slope pipe acting as a tubular control orifice to provide the required quality controls. The second stage of control consists of a ditch inlet catch basin and outlet pipe which provides an outlet for flows exceeding the extended detention volume. An emergency spillway will provide an outlet for major storm events.

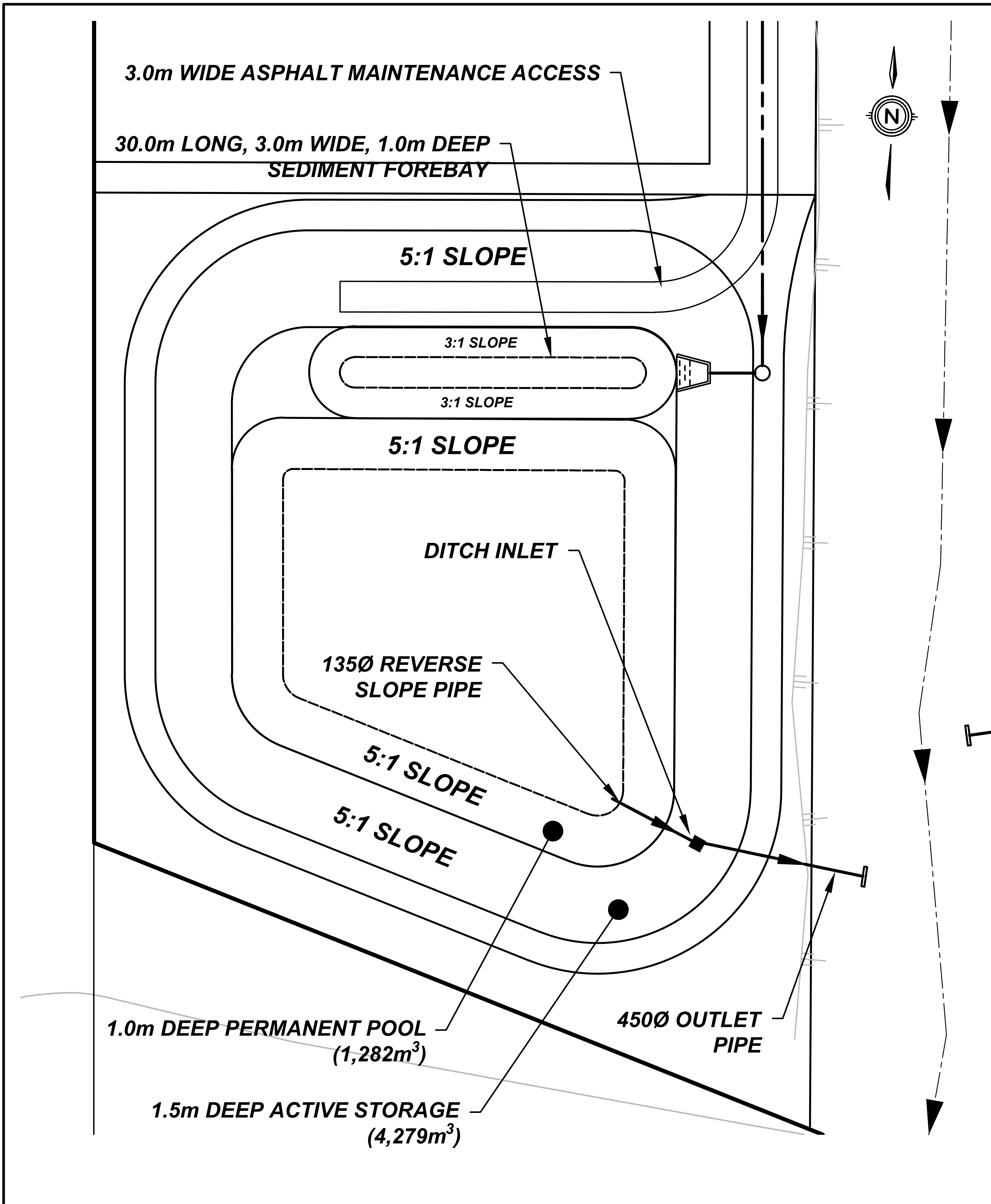
The proposed bottom elevation of the facility is 174.50 m, and the permanent pool water level is proposed at 175.50 m, for a permanent water depth of 1.0 metre. The configuration of the facility provides 1,282 m^3 of permanent pool volume, which is more than the required 1,043 m^3. The proposed top of pond is at an elevation of 177.00 m which provides a total active volume of 4,279 m^3 with 5:1 side slopes.

Based on the configuration of the proposed facility, it was determined that a 135 mm diameter quality orifice at an invert of 175.50 m can provide 25 hours of detention with the proposed ditch inlet catch basin being constructed with a rim elevation of 176.10 m, which is greater than the minimum drawdown time of 24 hours. This configuration will provide an extended detention volume of 1,435 m^3, which is greater than the minimum volume of 241 m^3 specified in Table 4.

Stage-storage-discharge calculations have been prepared for this facility and are included in Appendix A for reference.

Major overland flows within the drainage area tributary to facility A1 will be directed either to the SWM facility or the existing drainage channel, ultimately outletting to the Eagle Marsh Drain.

The proposed facility has a single storm sewer inlet. Therefore, a sediment forebay has been designed to minimize the transport of heavy sediments from the storm sewer outlet throughout the facility and localize maintenance activities. Calculations for the forebay sizing follow MECP guidelines and are shown in Appendix A.



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WESTWOOD ESTATES (PHASE 3)

CITY OF PORT COLBORNE
STORMWATER MANAGEMENT POND A1

DATE	2022–12–22
SCALE	1:500 m
REF No.	2160
DWG No.	FIGURE 3

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Table 5. SWM Facility 'A1' – MECP Quality Requirements Comparison		
SWM Facility Characteristic	**MECP Requirement**	**Provided by SWM Facility**
Permanent Pool Volume (m^3) - *minimum*	1,043	1,282
Extended Detention Volume (m^3) – *minimum*	241	1,435
Total Quality + Detention Storage (m^3) – *minimum*	1,284	2,717
Facility Drawdown Time (hours) – *minimum*	24	25
Forebay Length (m) – *minimum*	21.60	30.00
Forebay Width (m) – *minimum*	2.70	3.00
Average Forebay Velocity (m/s) – *maximum*	0.15	0.06
Cleanout Frequency (years) - *minimum*	10	11

As shown in Table 5, the proposed stormwater management facility configuration satisfies the quality requirements outlined by the MECP for the 6.03 hectare drainage area.

Table 6. SWM Facility 'A1' Characteristics				
Design Storm (Return Period)	**Peak Flows (m^3/s)**		**Maximum Elevation (m)**	**Maximum Volume (m^3)**
	Inflow	**Outflow**		
25 mm	0.369	0.014	175.85	677
5 Year	0.671	0.040	176.26	1,482

As shown in Table 6, the proposed stormwater management facility has adequate storage capacity to detain future 25mm and 5 year design storm flows to provide the required quality controls.

5.2 Proposed SWM Facility 'A2'

5.2.1 Stormwater Quality Control

Based on Table 3.2 of SWMP & Design Manual, the water quality storage requirement is approximately 153 m^3/ha for *Enhanced* protection for developments with 40% impervious areas. The drainage area contributing peak stormwater flows to facility A2 is 19.09 hectares. The storage volumes required for the proposed quality controls are shown in Table 7.

Table 7. SWM Facility 'A2' - Stormwater Quality Volume Calculations	
Total Water Quality Volume = 19.09 ha x 153 m^3/ha = 2,921 m^3	Reference: Table 3.2, SWMP & Design Manual (MECP 2003)
Permanent Pool Volume = 19.09 ha x 113 m^3/ha = 2,157 m^3	**Extended Detention Volume** = 19.09 ha x 40 m^3/ha = 764 m^3

5.2.2 Stormwater Management Facility Configuration

As shown in Figure 4, it is proposed to construct a two-stage control outlet for the proposed stormwater management facility. The first stage of control consists of a reverse slope pipe acting as a tubular control orifice to provide the required quality controls. The second stage of control consists of a ditch inlet catch basin and outlet pipe which provides an outlet for flows exceeding the extended detention volume. An emergency spillway will provide an outlet for major storm events.

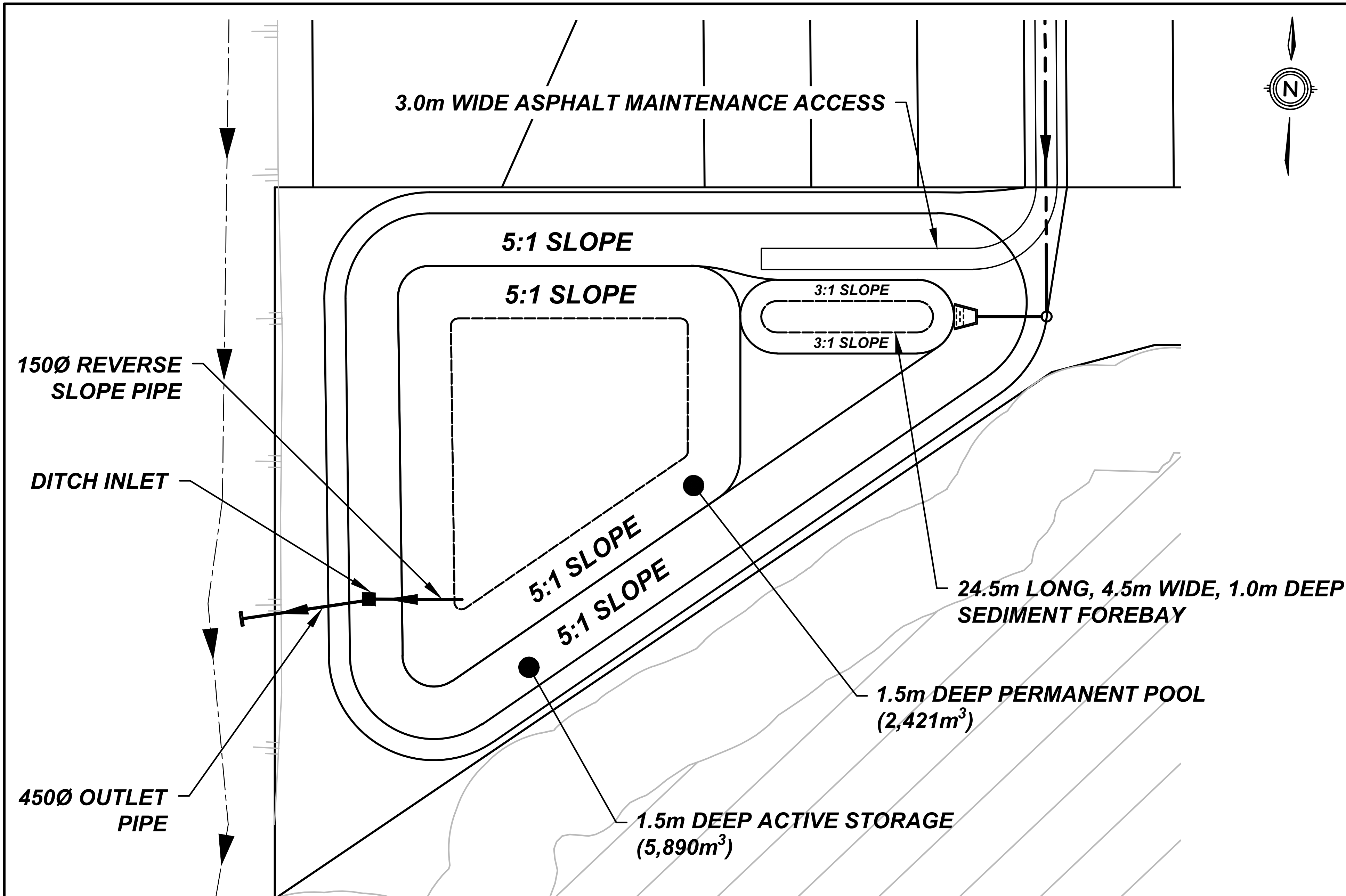
The proposed bottom elevation of the facility is 174.00 m, and the permanent pool water level is 175.50 m for a water depth of 1.5 metres. The configuration of the facility provides 2,421 m^3 of permanent pool volume, which is more than the required 2,157 m^3. The proposed top of pond is at an elevation of 177.00 m which provides a total active volume of 5,890 m^3 with 5:1 side slopes.

Based on the configuration of the proposed facility, it was determined that a 150 mm diameter quality orifice at an invert of 175.50 m can provide 32 hours of detention with the proposed ditch inlet catch basin being constructed with a rim elevation of 176.25 m, which is greater than the minimum drawdown time of 24 hours. This configuration will provide an extended detention volume of 2,557 m^3, which is greater than the minimum volume of 764 m^3 specified in Table 6.

Stage-storage-discharge calculations have been prepared for this facility and are included in Appendix B for reference.

Major overland flows within the drainage area tributary to facility A2 will be directed either to the SWM facility or the existing drainage channel, ultimately outletting to the Eagle Marsh Drain.

The proposed facility has a single storm sewer inlet. Therefore, a sediment forebay has been designed to minimize the transport of heavy sediments from the storm sewer outlet throughout the facility and localize maintenance activities. Calculations for the forebay sizing follow MECP guidelines and are shown in Appendix B.



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WESTWOOD ESTATES (PHASE 3)
CITY OF PORT COLBORNE
STORMWATER MANAGEMENT POND A2

DATE	2022–12–22
SCALE	1:750 m
REF No.	2160
DWG No.	FIGURE 4

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Table 8. SWM Facility 'A2' – MECP Quality Requirements Comparison		
SWM Facility Characteristic	**MECP Requirement**	**Provided by SWM Facility**
Permanent Pool Volume (m^3) - *minimum*	2,157	2,421
Extended Detention Volume (m^3) – *minimum*	764	2,557
Total Quality + Detention Storage (m^3) – *minimum*	2,921	4,978
Facility Drawdown Time (hours) – *minimum*	24	32
Forebay Length (m) – *minimum*	21.54	24.50
Forebay Width (m) – *minimum*	2.69	4.50
Average Forebay Velocity (m/s) – *maximum*	0.15	0.09
Cleanout Frequency (years) - *minimum*	10	10

As shown in Table 8, the proposed stormwater management facility configuration satisfies the quality requirements outlined by the MECP for the 19.09 hectare drainage area.

Table 9. SWM Facility 'A2' Characteristics				
Design Storm (Return Period)	**Peak Flows (m^3/s)**		**Maximum Elevation (m)**	**Maximum Volume (m^3)**
	Inflow	**Outflow**		
25 mm	0.670	0.022	175.91	1,405
5 Year	1.346	0.122	176.40	3,170

As shown in Table 9, the proposed stormwater management facility has adequate storage capacity to detain future 25mm and 5 year design storm flows to provide the required quality controls.

5.3 100 Year Floodplain

The NPCA generated a 100 year floodplain for the Eagle Marsh Drain with a detailed HEC-RAS model. The HEC-RAS model includes detailed cross sections along the watercourse to determine the extents of the existing 100 year floodplain to the outlet at Lake Erie. The cross sections along the southern limit of the site and the existing 100 year floodplain are shown in Figure 3.

The construction of SWM facilities A1 and A2 will include earthworks within Block 190 and 187 of the proposed Draft Plan of Subdivision respectively, which can potentially impact the 100 year floodplain associated to the Eagle Marsh Drain.

In accordance with NPCA policies, no earthworks will occur within the adjacent regulated wetland or the associated 15m regulated Wetland Buffer (Block 186). Therefore, since the existing 100 year floodplain is completely contained within Block 186, the proposed lots along the boundary of this Block will not impact the existing 100 year floodplain.

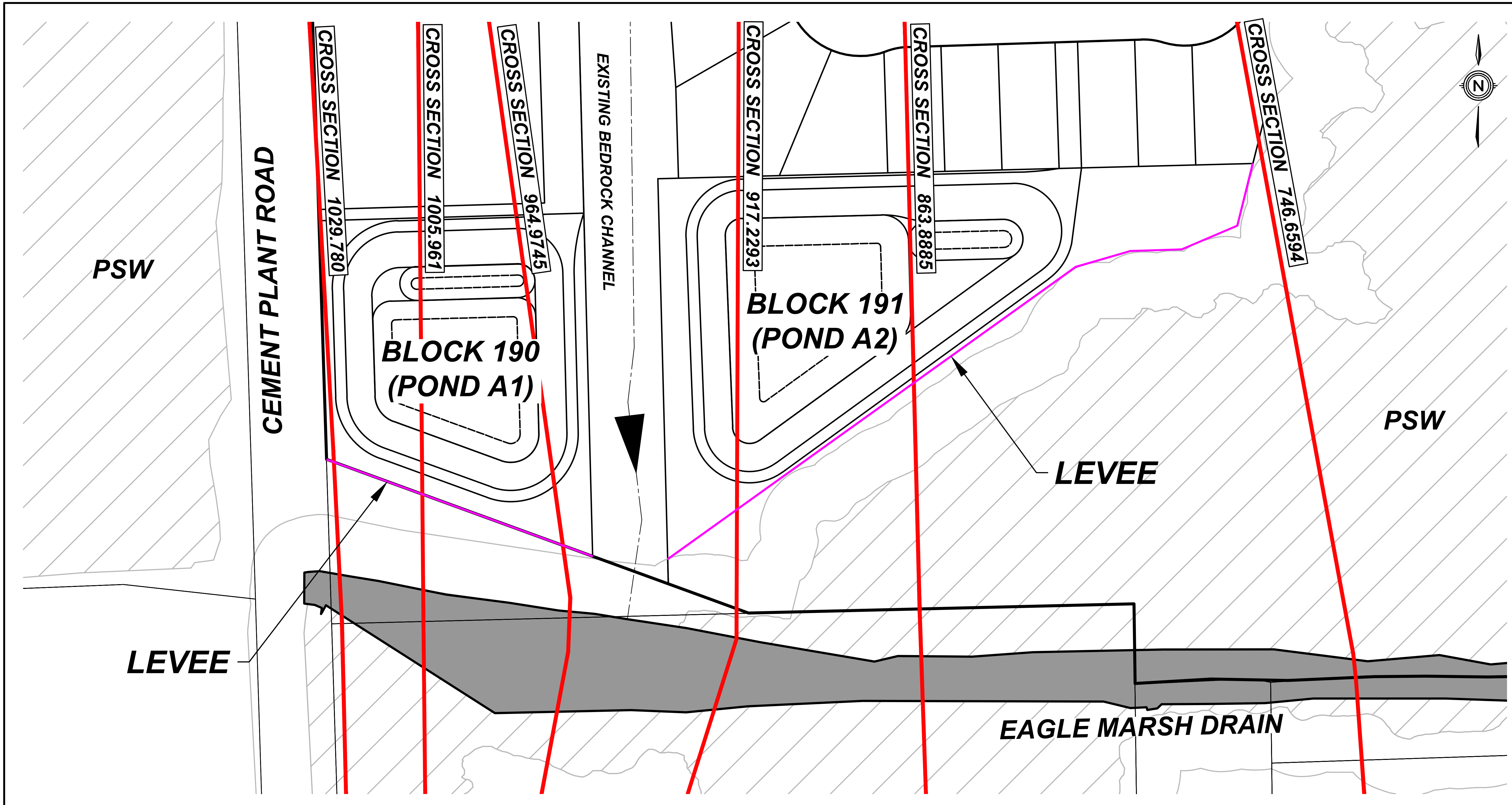
To determine the impact of future grading works within Blocks 190 and 191, a "levee" was added to the HEC-RAS model at the southern limits of these Blocks to simulate future conditions, where the footprint of the floodplain will be reduced by the future pond banks. A comparison of the 100 year flood elevations modelled with and without the "levee" is shown in Table 8.

Table 10. Comparison of Existing and Future 100 Year Floodplain Elevations

Cross-section ID	Flood Elevation (m)		
	Existing Conditions (without levee)	Future Conditions (with levee)	Change
1029.780	175.21	175.20	-0.01
1005.961	175.18	175.18	0
964.9745	175.13	175.13	0
917.2293	175.11	175.11	0
863.8885	175.07	175.07	0

As shown in the above table, there is no measurable impact on the existing 100 year floodplain elevations resulting from the construction of the proposed SWM facilities. The 0.01m decrease at cross section 1029.780 is likely due to internal rounding and is considered within the margin of error associated to the model. Therefore, the proposed wet pond facility can be permitted to be constructed within the existing 100 year floodplain extent without negatively impacting neighbouring or upstream properties.

The existing and future HEC-RAS cross sections summarized above have been enclosed in Appendix D and E for reference.



UPPER CANADA CONSULTANTS
ENGINEERS / PLANNERS

WESTWOOD ESTATES (PHASE 3)

CITY OF PORT COLBORNE

HEC-RAS CROSS SECTION LOCATIONS

DATE	2022-12-22
SCALE	1:1000 m
REF No.	2160
DWG No.	FIGURE 5

DRAWING FILE: F:\2160\SWM\2160 Base.dwg PLOTTED: Jan 26, 2023 – 3:36pm PLOTTED BY: brendan

6.0 SEDIMENT CONTROL

Sediment controls are required during construction. The proposed extended detention facility can be used for this purpose. Therefore, the proposed constructed wet pond facility should be constructed prior to the facility for sediment control during construction.

The following additional erosion and sediment controls will also be implemented during construction:

- Install silt control fencing along the limits of construction where overland flows will flow beyond the limits of the development or into downstream watercourse.
- Re-vegetate disturbed areas as soon as possible after grading works have been completed.
- Lot grading and siltation controls plans will be provided with sediment and erosion control measures to the appropriate agencies for approval during the final design stage.
- The Stormwater management facility be cleaned after construction prior to assumption by municipality.

7.0 STORMWATER MANAGEMENT FACILITY MAINTENANCE

Maintenance is a necessary and important aspect of urban stormwater quality and quantity measures such as constructed wetlands. Many pollutants (i.e. nutrients, metals, bacteria, etc.) bind to sediment and therefore removal of sediment on a scheduled basis is required.

The wet pond for this development is subject to frequent wetting and deposition of sediments as a result of frequent low intensity storm event. The purpose of the wet pond is to improve post development sediment and contaminant loadings by detaining the 'first flush' flow for a 24 hour period. For the initial operation period of the stormwater management facility, the required frequency of maintenance is not definitively known and many of the maintenance tasks will be performed on an 'as required' basis. For example, during the home construction phase of the development there will be a greater potential for increased maintenance frequency, which depends on the effectiveness of sediment and erosion control techniques employed.

Inspections of the wet pond will indicate whether or not maintenance is required. Inspections should be made after every significant storm during the first two years of operation or until all development is completed to ensure the wet pond is functioning properly. This may translate into an average of six inspections per year. Once all building activity is finalized, inspections shall be performed annually. The following points should be addressed during inspections of the facility.

a) Standing water above the inlet storm sewer invert a day or more after a storm may indicate a blockage in the reverse slope pipe or orifice. The blockage may be caused by trash or sediment and a visual inspection would be required to determine the cause.

b) The vegetation around the wet pond should be inspected to ensure its function and aesthetics. Visual inspections will indicate whether replacement of plantings are required. A decline in vegetation habitat may indicate that other aspects of the constructed wet pond are operating improperly, such as the detention times may be inadequate or excessive.

c) The accumulation of sediment and debris at the wet pond inlet sediment forebay or around the high water line of the wet pond should be inspected. This will indicate the need for sediment removal or debris clean up.

d) The wet pond has been created by excavating a detention area. The integrity of the embankments should be periodically checked to ensure that it remains watertight and the side slopes have not sloughed.

Grass cutting is a maintenance activity that is done solely for aesthetic purposes. It is recommended that grass cutting be eliminated. It should be noted that municipal by-laws may require regular grass maintenance for weed control.

Trash removal is an integral part of maintenance and an annual clean-up, usually in the spring, is a minimum requirement. After this, trash removal is performed as required basis on observation of trash build-up during inspections.

To ensure long term effectiveness, the sediment that accumulates in the forebay area should be removed periodically to ensure that sediment in not deposited throughout the facility. For sediment removal operations, typical grading/excavating equipment should be used to remove sediment from the inlet forebay and detention areas. Care should be taken to ensure that limited damage occurs to existing vegetation and habitat.

Generally, the sediment which is removed from the detention pond will not be contaminated to the point that it would be classified as hazardous waste. However, the sediment should be tested to determine the disposal options.

8.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the findings of this study, the following conclusions are offered:

- Infiltration techniques are not suitable for this site as the primary control facility due to the low soil infiltration rates.
- Two proposed stormwater management facilities wet pond facilities will provide stormwater quality control, quantity control and erosion controls to the proposed development.
- Various lot level vegetative stormwater management practices can be implemented to enhance stormwater quality.
- This report was prepared in accordance with the provincial guidelines contained in "Stormwater Management Planning and Design Manual, March 2003".

The above conclusions lead to the following recommendations:

- That the stormwater management criteria established in this report be accepted.
- That two stormwater management wet pond facilities be constructed to provide stormwater quality protection to MECP *Enhanced* Protection levels.
- That additional lot level controls and vegetative stormwater management practices as described previously in this report be implemented.
- That the sediment during construction as described in this report be implemented.

Respectfully Submitted,

Brendan Kapteyn, P.Eng.

APPENDICES

APPENDIX A

Stormwater Management Facility Calculations (A1)

Upper Canada Consultants
3-30 Hannover Drive
St. Catharines, ON, L2W 1A3
PROJECT NAME: **Westwood Estates (Phase 3)**
PROJECT NO.: **2160**

PROPOSED WET POND 'A1' CALCULATIONS

Quality Requirements		Quality Orifice		Outlet Weir		Overflow Spillway		Outflow Pipe Orifice	
Drainage Area (ha) =	6.03	Diameter (m) =	0.100	Perimeter Length (m) =	0.60	Length (m) =	2.50	Diameter (m) =	0.450
Level 1 (m^3/ha) =	213 @ 65%	Cd =	0.63	Inlet Elevation (m) =	176.00	Slopes (X:1) =	3.00	Cd =	0.65
Perm Pool (m^3/ha) =	173	Invert (m) =	175.50			Invert (m) =	176.70	Invert (m) =	175.50
Perm Pool Vol (m^3) =	1,043							Obvert (m) =	175.95
Active Vol (m^3)	241							Top of Pipe (m) =	176.05
Total Quality Volume =	1,284								
Water Level Elev. =	175.50 m								

Pond Drawdown Time Calculation (MOE, 2003)

MOE Equation 4.11 Drawdown Coefficient 'C2' = 919
MOE Equation 4.11 Drawdown Coefficient 'C3' = 2,113
MOE Equation 4.11 Drawdown Time (h) = **41**

Elevation	Increment Depth (m)	Active Depth (m)	Surface Area (m^2)	Average Surface Area (m^2)	Increment Volume (m^3)	Permanent Volume (m^3)	Active Volume (m^3)	Quality Orifice (m^3/s)	Ditch Inlet (m^3/s)	Max Pipe Orifice (m^3/s)	Overflow Spillway (m^3/s)	Total Outflow (m^3/s)	Average Discharge (m^3/s)
174.50		-1.00	1,012			0							
	0.50			1,181	591								
175.00		-0.50	1,351			591							
	0.50			1,539	770								
175.50		**0.00**	**1,728**			**1,360**							
175.50		0.00	2,113				0	0.000	0.000	0.000	0.000	0.000	
	0.50			2,343	1,171								0.014
176.00		0.50	2,573				1,171	0.014	0.000	0.205	0.000	0.014	
	0.50			2,822	1,411								0.199
176.50		1.00	3,071				2,582	0.021	0.362	0.383	0.000	0.383	
	0.20			3,230	646								0.409
176.70		1.20	3,389				3,228	0.023	0.599	0.434	0.000	0.434	
	0.30			3,500	1,050								0.911
177.00		1.50	3,610				4,278	0.026	1.023	0.502	0.886	1.388	

Notes

1. Quality Orifice flow is the orifice controlling for the 24 hour detention period and uses an orifice formula.
2. Pipe Orifice flow is calcuated using an orifice formula on the pipe from the ditch inlet to the outlet and uses the total head on the orifice.
3. Overflow Weir flow is calculated using a trapezondial weir to convey outflow for less frequent storms through the embankment with an emergency spillway.
4. Total Outflow is calculated by adding the Overflow Spillway with the lowest of Quality Orifice plus Ditch Inlet or Max Pipe Orifice.

Stormwater Management Facility Forebay Sizing (A1)

a) Forebay Settling Length (MOE SWMP&D, Equation 4.5)

$$Settling\ Length = \sqrt{\left(\frac{r \times Q}{V_s}\right)}$$

r =	10.0	:1	(Length:Width Ratio)
Q_p =	0.009	m^3/s	(25mm Storm Pond Discharge)
V_s =	0.0003	m/s	(Settling Velocity)

Settling Length = **17.32** **m**

b) Dispersion Length (MOE SWMP&D, Equation 4.6)

$$Dispersion\ Length = \frac{8 \times Q}{D \times V_f}$$

Q =	0.671	m^3/s	(5 Yr Stm Sew Design Inflow)
D =	1.00	m	(Depth of Forebay)
V_f =	0.5	m/s	(Desired Velocity)

Dispersion Length = **10.74** **m**

c) Minimum Forebay Deep Zone Bottom Width (MOE SWMP&D), Equation 4.7)

$$Width = \frac{Min. Forebay\ Length}{8}$$

17.32 m DI (minimum required length)

Width = **2.17** **m** (minimum required width)

d) Average Velocity of Flow

$$Average\ Velocity = \frac{Q}{A}$$

Q =	0.369	m^3/s	(25mm Storm Design Inflow)
A =	6.00	m^2	(Cross Sectional Area)
D =	1.00	m	(Depth of Forebay)
W =	3.00	m	(Proposed Bottom Width)
SS =	3	:1	(Side Slopes - Minimum)

Average Velocity = **0.06** **m/s**

Is this Acceptable? **Yes** (Maximum velocity of flow = 0.15 m/s)

e) Cleanout Frequency

Is this Acceptable? **Yes**

L =	30.0	m	(Proposed Bottom Length)
ASL =	2.5	m^3/ha	(Annual Sediment Loading)
A =	6.03	ha	(Drainage Area)
FRC =	80	%	(Facility Removal Efficiency)
FV =	207.0	m^3	(Forebay Volume)

Cleanout Frequency = **11.0** **Years**

Is this Acceptable? **Yes** (10 Year Minimum Cleanout Frequency)

APPENDIX B

Stormwater Management Facility Calculations (A2)

Upper Canada Consultants
3-30 Hannover Drive
St. Catharines, ON, L2W 1A3
PROJECT NAME: **Westwood Estates (Phase 3)**
PROJECT NO.: **2160**

PROPOSED WET POND 'A2' CALCULATIONS

Quality Requirements

Drainage Area (ha) = 19.09
Level 1 (m3/ha) = 153 @ 40%
Perm Pool (m3/ha) = 113
Perm Pool Vol (m3) = 2,157
Active Vol (m3) 764
Total Quality Volume = 2,921
Water Level Elev. = 175.50 m

Quality Orifice

Diameter (m) = 0.135
Cd = 0.63
Invert (m) = 175.50

Outlet Weir

Perimeter Length (m) = 0.60
Inlet Elevation (m) = 176.00

Overflow Spillway

Length (m) = 2.50
Slopes (X:1) = 3.00
Invert (m) = 176.70

Outflow Pipe Orifice

Diameter (m) = 0.450
Cd = 0.65
Invert (m) = 175.50
Obvert (m) = 175.95
Top of Pipe (m) = 176.05

Pond Drawdown Time Calculation (MOE, 2003)

MOE Equation 4.11 Drawdown Coefficient 'C2' = 1,247
MOE Equation 4.11 Drawdown Coefficient 'C3' = 2,935
MOE Equation 4.11 Drawdown Time (h) = **31**

Elevation	Increment Depth (m)	Active Depth (m)	Surface Area (m^2)	Average Surface Area (m^2)	Increment Volume (m^3)	Permanent Volume (m^3)	Active Volume (m^3)	Quality Orifice (m^3/s)	Ditch Inlet (m^3/s)	Max Pipe Orifice (m^3/s)	Overflow Spillway (m^3/s)	Total Outflow (m^3/s)	Average Discharge (m^3/s)
174.00		-1.50	1,094			0							
	0.50			1,277	638								
174.50		-1.00	1,459			638							
	0.50			1,661	830								
175.00		-0.50	1,863			1,469							
	0.50			2,113	1,057								
175.50		**0.00**	**2,364**			**2,526**							
175.50		0.00	2,935				0	0.000	0.000	0.000	0.000	0.000	
	0.50			3,246	1,623								0.000
176.00		0.50	3,558				1,623	0.026	0.000	0.205	0.000	0.026	
	0.50			3,889	1,945								0.204
176.50		1.00	4,220				3,568	0.038	0.362	0.383	0.000	0.383	
	0.20			4,429	886								0.409
176.70		1.20	4,637				4,453	0.042	0.599	0.434	0.000	0.434	
	0.30			4,779	1,434								0.911
177.00		1.50	4,922				5,887	0.047	1.023	0.502	0.886	1.388	

Notes

1. Quality Orifice flow is the orifice controlling for the 24 hour detention period and uses an orifice formula.
2. Pipe Orifice flow is calcuated using an orifice formula on the pipe from the ditch inlet to the outlet and uses the total head on the orifice.
3. Overflow Weir flow is calculated using a trapezondial weir to convey outflow for less frequent storms through the embankment with an emergency spillway.
4. Total Outflow is calculated by adding the Overflow Spillway with the lowest of Quality Orifice plus Ditch Inlet or Max Pipe Orifice.

Table 9. Stormwater Management Facility Forebay Sizing (A2)

a) Forebay Settling Length (MOE SWMP&D, Equation 4.5)

$$Settling\ Length = \sqrt{\left(\frac{r \times Q}{V_s}\right)}$$

r =	5.4	:1	(Length:Width Ratio)
Q_p =	0.022	m^3/s	(25mm Storm Pond Discharge)
V_s =	0.0003	m/s	(Settling Velocity)

Settling Length = **19.98** **m**

b) Dispersion Length (MOE SWMP&D, Equation 4.6)

$$Dispersion\ Length = \frac{8 \times Q}{D \times V_f}$$

Q =	1.346	m^3/s	(5 Yr Stm Sew Design Inflow)
D =	1.00	m	(Depth of Forebay)
V_f =	0.5	m/s	(Desired Velocity)

Dispersion Length = **21.54** **m**

c) Minimum Forebay Deep Zone Bottom Width (MOE SWMP&D), Equation 4.7)

$$Width = \frac{Min. Forebay\ Length}{8}$$

21.54 m DI (minimum required length)

Width = **2.69** **m** (minimum required width)

d) Average Velocity of Flow

$$Average\ Velocity = \frac{Q}{A}$$

Q =	0.670	m^3/s	(25mm Storm Design Inflow)
A =	7.50	m^2	(Cross Sectional Area)
D =	1.00	m	(Depth of Forebay)
W =	4.50	m	(Proposed Bottom Width)
SS =	3	:1	(Side Slopes - Minimum)

Average Velocity = **0.09** **m/s**

Is this Acceptable? **Yes** (Maximum velocity of flow = 0.15 m/s)

e) Cleanout Frequency

Is this Acceptable? **Yes**

L =	24.5	m	(Proposed Bottom Length)
ASL =	0.9	m^3/ha	(Annual Sediment Loading)
A =	19.09	ha	(Drainage Area)
FRC =	80	%	(Facility Removal Efficiency)
FV =	215.3	m^3	(Forebay Volume)

Cleanout Frequency = **10.0** **Years**

Is this Acceptable? **Yes** (10 Year Minimum Cleanout Frequency)

APPENDIX C

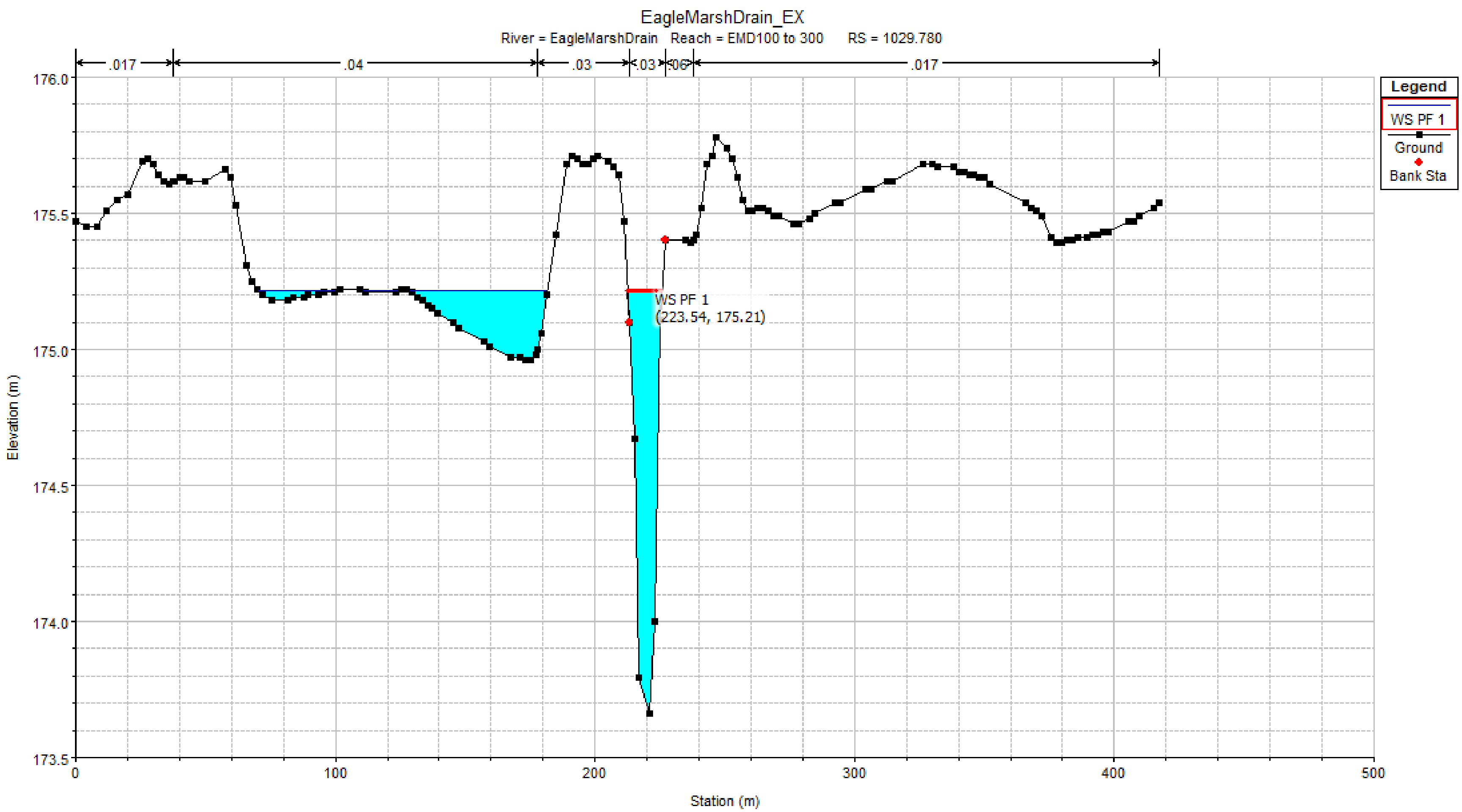
MIDUSS Output Files

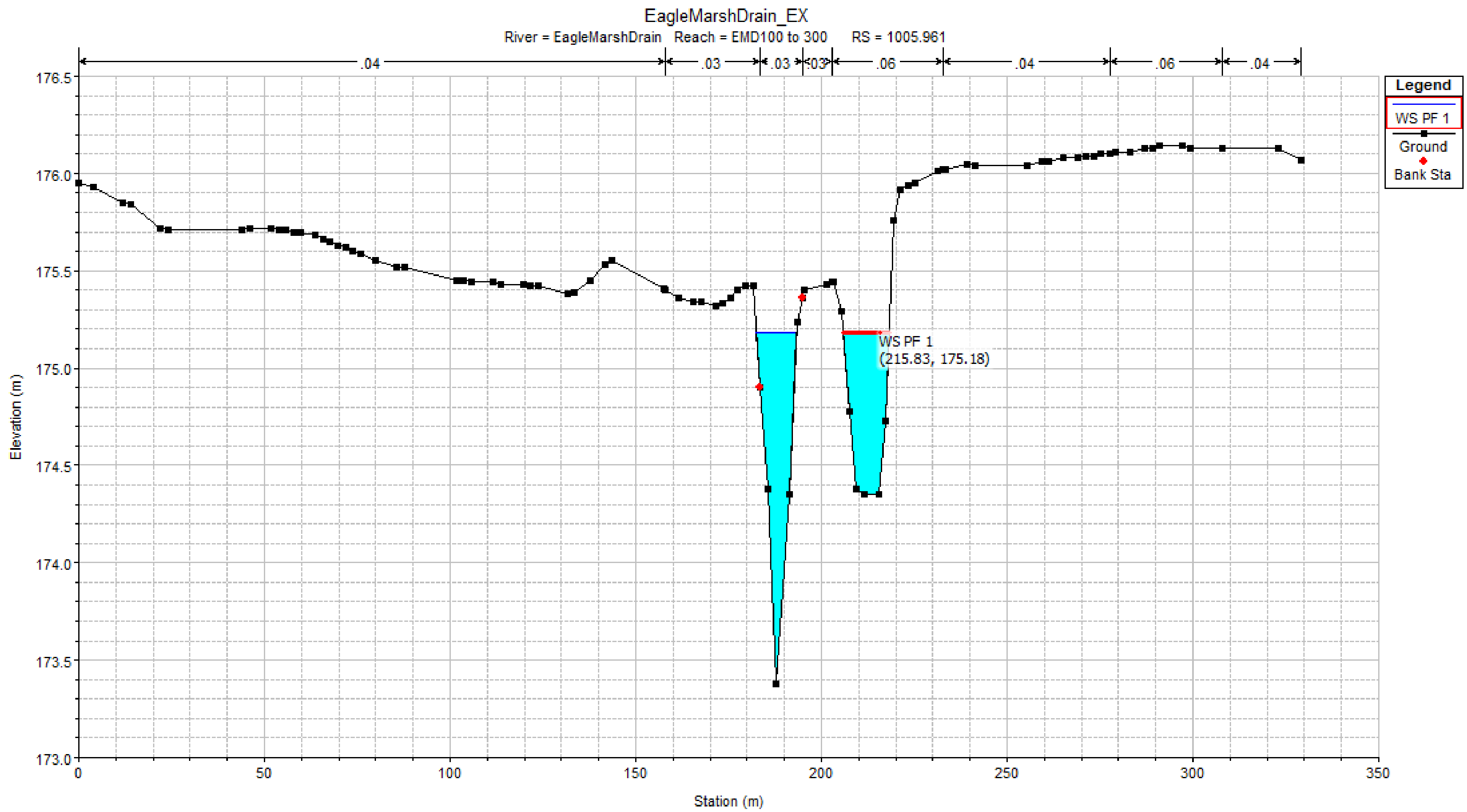
```
        Output File (4.7) SWM.OUT      opened 2022-12-22  13:55
        Units used are defined by G =    9.810
            24   144    10.000        are MAXDT MAXHYD & DTMIN values
        Licensee: UPPER CANADA CONSULTANTS
 35     COMMENT
       2     line(s) of comment
        WESTWOOD PHASE 3, CITY OF PORT COLBORNE
        STORMWATER MANAGEMENT PLAN
 35     COMMENT
       3     line(s) of comment
        **********************************
        ** 25mm MECP DESIGN STORM EVENT **
        **********************************
  2     STORM
            1     1=Chicago;2=Huff;3=User;4=Cdn1hr;5=Historic
      512.000     Coefficient  a
        6.000     Constant  b    (min)
         .800     Exponent  c
         .450     Fraction to peak  r
      210.000     Duration ó  240 min
               24.309 mm     Total depth
  3     IMPERVIOUS
            1     Option 1=SCS CN/C; 2=Horton; 3=Green-Ampt; 4=Repeat
         .015     Manning "n"
       98.000     SCS Curve No or C
         .100     Ia/S Coefficient
         .518     Initial Abstraction
 35     COMMENT
       3     line(s) of comment
        *******************************
        ** FROM SWM POND 1 TO OUTLET **
        *******************************
  4     CATCHMENT
        1.000     ID No.ó 99999
        6.030     Area in hectares
      200.000     Length (PERV) metres
        1.000     Gradient (%)
       65.000     Per cent Impervious
      200.000     Length (IMPERV)
         .000     %Imp. with Zero Dpth
            1     Option 1=SCS CN/C; 2=Horton; 3=Green-Ampt; 4=Repeat
         .250     Manning "n"
       77.000     SCS Curve No or C
         .100     Ia/S Coefficient
        7.587     Initial Abstraction
            1     Option 1=Trianglr; 2=Rectanglr; 3=SWM HYD; 4=Lin. Reserv
               .369       .000       .000       .000 c.m/s
               .124       .801       .564     C perv/imperv/total
 15     ADD RUNOFF
               .369       .369       .000       .000 c.m/s
 27     HYDROGRAPH DISPLAY
       4     is # of Hyeto/Hydrograph chosen
        Volume  =  .8249190E+03 c.m
 10     POND
       5 Depth - Discharge - Volume sets
        175.500        .000          .0
        176.250       .0290      1435.0
        176.500        .297      2583.0
        176.700        .434      3229.0
        177.000       1.388      4279.0
        Peak Outflow    =      .014 c.m/s
        Maximum Depth   =   175.854 metres
        Maximum Storage =      677. c.m
               .369       .369       .014       .000 c.m/s
 14     START
       1     1=Zero; 2=Define
 35     COMMENT
       3     line(s) of comment
        *******************************
        ** FROM SWM POND 2 TO OUTLET **
        *******************************
  4     CATCHMENT
        3.000     ID No.ó 99999
       19.090     Area in hectares
      357.000     Length (PERV) metres
        1.000     Gradient (%)
       40.000     Per cent Impervious
      357.000     Length (IMPERV)
         .000     %Imp. with Zero Dpth
            1     Option 1=SCS CN/C; 2=Horton; 3=Green-Ampt; 4=Repeat
         .250     Manning "n"
       77.000     SCS Curve No or C
         .100     Ia/S Coefficient
        7.587     Initial Abstraction
            1     Option 1=Trianglr; 2=Rectanglr; 3=SWM HYD; 4=Lin. Reserv
               .670       .000       .014       .000 c.m/s
               .124       .802       .395     C perv/imperv/total
 15     ADD RUNOFF
               .670       .670       .014       .000 c.m/s
 27     HYDROGRAPH DISPLAY
       4     is # of Hyeto/Hydrograph chosen
        Volume  =  .1831176E+04 c.m
 10     POND
       5 Depth - Discharge - Volume sets
        175.500        .000          .0
        176.250       .0400      2557.0
        176.500        .175      3570.0
        176.700        .361      4456.0
        177.000       1.388      5890.0
        Peak Outflow    =      .022 c.m/s
        Maximum Depth   =   175.912 metres
        Maximum Storage =     1405. c.m
               .670       .670       .022       .000 c.m/s
 14     START
       1     1=Zero; 2=Define
```

```
 35     COMMENT
       3     line(s) of comment
        ***************************
        ** 5YR DESIGN STORM EVENT **
        ***************************
  2     STORM
            1     1=Chicago;2=Huff;3=User;4=Cdn1hr;5=Historic
      830.000     Coefficient  a
        7.300     Constant  b    (min)
         .777     Exponent  c
         .450     Fraction to peak  r
      240.000     Duration ó  240 min
               45.874 mm     Total depth
  3     IMPERVIOUS
            1     Option 1=SCS CN/C; 2=Horton; 3=Green-Ampt; 4=Repeat
         .015     Manning "n"
       98.000     SCS Curve No or C
         .100     Ia/S Coefficient
         .518     Initial Abstraction
 35     COMMENT
       3     line(s) of comment
        *******************************
        ** FROM SWM POND 1 TO OUTLET **
        *******************************
  4     CATCHMENT
        1.000     ID No.ó 99999
        6.030     Area in hectares
      200.000     Length (PERV) metres
        1.000     Gradient (%)
       65.000     Per cent Impervious
      200.000     Length (IMPERV)
         .000     %Imp. with Zero Dpth
            1     Option 1=SCS CN/C; 2=Horton; 3=Green-Ampt; 4=Repeat
         .250     Manning "n"
       77.000     SCS Curve No or C
         .100     Ia/S Coefficient
        7.587     Initial Abstraction
            1     Option 1=Trianglr; 2=Rectanglr; 3=SWM HYD; 4=Lin. Reserv
               .671       .000       .022       .000 c.m/s
               .280       .878       .669     C perv/imperv/total
 15     ADD RUNOFF
               .671       .671       .022       .000 c.m/s
 27     HYDROGRAPH DISPLAY
       4     is # of Hyeto/Hydrograph chosen
        Volume  =  .1850375E+04 c.m
 10     POND
       5 Depth - Discharge - Volume sets
        175.500        .000          .0
        176.250       .0290      1435.0
        176.500        .297      2583.0
        176.700        .434      3229.0
        177.000       1.388      4279.0
        Peak Outflow    =      .040 c.m/s
        Maximum Depth   =   176.260 metres
        Maximum Storage =     1482. c.m
               .671       .671       .040       .000 c.m/s
 14     START
       1     1=Zero; 2=Define
 35     COMMENT
       3     line(s) of comment
        *******************************
        ** FROM SWM POND 2 TO OUTLET **
        *******************************
  4     CATCHMENT
        3.000     ID No.ó 99999
       19.090     Area in hectares
      357.000     Length (PERV) metres
        1.000     Gradient (%)
       40.000     Per cent Impervious
      357.000     Length (IMPERV)
         .000     %Imp. with Zero Dpth
            1     Option 1=SCS CN/C; 2=Horton; 3=Green-Ampt; 4=Repeat
         .250     Manning "n"
       77.000     SCS Curve No or C
         .100     Ia/S Coefficient
        7.587     Initial Abstraction
            1     Option 1=Trianglr; 2=Rectanglr; 3=SWM HYD; 4=Lin. Reserv
              1.346       .000       .040       .000 c.m/s
               .280       .877       .519     C perv/imperv/total
 15     ADD RUNOFF
              1.346      1.346       .040       .000 c.m/s
 27     HYDROGRAPH DISPLAY
       4     is # of Hyeto/Hydrograph chosen
        Volume  =  .4542744E+04 c.m
 10     POND
       5 Depth - Discharge - Volume sets
        175.500        .000          .0
        176.250       .0400      2557.0
        176.500        .175      3570.0
        176.700        .361      4456.0
        177.000       1.388      5890.0
        Peak Outflow    =      .122 c.m/s
        Maximum Depth   =   176.401 metres
        Maximum Storage =     3170. c.m
              1.346      1.346       .122       .000 c.m/s
 14     START
       1     1=Zero; 2=Define
```

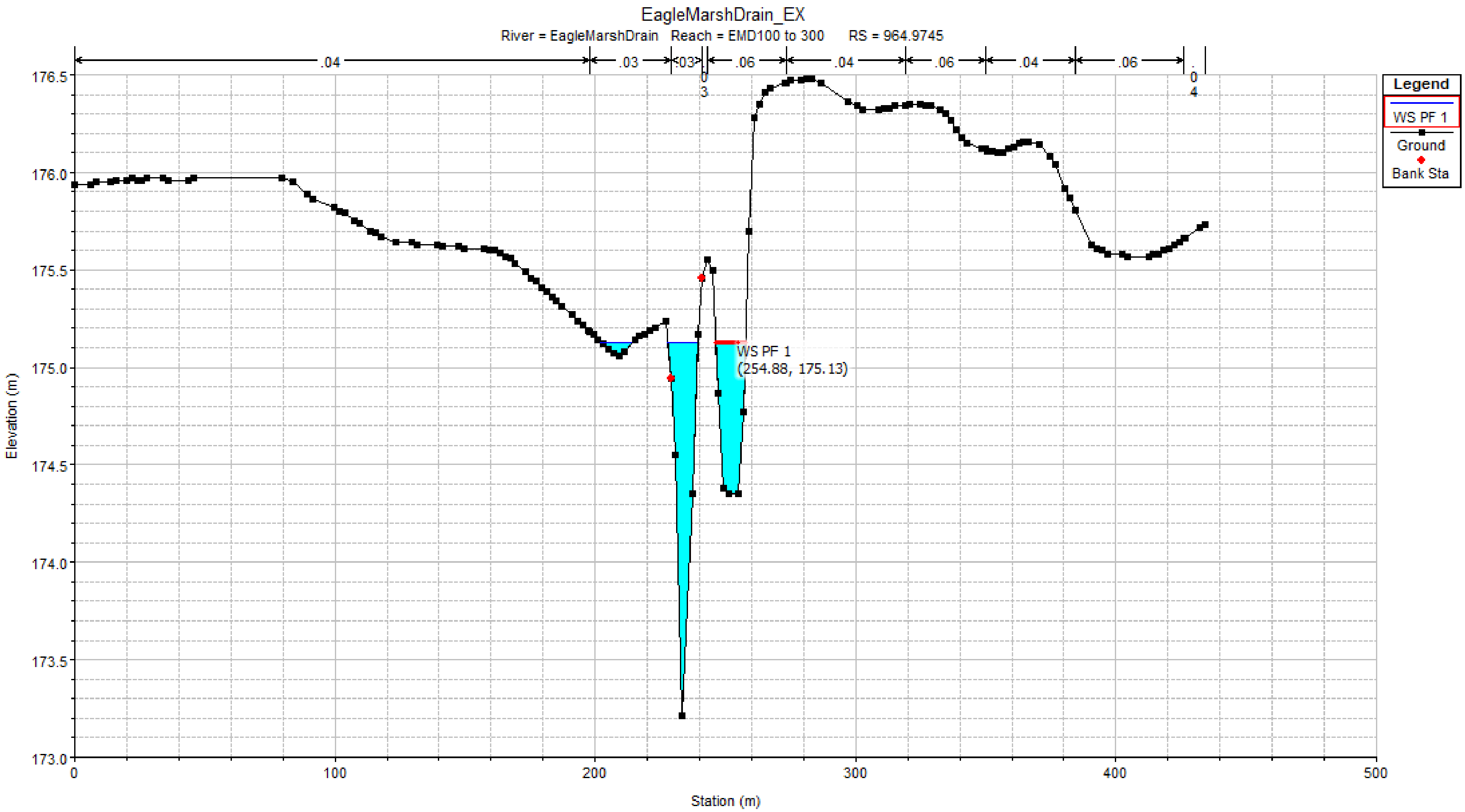
APPENDIX D

Existing HEC-RAS Cross Sections (without Levee)

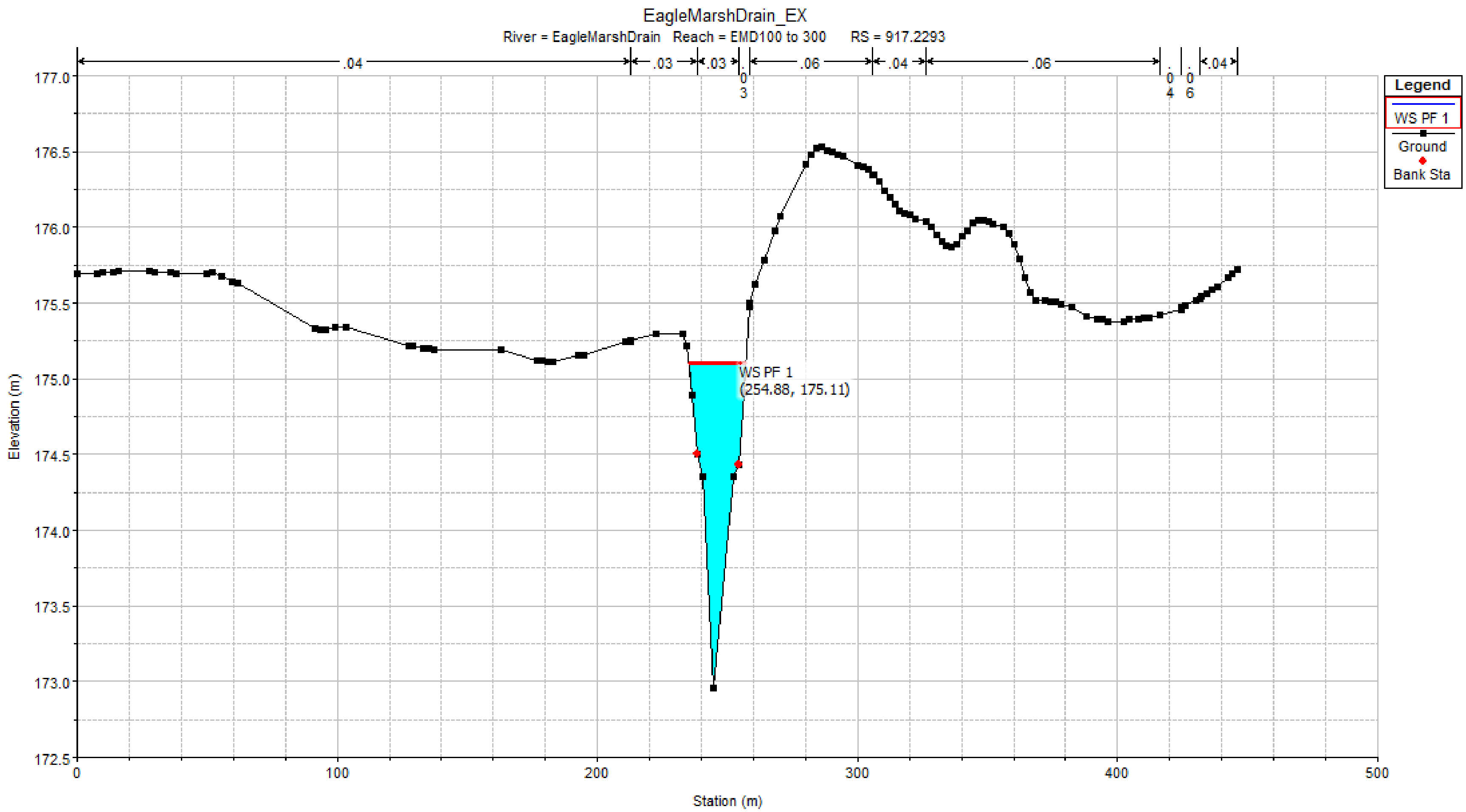

EagleMarshDrain_EX
River = EagleMarshDrain Reach = EMD100 to 300 RS = 1029.780
.017
.04
.03
.03
.06
.017
Legend
WS PF 1
Ground
Bank Sta
WS PF 1
(223.54, 175.21)
176.0
175.5
175.0
174.5
174.0
173.5
Elevation (m)
0
100
200
300
400
500
Station (m)

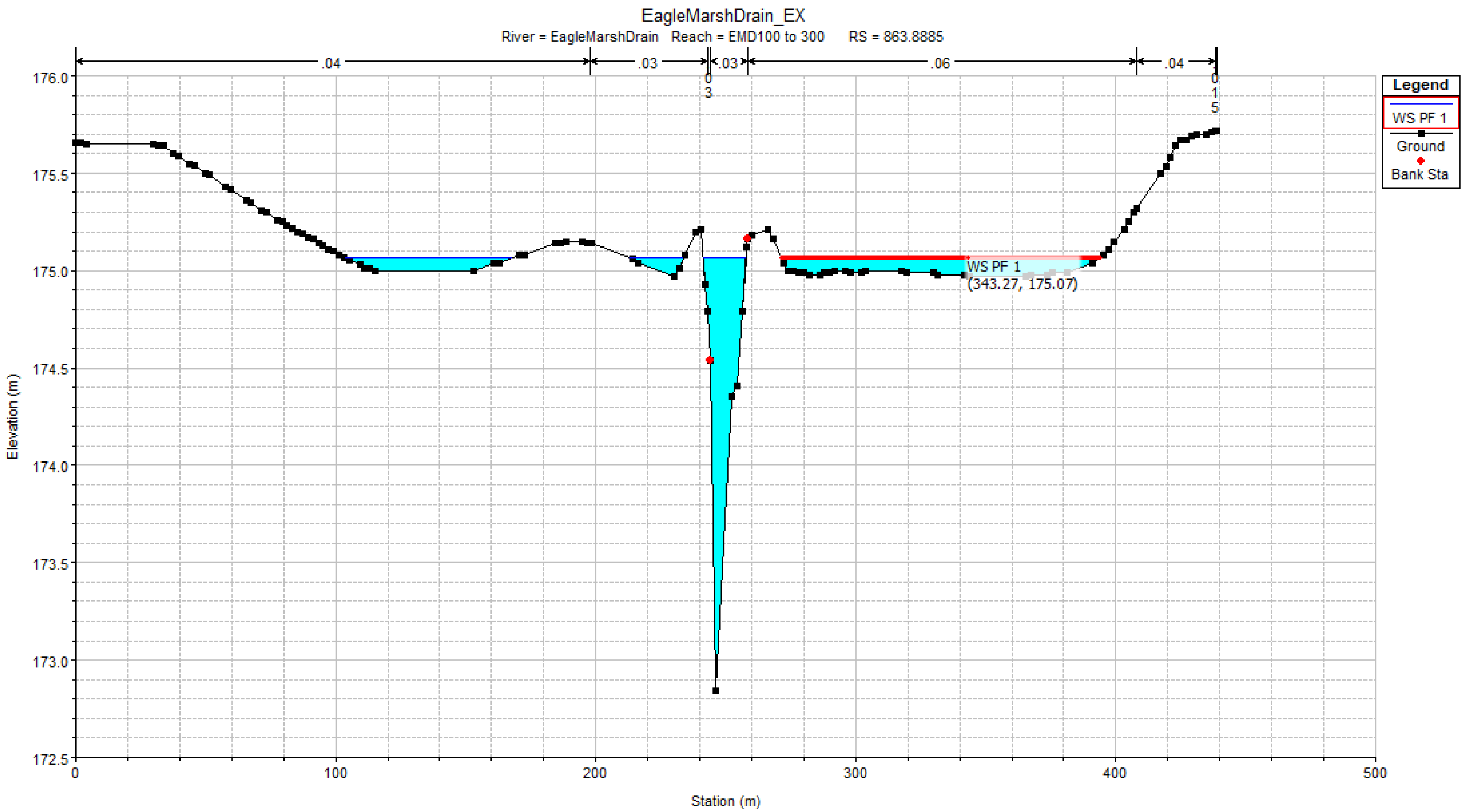


EagleMarshDrain_EX
River = EagleMarshDrain Reach = EMD100 to 300 RS = 1005.961
.04
.03
.03
.03
.06
.04
.06
.04
Legend
WS PF 1
Ground
Bank Sta
WS PF 1
(215.83, 175.18)
Elevation (m)
176.5
176.0
175.5
175.0
174.5
174.0
173.5
173.0
0
50
100
150
200
250
300
350
Station (m)



EagleMarshDrain_EX
River = EagleMarshDrain Reach = EMD100 to 300 RS = 964.9745
.04
.03
.03
0
3
.06
.04
.06
.04
.06
0
4
Legend
WS PF 1
Ground
Bank Sta
WS PF 1
(254.88, 175.13)
Elevation (m)
Station (m)
176.5
176.0
175.5
175.0
174.5
174.0
173.5
173.0
0
100
200
300
400
500

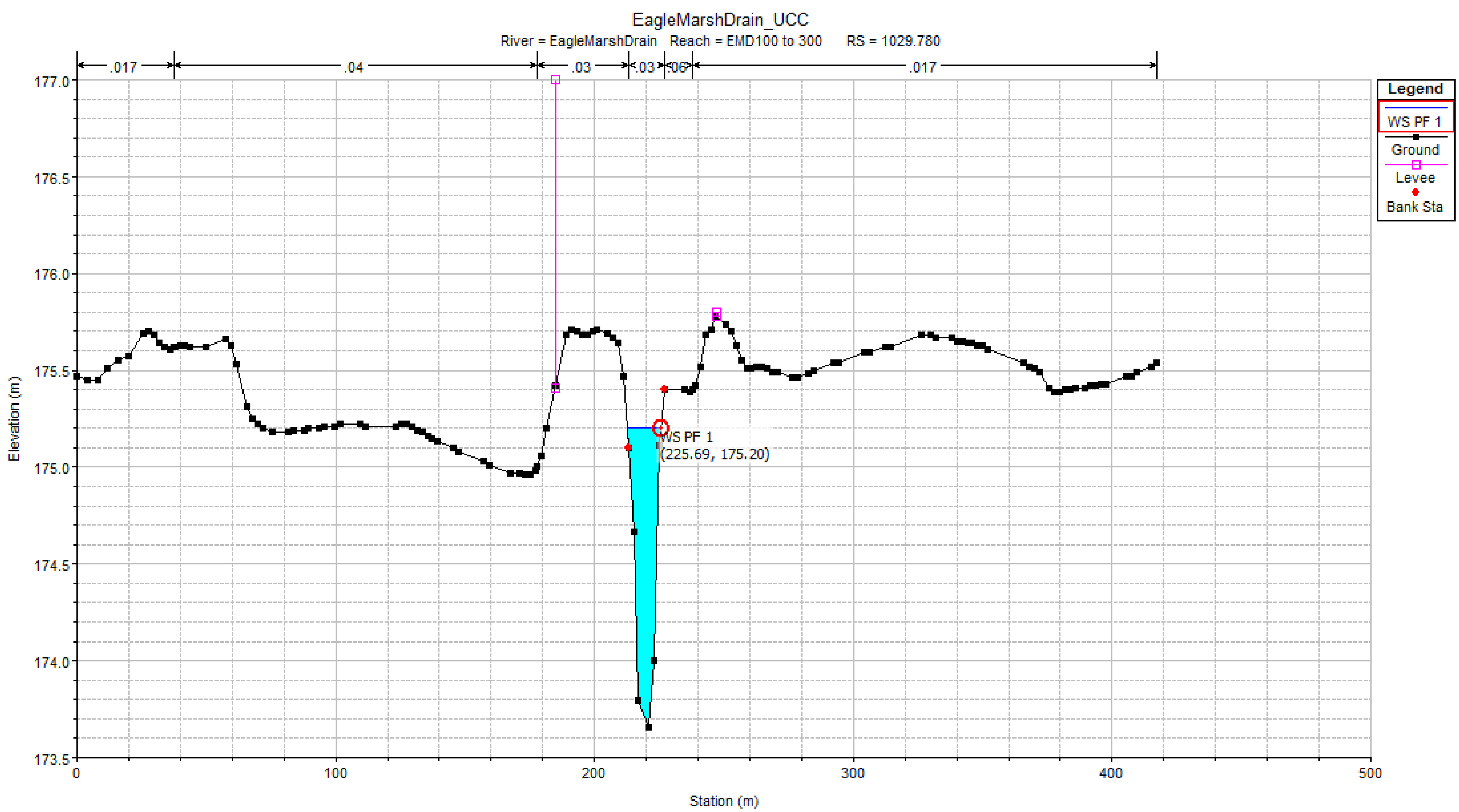

EagleMarshDrain_EX
River = EagleMarshDrain Reach = EMD100 to 300 RS = 917.2293
.04
.03
.03
.06
.04
.06
.04
Legend
WS PF 1
Ground
Bank Sta
WS PF 1
(254.88, 175.11)
Elevation (m)
Station (m)
177.0
176.5
176.0
175.5
175.0
174.5
174.0
173.5
173.0
172.5
0
100
200
300
400
500



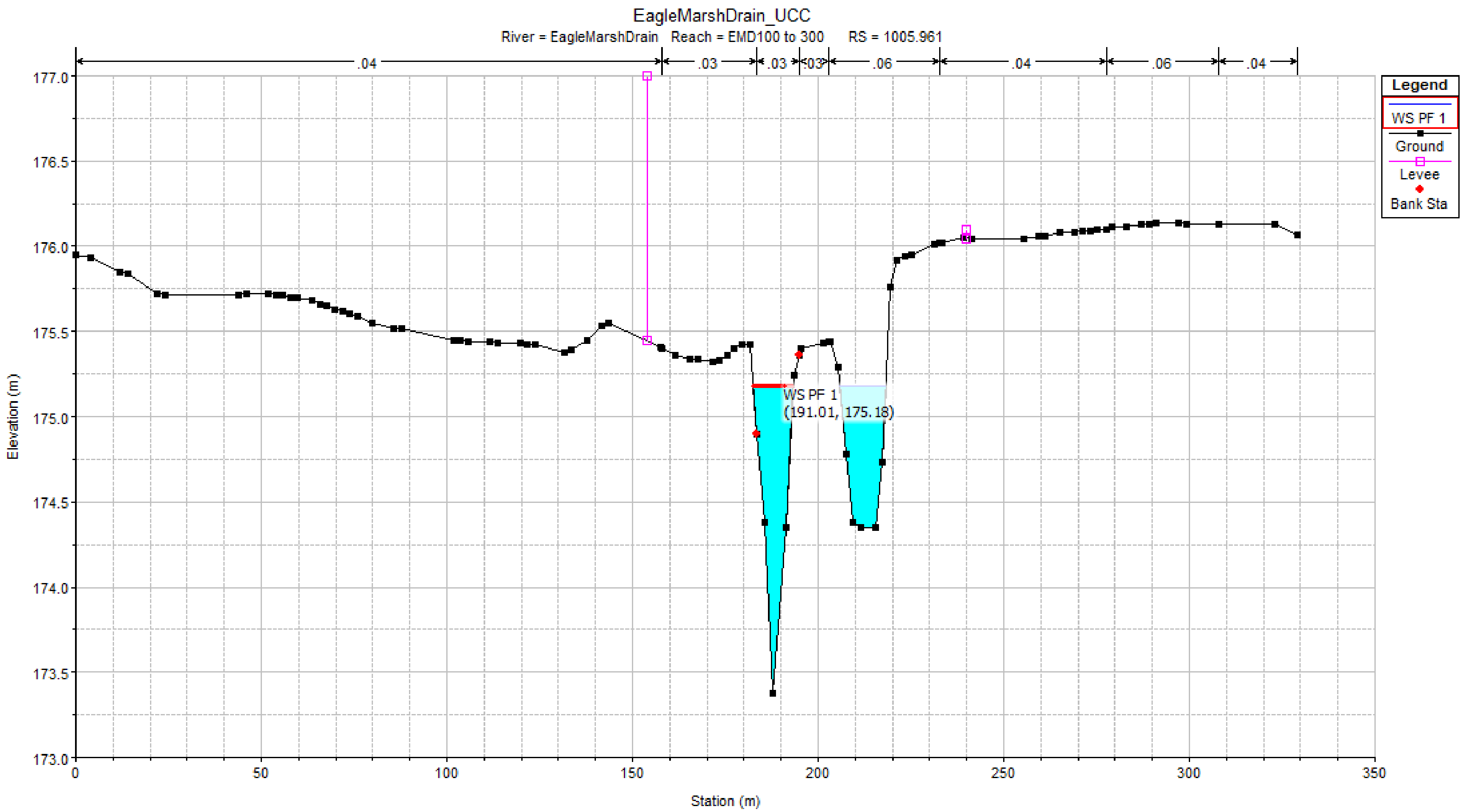
EagleMarshDrain_EX
River = EagleMarshDrain Reach = EMD100 to 300 RS = 863.8885
.04
.03
.03
.06
.04
Legend
WS PF 1
Ground
Bank Sta
WS PF 1
(343.27, 175.07)
Elevation (m)
Station (m)
176.0
175.5
175.0
174.5
174.0
173.5
173.0
172.5
0
100
200
300
400
500

APPENDIX E

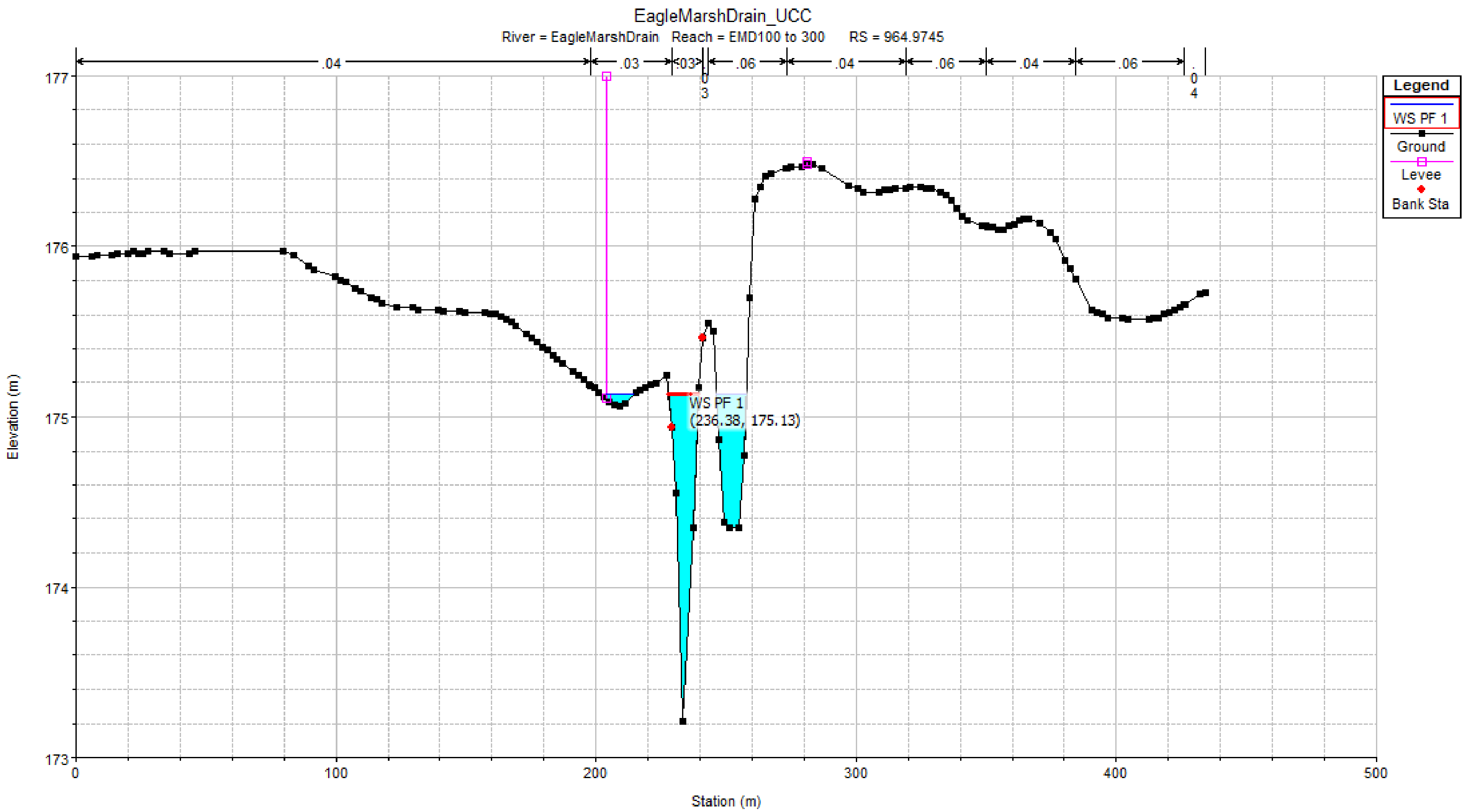
Future HEC-RAS Cross Sections (with Levee)



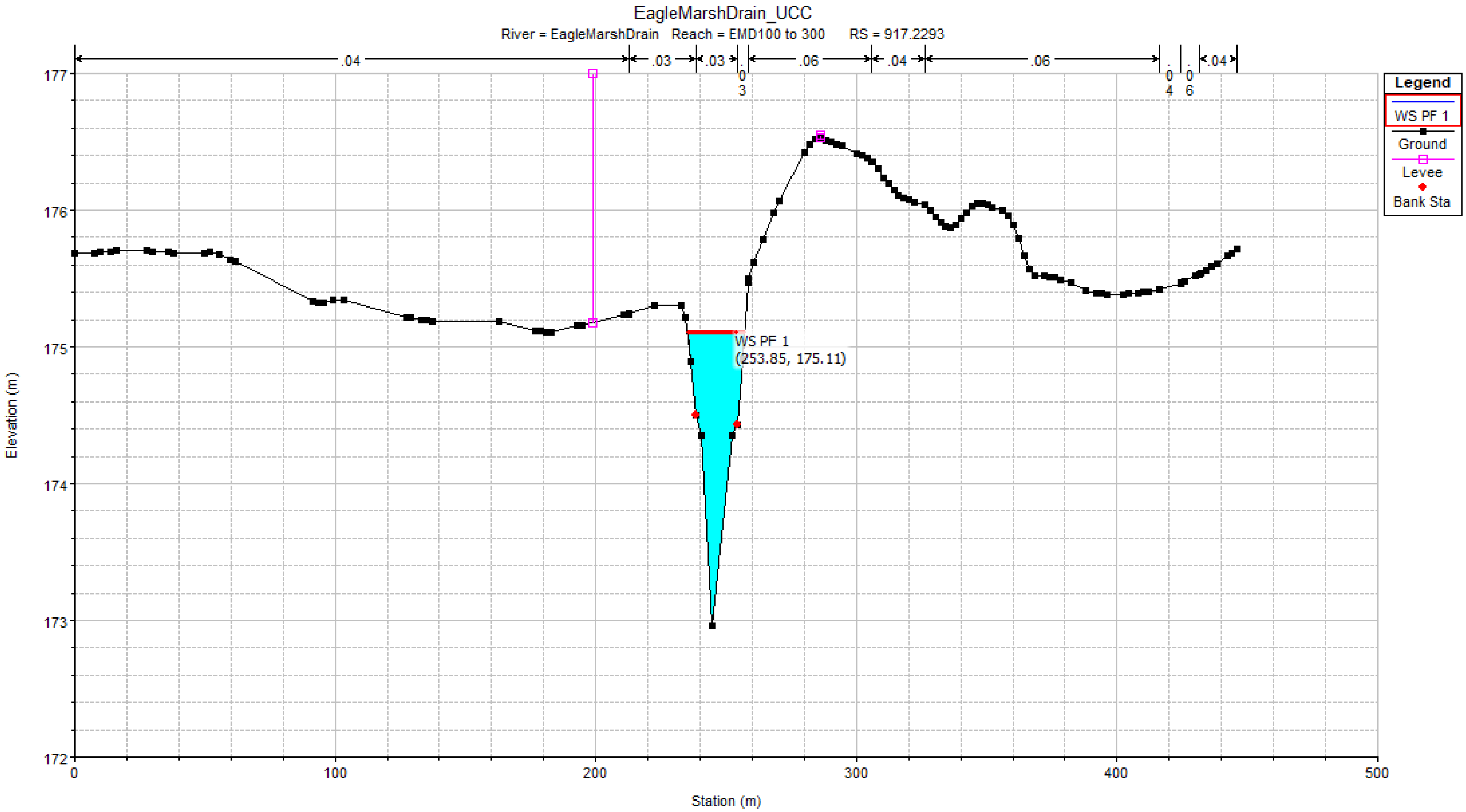
EagleMarshDrain_UCC
River = EagleMarshDrain   Reach = EMD100 to 300   RS = 1029.780
.017
.04
.03
.03
.06
.017
Legend
WS PF 1
Ground
Levee
Bank Sta
WS PF 1
(225.69, 175.20)
177.0
176.5
176.0
175.5
175.0
174.5
174.0
173.5
Elevation (m)
0
100
200
300
400
500
Station (m)



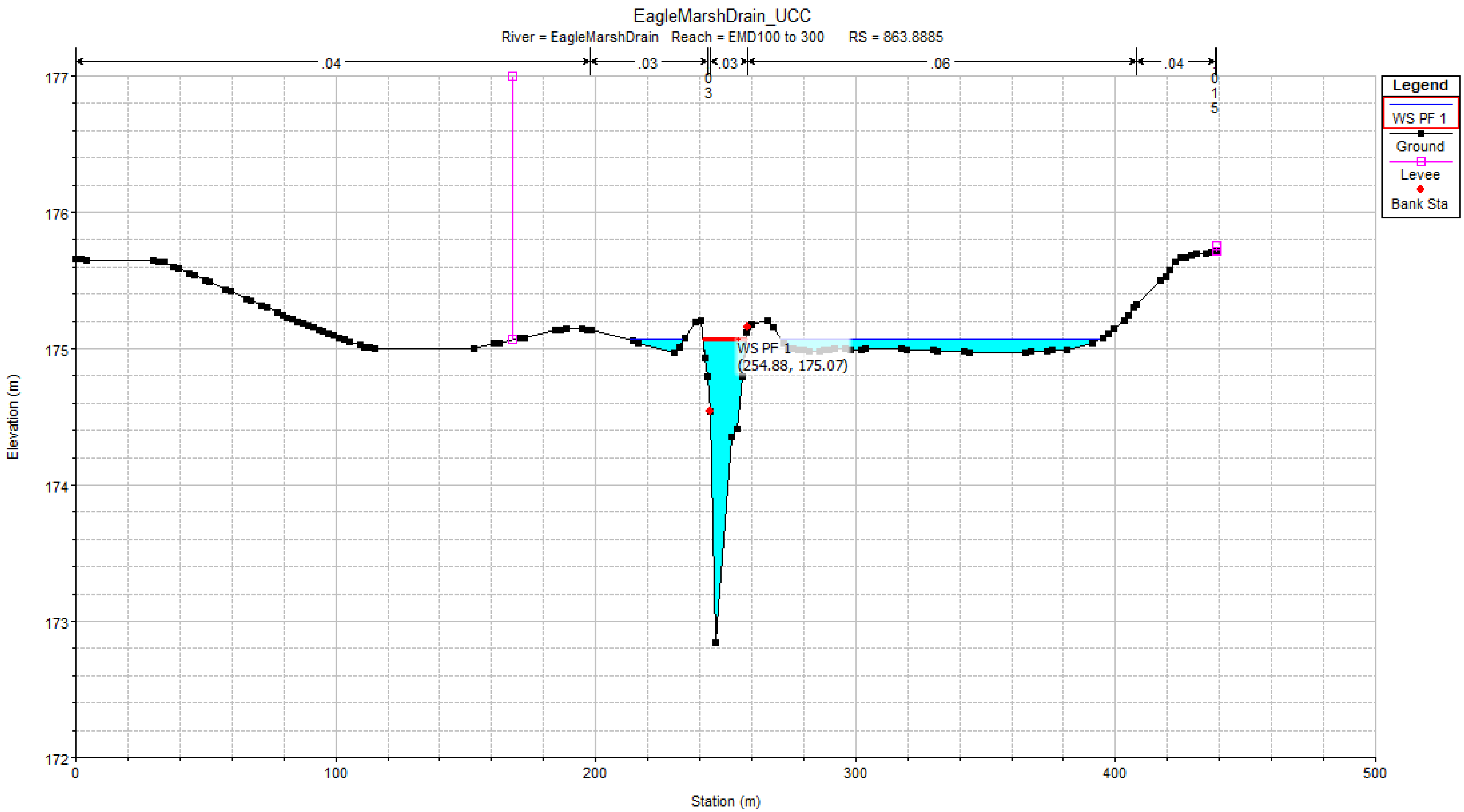
EagleMarshDrain_UCC
River = EagleMarshDrain Reach = EMD100 to 300 RS = 1005.961
.04
.03
.03
.03
.06
.04
.06
.04
Legend
WS PF 1
Ground
Levee
Bank Sta
WS PF 1
(191.01, 175.18)
Elevation (m)
Station (m)
177.0
176.5
176.0
175.5
175.0
174.5
174.0
173.5
173.0
0
50
100
150
200
250
300
350



EagleMarshDrain_UCC
River = EagleMarshDrain Reach = EMD100 to 300 RS = 964.9745
.04
.03
.03
.06
.04
.06
.04
.06
Legend
WS PF 1
Ground
Levee
Bank Sta
WS PF 1
(236.38, 175.13)
Elevation (m)
Station (m)
177
176
175
174
173
0
100
200
300
400
500



EagleMarshDrain_UCC
River = EagleMarshDrain Reach = EMD100 to 300 RS = 917.2293
.04
.03
.03
.06
.04
.06
.04
Legend
WS PF 1
Ground
Levee
Bank Sta
WS PF 1
(253.85, 175.11)
Elevation (m)
Station (m)
177
176
175
174
173
172
0
100
200
300
400
500



EagleMarshDrain_UCC
River = EagleMarshDrain Reach = EMD100 to 300 RS = 863.8885
.04
.03
.03
.06
.04
0
3
0
1
5
Legend
WS PF 1
Ground
Levee
Bank Sta
WS PF 1
(254.88, 175.07)
Elevation (m)
Station (m)
177
176
175
174
173
172
0
100
200
300
400
500