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PROPOSED RESIDENTIAL DEVELOPMENT
115 WATSON PARKWAY NORTH
GUELPH, ONTARIO

PROJECT No.: 22202

FUNCTIONAL SERVICING & STORMWATER MANAGEMENT REPORT

OWNER:

GUELPH WATSON HOLDINGS INC.

Prepared By:

THE ODAN/DETECH GROUP INC.

1st Submission –November 2023

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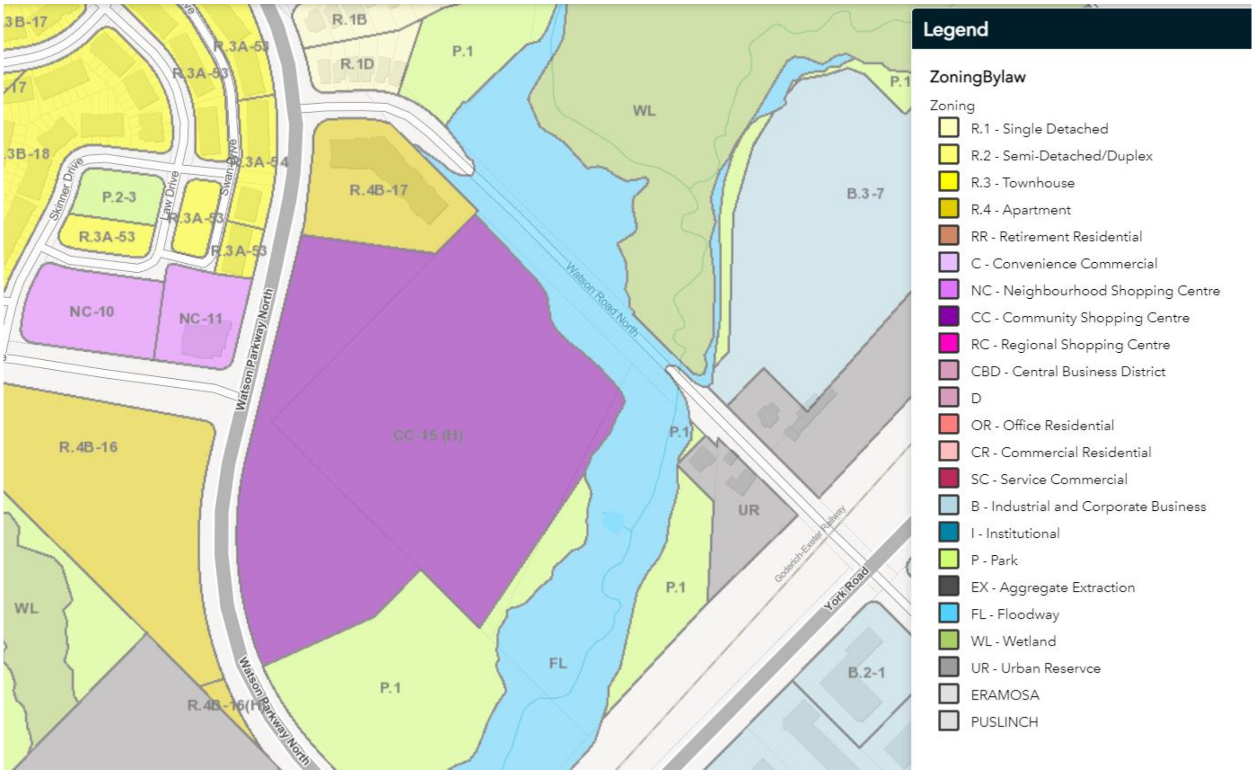
1.0 BACKGROUND

The property under study is approximately 6.44ha. (15.93ac.) site located at 115 Watson Parkway North in Guelph, ON. The site is bounded by the following streets and adjacent lands. The net site area excluding the Natural Heritage Feature is 5.98ha. (14.77ac.) and excluding the Natural Heritage Feature, Roads and Parks is 4.28ha. (10.57 ac.).

- North – Existing Residential abutting Watson Road North.
- South – Existing Storm Water Pond and Creek Valley Lands.
- East – Watson Road North
- West – Watson Parkway North

Presently the site is vacant with vegetation present and is currently zoned for Community Shopping Centre (CC-15 (H)) with a Holding provision.



Refer to the Aerial Photo of the Existing Site in **Appendix A** for additional details.

It is proposed to construct multiple midrise buildings and townhomes. The buildings will consist of 4 apartment buildings ranging between 10-14 storeys in height with commercial and amenity space at ground level and two levels of underground parking. The remainder of the development will consist of Townhomes with roadways and surface parking and landscaped areas. A parkland is proposed as part of the development. Refer to **Figure 1** below for further information regarding the proposed layout of the site.

In general, the property surface topography is higher at the north end and slopes gently towards the south to the creek and valley lands. For detailed topography of the existing site conditions, refer to **Appendix A** for the latest topographic survey.

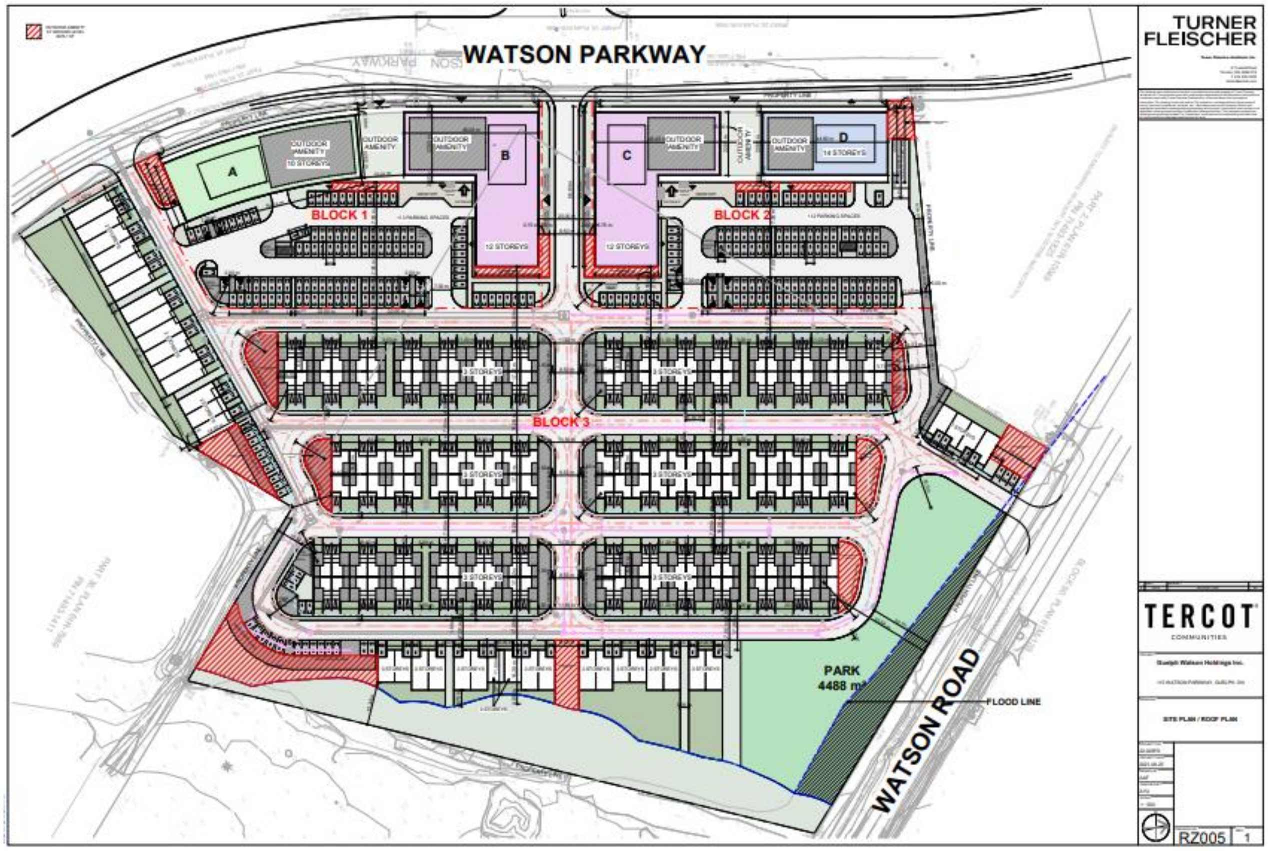


Figure 1 is an Excerpt from the Architectural Site Plan, prepared by Turner Fleischer Architects Inc. (TFAI) For detailed information regarding the layout of the proposed development, please refer to the latest drawings prepared by TFAI. For general existing site conditions see **Appendix A**.

2.0 SCOPE OF WORK

THE ODAN/DETECH GROUP INC. was retained by the owner, Guelph Watson Holdings Inc., to review the site, collect data, evaluate the site for the proposed land use and present the findings within this Engineering Report.

This report will evaluate the serviceability of the site with respect to sanitary, water and storm services and also evaluate the stormwater management (SWM) strategy that will be implemented to meet the City of Guelph design criteria and related studies including any other commenting agencies related to the noted scope.

3.0 WASTEWATER SERVICING

Existing Condition

There are existing sanitary sewer available on Watson Parkway North fronting the development with laterals provided to the proposed site as follows.

- There are three (3) 200mm dia. stubs sanitary sewer stubs provided to the property from the Watson Parkway N. that terminate at the property line and connect to the existing 525mm and existing 375mm dia. sewers fronting the site.
- The depth of the services are approximately 4-5m at the property line grade.

Proposed Condition

It is proposed to utilize the existing lateral services where possible to service the proposed buildings. A new service connection is proposed at the south end of the development. This new service will service the Townhome Block (Block 3) portion of the site. Blocks 1 & 2 will be serviced via the existing laterals where service capacity is provided. This will be reviewed further at the detailed design stage to determine if the existing laterals have sufficient capacity to service the proposed buildings or if upsizing or connection internally to the new laterals would be required.

For calculating the population for the development the following City standards for population densities and flow rates were used.

Apartments = 6 l/sec/ha. (150 u/ha)
Apartments = 7 l/sec/ha. (295 u/ha)
Townhomes = 2.5 l/sec/ha. (Schools/Townhomes)
Commercial = 1.7 l/sec/ha. (Commercial/Industrial)

A summary of the proposed land uses based on land area for the sanitary flows to Watson Parkway North are shown in Table 1 below.

Table 1 – Summary of Land Uses for Sanitary Flow Calculations

Land Use	Site Area (ha)	Flow Allocation (l/sec/ha)	Total Flow (l/sec)
*Apartments	1.79	7	12.53
Townhomes	3.75	2.5	9.38
Commercial	0.282	1.7	0.48
Total			**22.39**

The proposed site will have a peak flow of 22.39 L/s.

Wastewater Main Capacity and Allocation

In order to determine if capacity is available within the downstream wastewater sewer system discussions with City staff occurred in advance of the submission. For initial assessment a preliminary flow of 21.72 l/sec was provided to the City to apply to their current model to determine impacts to the downstream sewer. Based on the modelling results the preliminary flows were within acceptable parameters. The following is an excerpt of the correspondence between Odan/Detech and the City of Guelph confirming downstream available capacity as of July 6 2022.

Mark Harris - Odan Detech Group

From:	Mary Angelo <Mary.Angelo@guelph.ca>
Sent:	Wednesday, July 06, 2022 4:27 PM
To:	mark@odandetech.com; drago@odandetech.com
Cc:	Michelle Thalen
Subject:	RE: Master Servicing - Sanitary Allocation - 115 Watson Parkway -
Attachments:	RE: Starwood and Watson Site - near SWM Pond - Sanitary Tributary Plans - (217 KB)

Hi Mark and Drago,

The model analysis with the 21.72L/s shows that the existing system has capacity today to manage this flow. Please note that the sewer on York is very full so we would not anticipate it being able to accept any more than the 21.72L/s. Also remember that we are not committing this capacity to you today; other developments could be approved before you that use up the capacity.

...Mary

Mary Angelo, P.Eng. (she/her) Manager, Development and Environmental Engineering,
Engineering and Transportation Services,
City of Guelph,
519-822-1260, ext. 2287,
mary.angelo@guelph.ca

From: Mary Angelo [mailto:Mary.Angelo@guelph.ca]
Sent: Friday, May 27, 2022 3:20 PM
To: drago@odandetech.com; mark@odandetech.com
Cc: Michelle Thalen <Michelle.Thalen@guelph.ca>
Subject: RE: Master Servicing - Sanitary Allocation - 115 Watson Parkway -

As noted by the City of Guelph this is not a commitment to capacity. It is only an assessment of the downstream sewer capacity at the time of assessment.

The requested allocation for the development has increased as a result of the proposed density and therefore additional assessment will need to be completed by the City of Guelph. The assessment was completed using 21.72 l/sec and the revised flows based on the revised densities shows a minor increase to the assessment of 22.39 l/sec based on City of Guelph Design Criteria.

As this is only a minor increase in flows from the original assessment of 0.67 l/sec it is expected that the increase will result in similar downstream results to the previous assessment completed by the City of Guelph and still be within the capacity of the modelled sanitary sewer capacity with minor surcharge conditions as previously identified by the City of Guelph during the initial assessment.

Commercial Allocation

The development is currently zoned for Commercial. As such the site was has an approved allocation as follows;

Commercial = 1.7 l/sec/ha. (Commercial/Industrial)
The total site area for Commercial flows is based on the Total Site Area and therefore allocation will be as follows;

Total Allocated Wastewater Flows: 1.7 l/sec/ha. x 6.45 ha. = 10.97 l/sec.

As such that total increase in flow from the allocated Commercial development to the Residential development will be as follows;

Table 2.

Table 2 – Summary of Sanitary Flows from the Site

	Peak Flow (l/s)	Total Increase (l/s)
Proposed Residential	22.39	
Allocated Commercial	10.97	
Commercial to Residential Increase		11.42

The final layout and outlets for each of the proposed Blocks, 1 to 3, will be determined at the detailed design stage. A Conceptual Design has been provided in support of this application and is found in Appendix F.

It is proposed to service each Block separately as Block 1 & 2 front Watson Parkway N. and are Apartment blocks and Block 3 is comprised of Townhomes. Driveway access for all blocks will be via the internal road network.

4.0 WATER

Existing Condition

Existing water is available on Watson Parkway North. There is an existing 300mm diameter PVC watermain located on the west side of Watson Parkway North. Water has been provided to the property line from the 300mm dia. PVC water as 150mm water main lateral services, approximately four (4) services have been provided to the development. Hydrants are located on the west side across the street on Watson Parkway North. The City has provided water flow tests in order to determine flows and pressures within the Pressure District.

Proposed Condition

It is proposed to connect the site to the existing 300mm diameter watermain located on Waterson Parkway North for domestic and fire-fighting purposes. A new 300mm water service connecting to a private watermain is proposed service the development. The existing 150mm dia. laterals will not be sufficient to service the proposed site, however, these laterals will be assessed at the detailed design stage to determine if they will be sufficient to service the Apartments, Blocks 1 & 2.

In order to assess the existing water main the unit rate and peaking factors of water consumption, minimum pipe size and allowable pressure in line were established from the City of Guelph. The fire flow water demand is calculated as per FUS 2020 manual.

The pressures and volumes must be sufficient for peak hour conditions and under fire conditions as established by the Ontario Building Code 2006. The minimal residual pressure under fire conditions is 140 kpa. (or 20.3 psi).

The firefighting calculations are based on a fire resistive rating of a sprinklered building with protected steel.

Please refer to **Appendix C** for further details.

The water demand for the proposed site is calculated as follows:

In order to establish a population for water demand it is assumed that the following distribution will be applied to the apartment units.

1 Bedroom Units	1.4 person/unit	(655 units) (75%)
2 Bedroom Units	2.1 person/unit	(218 units) (25%)
Townhomes	2.7 person/unit	(197 units) (100%)
Commercial	1.1 person/100m2	(2820 m2) (100%)

The following provides for the flows required for the development based on the above distribution for the site.

Residential Water Demand

a)	Average Day domestic demand	using 300L/cap/day (1,456 based on Unit Count & Population)	5.05 L/sec
b)	Peak day demand (Max. Day)	1.5 x daily demand	7.58 L/sec
c)	Peak hour demand	2.5 x daily demand	12.63 L/sec
d)	Fire flow (Fire Resistive)		150 L/sec*

* - Based on typical Residential Demands.

Table 3 – Total Water Demand for the Site

	L/sec	L/min	USGM
Peak Day Demand	7.58	471	120
Fire Flow Demand	150	9,000	2,378
Total Water Demand	157.58	9,471	2,498
Actual Flow at 20 PSI Residual Pressure	160	9,600	2,539

The total water demand, 2501 USGM (fire + peak day), is less than the available flow at 20 psi in the Watson Parkway North watermain (2,539 USGM).

The City of Guelph has provided flows and pressure from a test completed north of the proposed development on Watson Parkway North on September 25 2020. The test was conducted at 78 Starwood Drive near the proposed development.

The Static Pressure from this test was approximately 86 psi with residual pressures at 76 psi and 64 psi.

The available flow at the test hydrant at 20 psi was 2539 USGPM (160 l/sec). For residential developments a fire flow of 150 l/sec is generally adequate for providing fire flow protection. At the time of detailed design the Mechanical and Sprinkler Consultants will confirm required fire flows under the Ontario Building Code.

In order to achieve required fire protection, the proposed development and related buildings should be provided with fire walls and adequate fire separation to meet the available fire flows from Watson Parkway North.

Looping of the water main will be completed to the proposed development to provide adequate flows and redundancy for the larger buildings (if required). This will be detailed during the detailed design and Site Plan Approval stage.

City of Guelph Fire Flow Test shown below. A flow test should be conducted in the Spring in proximity of the proposed development connections to confirm the available flows on Watson Parkway North.

HYDRANT FLOW TEST REPORT

Date: 25-Sep-20 Time: 8pm Operator:

Test Hydrant Information:

Static/Residual Hydrant: 86 Location: 78 Starwood Dr

STATIC PRESSURE: 86 psi
RESIDUAL PRESSURE (1 PORT): 76 psi RESIDUAL PRESSURE (2 PORTS): 64

Flow Hydrants Information: Outlet port type: SMOOTH ROUNDED

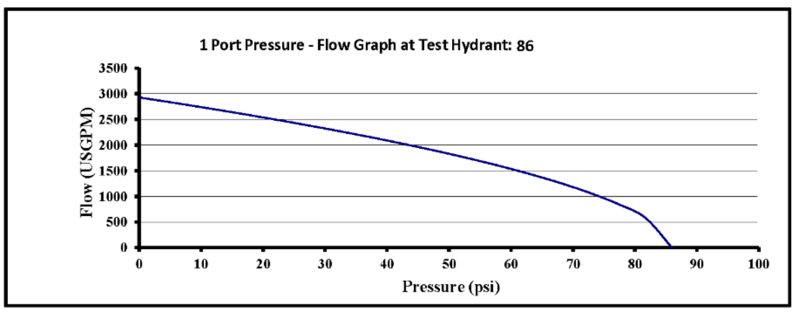
	Hydrant No.	Diffuser Coeff	Outlet Dia. (in.)	Outlet Coefficient	Pitot Gauge Reading (psi)	Total Flow (USGPM)
1 PORT		0.815	2.5	0.9	45	916

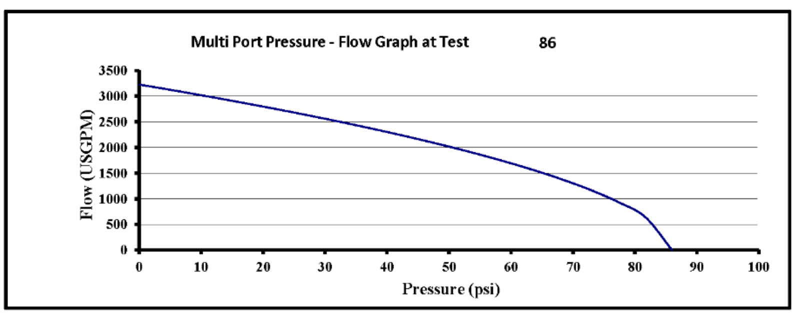
Available Flow At Test Hydrant at 20 psi 2539 USGPM 2114 IGPM
N.F.P.A. Colour Code: BLUE
Minutes of flow: 0.5 1 Port Water Consumption: 1.73 m^3

MULTI PORT Number of ports flowed: 2

Hydrant 1		0.815	2.5	0.9	32	1546
Hydrant 2		Flow calculated from Hydrant 1 row. Pick number of ports above.				

Available Flow At Test Hydrant at 20 psi 2797 USGPM 2329 IGPM
N.F.P.A. Colour Code: BLUE
Minutes of flow: 0.5 Multi Port Water Consumption: 2.93 m^3
Total Water Consumption: 4.66 m^3





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This document can be accessed electronically by searching the title on Guelph's Electronic Document Management System (EDMS)

5.0 STORMWATER MANAGEMENT

5.1 Quantity Control

Existing Condition

On the west side of the subject property within Watson Parkway North there is an existing box culvert located on Watson Parkway North. This box culvert services the subject site as well as the development lands to the north and discharges to the existing downstream stormwater management pond to the south.

This stormwater management pond discharges to Clythe Creek. Outlet location for the proposed development will be to the west of the proposed development to Watson Parkway North. There are multiple outlets provided for the development. An additional outlet will be required to the south end of the site to drain the full development. This secondary outlet will serve to re-charge the existing wetland area and will be detailed at the detailed design stage. The existing outlets provided to the site will be utilized for the development areas fronting Watson Parkway North due to the underground parking structures proposed, if the laterals have adequate capacity, otherwise they will need to be upsized. This will require approval from the City. The existing drainage pattern is from west to east, however, the site tributary has been included as part of the Watson Parkway North storm sewer system and tributary area as part of the Grangehill Estates Phase 3 – Storm Sewer Design Brief prepared by Stantec dated January 1999, File No. 06-8787/20.

Proposed Condition & Design Criteria

Stormwater management for the proposed development will follow the requirements as specified by the City of Guelph. The allowable peak flow for the site will follow the Grangehill Estates Phase 3 – Storm Sewer Design Brief in which the 5 year design storm is to be controlled to the allowable flows. Storms greater than the 5 year design storm are to be directed to a suitable overland flow route. The Minor Storm 5 Year Event is to be conveyed to downstream storm water management pond (SWMP) through the storm sewer system on Watson Parkway North. Major Storm Events are to by bypass SWMP through the flow splitter at the downstream inlet to the SWMP. Low flows are directed to the fore bay and large flows are directed to the quantity storage of the SWMP.

In addition to the above, a water balance will be required for the development in which a portion of storm water will be directed to an infiltration gallery. See section 5.4 of this report for further water balance details. This water will not contribute to the storm water management quantity controls and thus the stormwater quantity controls measures within this section are considered conservative.

Design storm data for the City of Guelph 2 to 100 year storm events are shown below. Visual OTTHYMO will be used to perform the stormwater runoff analysis for the proposed conditions. The 5 year design storm event will be used to determine the total storage required throughout the development during a 5 year design storm. Storms in excess of the 5 year event can be directed to suitable overland flow paths.

Return Period	Equation of Curve
2 Year	$I = \frac{743}{(td+6)^{0.7989}}$
5 year	$I = \frac{1593}{(td+11)^{0.8789}}$
10 Year	$I = \frac{2221}{(td+12)^{0.9080}}$
25 Year	$I = \frac{3158}{(td+15)^{0.9355}}$
50 Year	$I = \frac{3886}{(td+16)^{0.9495}}$
100 Year	$I = \frac{4688}{(td+17)^{0.9624}}$

The initial time of concentration (td) shall be 5 minutes in all cases, except single family residential unit and park areas, where td shall be 10 minutes.

The allowable flow from the development during the City of Guelph 5 year design storm has been established at 0.798 m3/s for a site area of approximately 6.44 ha. This equates to an allowable flow rate of 0.124 m3/s/ha.

The proposed site areas based on developable lands area has been established as approximately 5.51 ha.

Therefore the allowable flow during a 5 Year Design Storm will be limited to the following for flows directed to the storm sewer system via underground sewers.

$$\mathbf{Q_{5allowable} = 5.51\ ha.\ x\ 0.124\ m^3/s/ha. = 0.683\ m^3/s}$$

$$\mathbf{Q_{5allowable} = 683\ l/sec}$$

For the purposes of post-development analysis, the proposed site has been divided into post-development tributary areas as shown in **Appendix D**. See the following **Table 4** for the description and characteristics of the post-development system.

Table 4 – Catchment Descriptions for the Post-Developed Site

Area ID	Description
1	Block 1 – Towers A & B
2	Block 2 – Towers C & D
3	Block 3 – Townhomes, including Roads & Driveways.

In order to control the post-development flows to pre-development levels above ground, underground storage, rooftop and parking lot storage will be required. This will be further detailed as the detailed site plan approval stage. For the purpose of modeling Visual OTTHYMO will be used determine the detention volumes required.

Since the site has three (3) specific development blocks it is proposed to view each block independently to determine storage required. If these blocks were to be severed at any time in the future then they would not impact or alter the design and could function independently.

The following Table provides for the allowable flows from each Block

Table 5 – Allowable Flows based on Blocks

Block	Allowable Flow (l/sec)
1	111
2	106
3	466
Total	683

The following **Table 6** & **Table 7** summarize the parameters used in Visual OTTHYMO to characterize each post-development catchment. Refer also to **Figure 2** in **Appendix D** for post-development tributary areas.

Table 6 – Catchment Descriptions for the Post-Developed Site

Catchment ID	Area (ha)	% impervious	Hydrograph Method	Loss Method for Pervious Areas
Block 1	0.90	95	STANDHYD	Horton's Method
Block 2	0.86	95	STANDHYD	Horton's Method
Block 3	3.75	85	STANDHYD	Horton's Method

Table 7 – Horton's Equation Parameters	
Maximum Initial Infiltration Rate	200 mm/hr
Minimum Initial Infiltration Rate	20 mm/hr
Decay rate of infiltration	2.0 /hr
Accumulated soil moisture at Beginning of Storm	0.0 mm
Pervious Area Depression Storage	5.0 mm

The stage/storage/discharge properties used to model the flow controls for this site are shown in **Appendix D**. This information, along with the surface characteristics of the areas, was used in Visual OTTHYMO to determine the storage requirements and discharge from the orifice-controlled area. Stage/storage/discharge properties were used to model the flow controls for the roof using Visual OTTHYMO. Based on final site plan layout adjustments to the stage/storage/discharge properties and required volumes will be made at the Site Plan approval stage.

Refer to the Visual OTTHYMO detailed output file in **Appendix D** for further details. Please note that in the event that the roof drains become plugged, roof scuppers along the perimeter of the parapet will provide emergency overflow. This relates mainly to Blocks 1 & 2. Block 3 overflow will be directed to the road allowance and directed to suitable overland flow outlet as required.

The following Table 8 summarizes the peak flows from the site.

Table 8 – Summary of Flows from Site (5 Year)	
Block ID	Post-Development Flow (L/s)
Block 1	69
Block 2	68
Block 3	503
Total	**683**

As can be seen above Block 1 & 2 have been over controlled since they will have underground tanks to meet the 5 year design storm. This results in a larger storage tank since the 100 year design storm will need to be considered for storage when connected to the storm sewer within the Right-of-Way. In most cases an adequate overland from underground parking storm water management tanks does not exist and thus the need to over control to a 100 year design storm is required. In addition it will be determined at the detailed design stage during SPA if pumping of this system will be required due to change in grade from Watson Parkway N. to the rear of the Apartment Building sites (Block 1 & 2).

Since Blocks 1 & 2 will be overcontrolled the allowable flow from Block 3 will increase resulting in lower required storage volumes for Block 3 during a 5 year design storm event.

The following **Table 12** summarizes the storage requirements for the 5 year (Block 3) and 5 & 100 year storm events for Blocks 1 & 2 as noted above.

For the purposes of the Zoning Application it has been assumed that Blocks 1 & 2 will have adequate space to provide for underground storage tanks within the underground parking level, this will be reviewed further during the detailed design stage and if required alternate methods to store water may be required for these Blocks. Block 3 will utilized super pipe storage or large culverts and alternate methods to achieve storage as required within the road allowance and on site. For Zoning the site has been modelled using Box Culverts which will be further assessed at the detailed design stage during SPA.

Table 9 – Summary of Volumes Required

Block	**Volume Required (m^3)**		**Volume Provided (m^3)**
	5 Year	**100 Year**	
Block 1	188	408	410
Block 2	184	394	405
Block 3	411	-	432

Based on the above the equivalent tank size for the Apartment Blocks would be as follows:

Block 1 – SWM Tank Vol. = 5.5m x 2.7m (Parking Spot) x 2.5m (assumed height) = $37m^3$
$37m^3$ x 12 Parking Spots = $444m^3$

Block 3 – SWM Tank Vol. = 5.5m x 2.7m (Parking Spot) x 2.5m (assumed height) = $37m^3$
$37m^3$ x 10 Parking Spots = $370m^3$

As demonstrated in above table, there is sufficient storage capacity underground to store the 5 year storm and 100 year storm events on Blocks 1 & 2 and the 5 Year Design Storm on Block 3.

5.2 Emergency Overland Flow Route

As required by the City of Guelph an adequate overland flow path must be provided and major storms are to be routed overland to the City's ROW without exceeding a maximum ponding depth of 0.30m. In order to provide an overland flow route for the proposed development the above criteria will be met via overland paths directed to the City's ROW or directly to Clythe Creek. In addition the proposed development is located adjacent to a Municipal pond facility and Wetland located to the south of the development. These are considered viable overland paths and will be used to provide for and adequate overland flow path.

The overland path will be provided via directing grades to the above noted locations through various grading techniques. These include drainage ditches, swales, overflow structures (such as Ditch Inlets, Culverts, etc.) that are directed to a suitable overland flow outlet.

Where flows cannot be directed to Watson Parkway North via overland flows path or to Watson Road North, they will be directed to the existing Stormwater Management Facility or adjacent wetland and Clythe Creek.

During the 5 Year Minor Storm, event flows from roadways and driveways including a portion of landscape will be directed to the Watson Parkway North storm sewer system. An underground storage system will be provided at the end-of-pipe within the proposed development to control flows to the 5 year design storm where flows exceed the allowable 5 Year design storm as provided by the City of Guelph. At this location, an overflow structure will be provided to direct flow to the stormwater management facility and to Clythe Creek providing for a suitable overland flow route. Portions of the site adjacent to Watson Road and directly abutting Clythe Creek will be directed to these locations due to grading constraints.

For overland flow paths refer to Concept Servicing and Concept Grading Plans.

5.3 Quality Control

Water quality treatment is required for all new development within the City of Guelph. The MOE SWM Planning & Design Manual defines various levels of stormwater quality treatment with the intent of maintaining or enhancing the existing aquatic habitat based on the efficiency of total suspended solids removal. Quality control facilities are required to remove suspended solids (oil and grit) from areas draining driveways and parking lots (i.e. oil/grit separators, catch basins, vegetated strips and combinations of such). In addition there are multiple LIDs and new technologies that can provide additional means to provide quality control. These will all be detailed at the site plan approval stage. The minimum acceptable water quality level for discharge to municipal collection system is 70% for the proposed development as per City of Guelph Stormwater Management Criteria for 115 Watson Parkway North.

Water quality for the site will be accomplished via an Oil/Grit Separator (OGS). The total upstream area contributing flow to the proposed OGS will be considered.

For the proposed development, Oil/Grit Separators will be utilized as the primary means for water quality treatment and will be sized at the detailed design stage.

Low Impact Development Strategies will be implemented at the detailed design stage where achievable and viable, in support of the Oil/Grit Separator as the primary means for water quality treatment.

Maintenance/Clean Out Frequency of Water Quality Manhole Oil/Grit Separators

The OGS will be privately owned and maintained.

The oil/grit separator should be inspected twice a year. The inspection should look for the following:

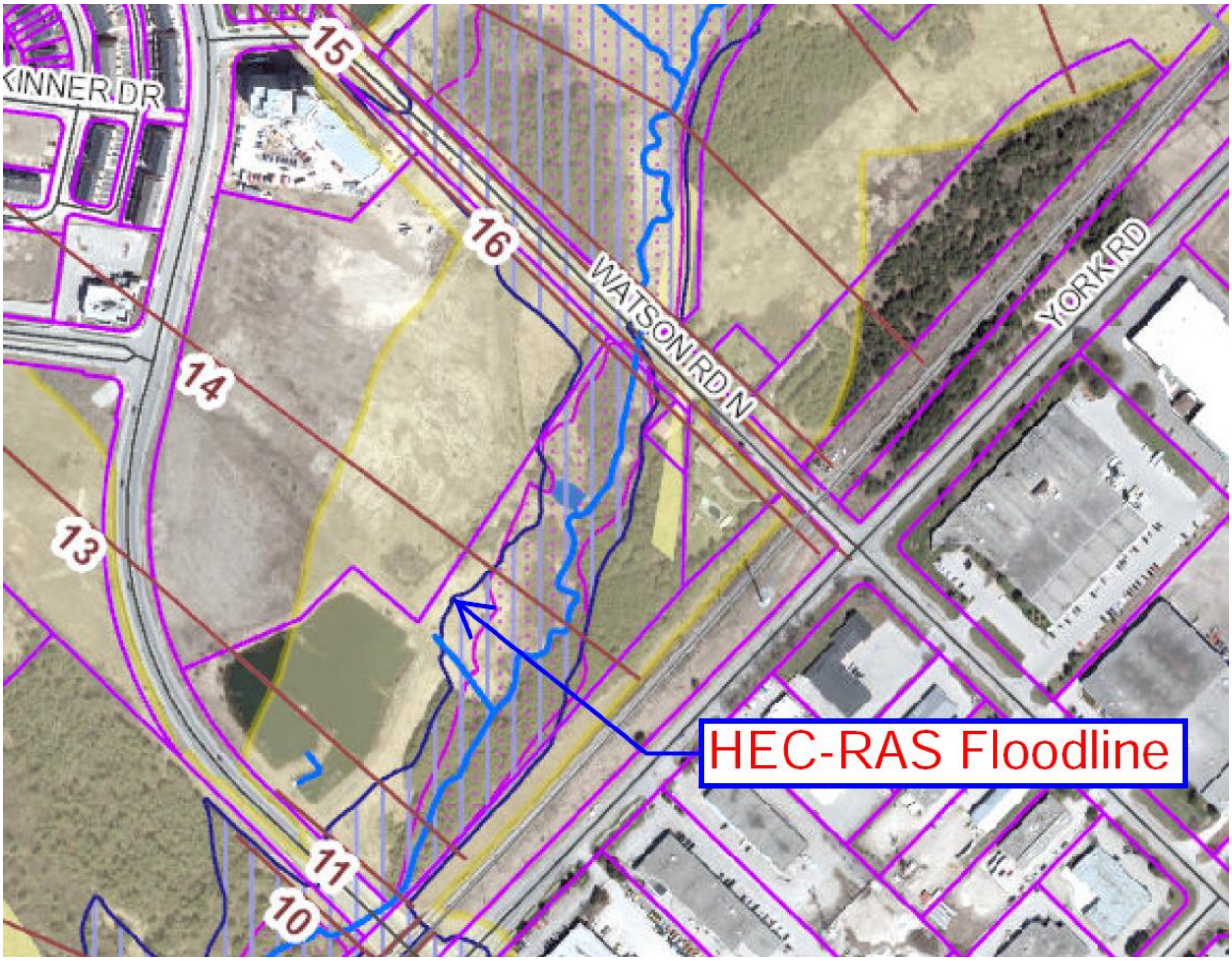
a) The amount of sediment in the bottom (Should be monitored by measurement).

b) Check to see if oil is visible. In addition, check to see if trash is visible. If an oil or industrial spill has occurred, the oil/grit separator should be cleaned immediately.

Sediment should be removed annually, or whenever the accumulation reaches approximately 15% of the operating depth as measured from base to the drain invert. Vacuum trucks are used to remove the sediment and oil from the unit. A licensed waste management firm should remove levels of oil greater than 2.5 cm immediately.

5.4 Flood Plain

The proposed development is located adjacent to lands regulated by the Grand River Conservation Authority. The regulated areas have a floodplain located at the south and east property limits within the development. At this time the floodplain limits are estimated based on HEC-RAS modelling as identified in the figure below.


15
16
14
13
11
10
WATSON RD N
YORK RD
HEC-RAS Floodline

The following excerpt from the GRCA Policies for the Administration of the Development, Interference with Wetlands and Alterations to Shorelines and Watercourses Regulation Ontario Regulation 150/06 Approved October 23rd, 2015 Resolution No. 124-15 Effective October 23rd, 2015, states the following with regards to establishing setbacks within a Riverine Floodplain.

Where the Riverine Flooding Hazard is determined by an engineering study using provincial standards and criteria, a 5 metre (16 foot) allowance is added. Where the Riverine Flooding Hazard is approximated or estimated, a 15 metre (50 foot) allowance is added. In headwater areas, an allowance of 15 metres (50 feet) from the channel bank defines the Regulated Area.

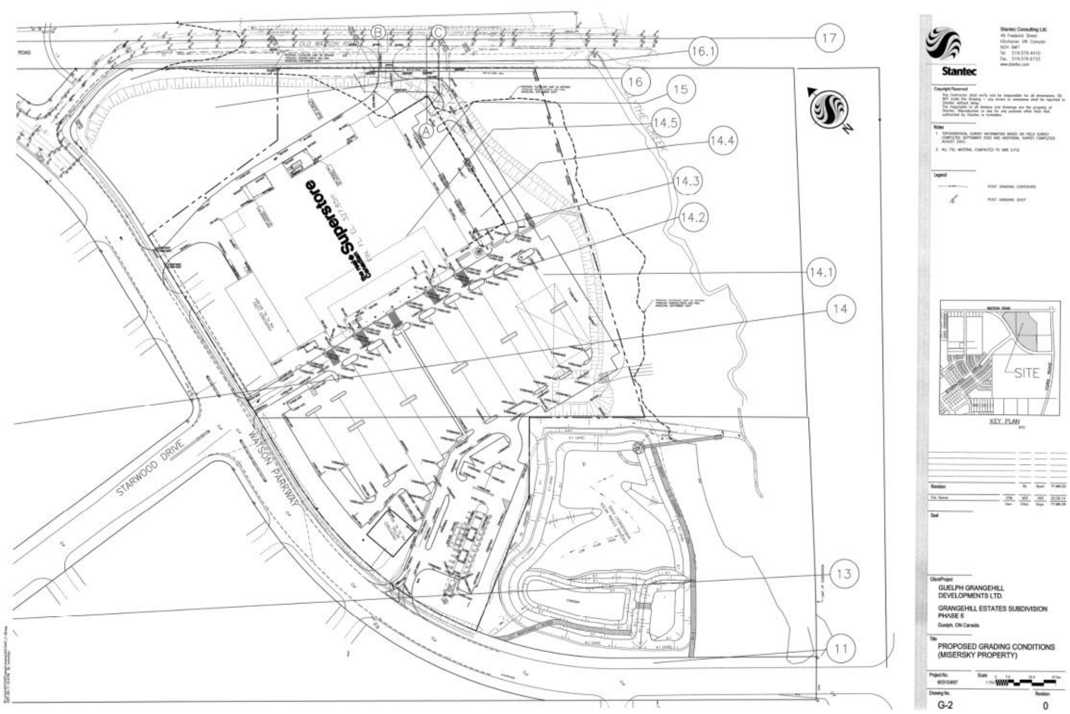
The regulated area within the development has been estimated based on current modelling and therefore a 15m setback is required until an engineering study is completed to allow reduction of the setback to 5m.

In order to determine the actual flood plain and regulated 5m setback the HEC-RAS model will be compared to actual elevations based on the topographic survey completed in 2022 against the HEC-RAS Flood line elevations. The higher of the flood line elevation, Regional vs. 100 Year Storm Event, will be plotted to establish the Flood Line. From this, the 5m setback will be plotted on the drawings to establish the proposed development limits.

In addition to the above there is a Natural Heritage System (NHS) that will need to be considered in addition to any wetland setback. Setbacks related to these features are identified in the EIS prepared by North-South.

The greater of the NHS and Flood Line will establish the development setback and limits.

The following Figure was obtained from the Grand River Conservation Authority (GRCA) and used to establish the flood line based on actual topographic survey information related to the existing development conditions.



Based on the HEC-RAS model the Floodplain Mapping for the location of the stations relative to the subject site are identified below and have been plotted based on the existing topographic survey information obtained within the field.

HEC-RAS Model Flood Elevation (m) -Station with Regional and 100 Year Storm Events-

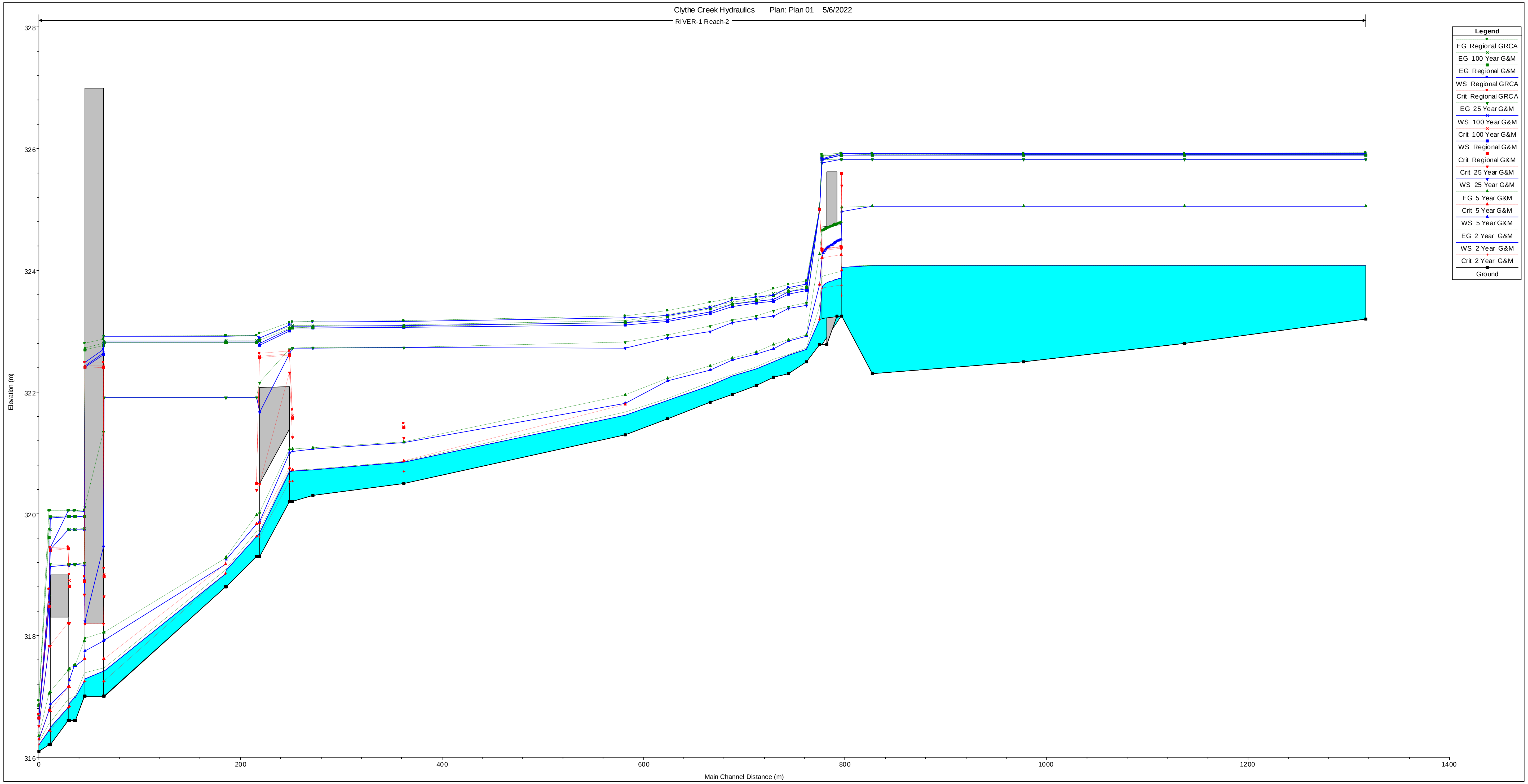
Station	100 Year Storm Event	Regional Storm Event
14	323.14	323.22
14.1	323.19	323.25
14.2	323.32	323.39
14.3	323.44	323.51
14.4	323.49	323.56
14.5	323.52	323.59
15	323.65	323.72
16	323.70	323.78
16.1	325.00	325.00

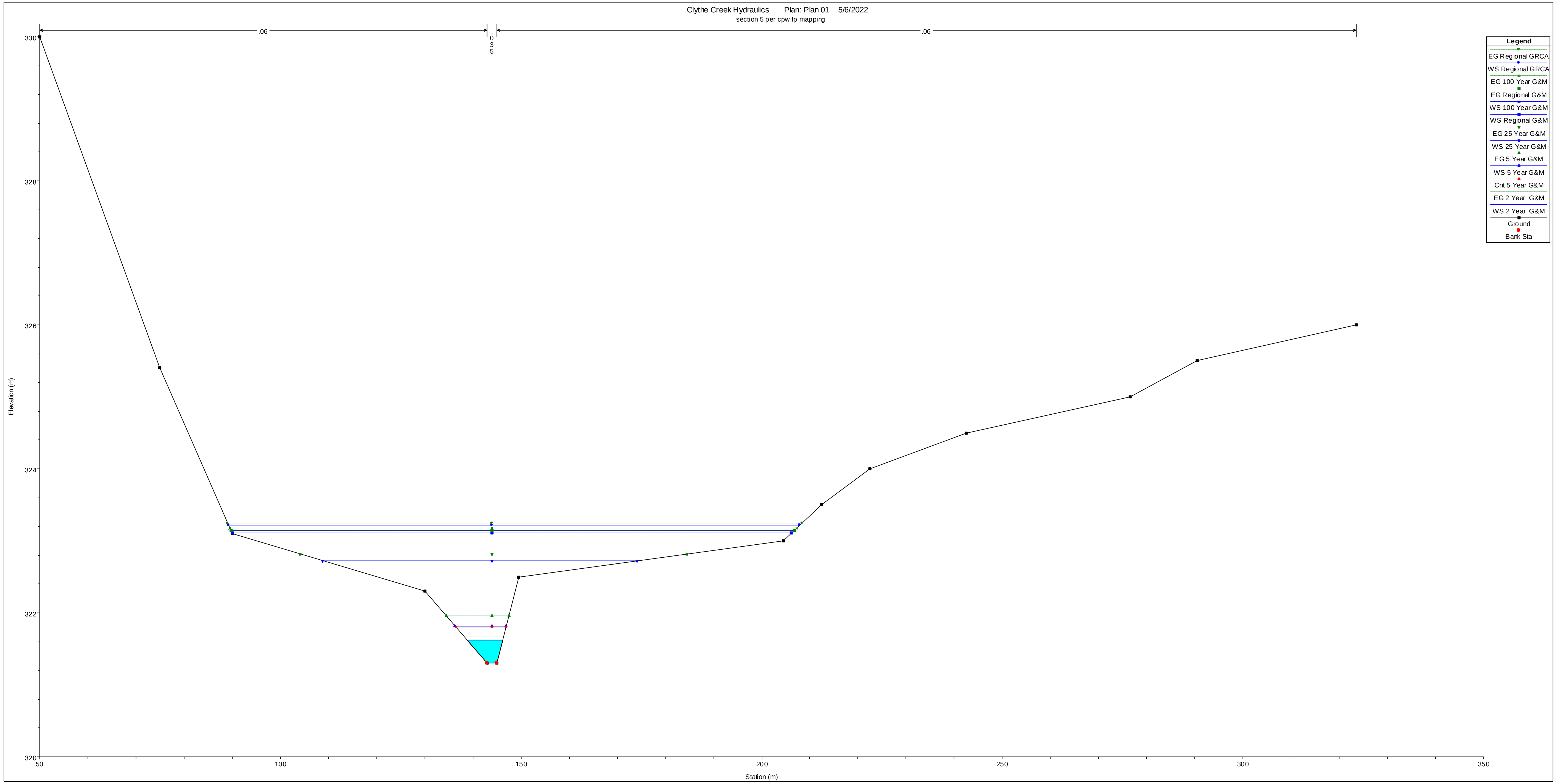
The following Figure shows the plotted Flood Line based on the topographic survey and above elevation

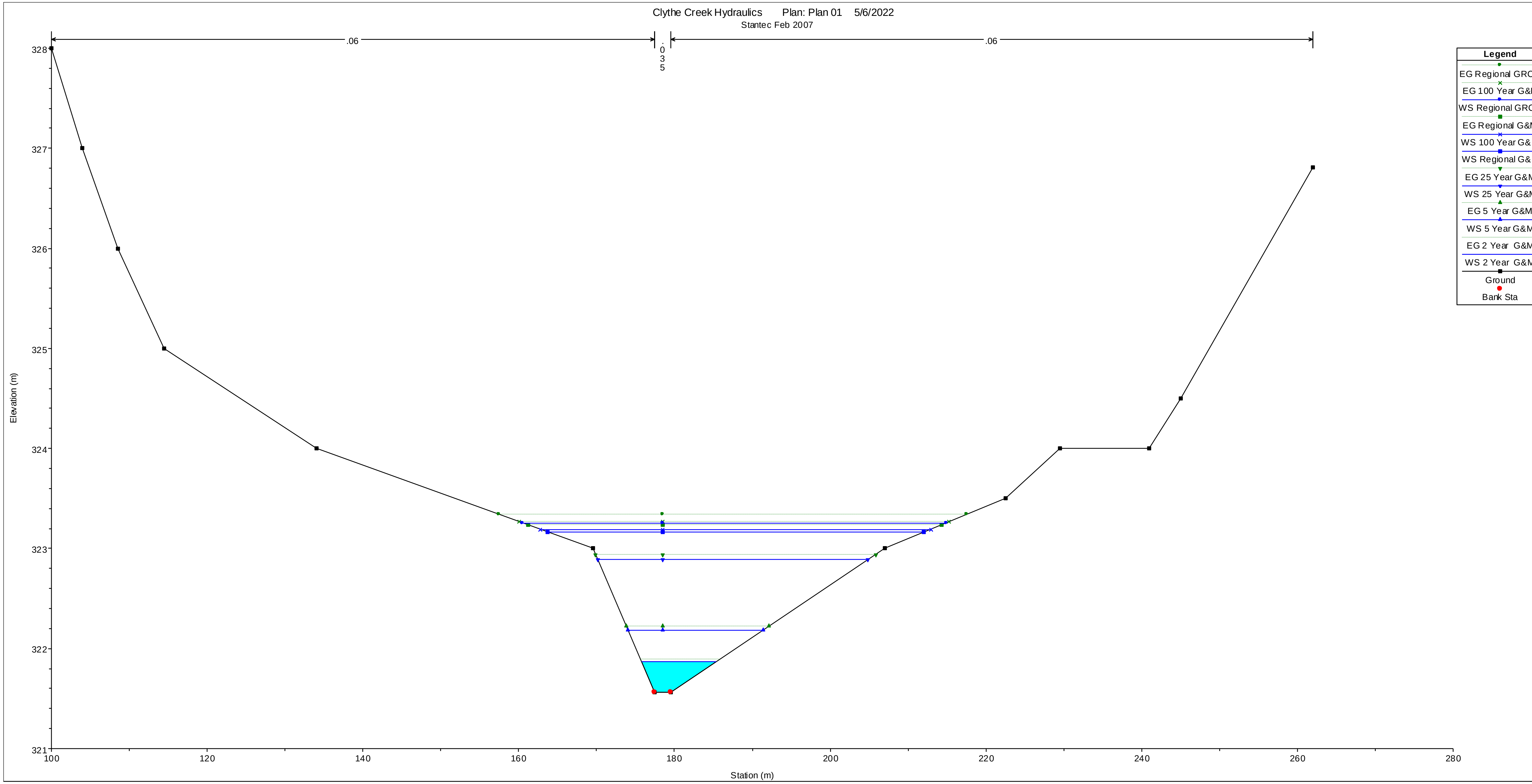

EXISTING STORM WATER MANAGEMENT POND
PARK
PROPERTY LINE
OLD WATSON ROAD
14
14.1
14.2
14.3
14.4
14.5
15
16
16.1
KEY PLAN
SUBJECT LANDS
BENCH MARK:
BEARING NOTE:
METRIC NOTE:
DISTANCES AND ELEVATIONS ON THIS PLAN ARE TYPICALLY SHOWN IN METRES AND CAN BE CONVERTED TO FEET BY DIVIDING BY 0.3048.
2 ISSUED FOR REVIEW & COORDINATION SEPT 13/23 MHH
1 ISSUED FOR REVIEW & COORDINATION JUNE 15/23 MHH
NO. REVISIONS DATE BY
SCALE : 1:500
0 5 10 20 30 40 50
DRAWING :
FIGURE - FP1
FLOODLINE MAPPING
HEC-RAS SECTIONS
CLIENT :
GUELPH WATSON HOLDINGSN INC.
PROJECT :
GUELPH WATSON HOLDINGSN INC.
115 WATSON PARKWAY NORTH
GUELPH, ON
ODAN-DETECH
CONSULTING ENGINEERS
SCALE : 1:500
PROJ. NO.: 22202
DATE STARTED: MAR/23
DESIGN BY: M.H.H.
DRAWN BY: A.M.
CHECKED BY: M.H.H.
APPROVED BY: P.H.
DWG. NO.: FP-1
FILE NUMBER
ENGINEER

The HEC-RAS cross sections are shown below based on the latest model provided by the GRCA for each station identified above.

CLYTHE CREEK PROFILE PLOT – RIVER 1 – REACH 2 -




Clythe Creek Hydraulics Plan: Plan 01 5/6/2022
section 5 per cpw fp mapping
.06
.035
.06
Legend
EG Regional GRCA
WS Regional GRCA
EG 100 Year G&M
EG Regional G&M
WS 100 Year G&M
WS Regional G&M
EG 25 Year G&M
WS 25 Year G&M
EG 5 Year G&M
WS 5 Year G&M
Crit 5 Year G&M
EG 2 Year G&M
WS 2 Year G&M
Ground
Bank Sta
Elevation (m)
330
328
326
324
322
320
50
100
150
200
250
300
350
Station (m)


Clythe Creek Hydraulics Plan: Plan 01 5/6/2022
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EG 100 Year G&M
WS Regional GRCA
EG Regional G&M
WS 100 Year G&M
WS Regional G&M
EG 25 Year G&M
WS 25 Year G&M
EG 5 Year G&M
WS 5 Year G&M
EG 2 Year G&M
WS 2 Year G&M
Ground
Bank Sta
Elevation (m)
Station (m)
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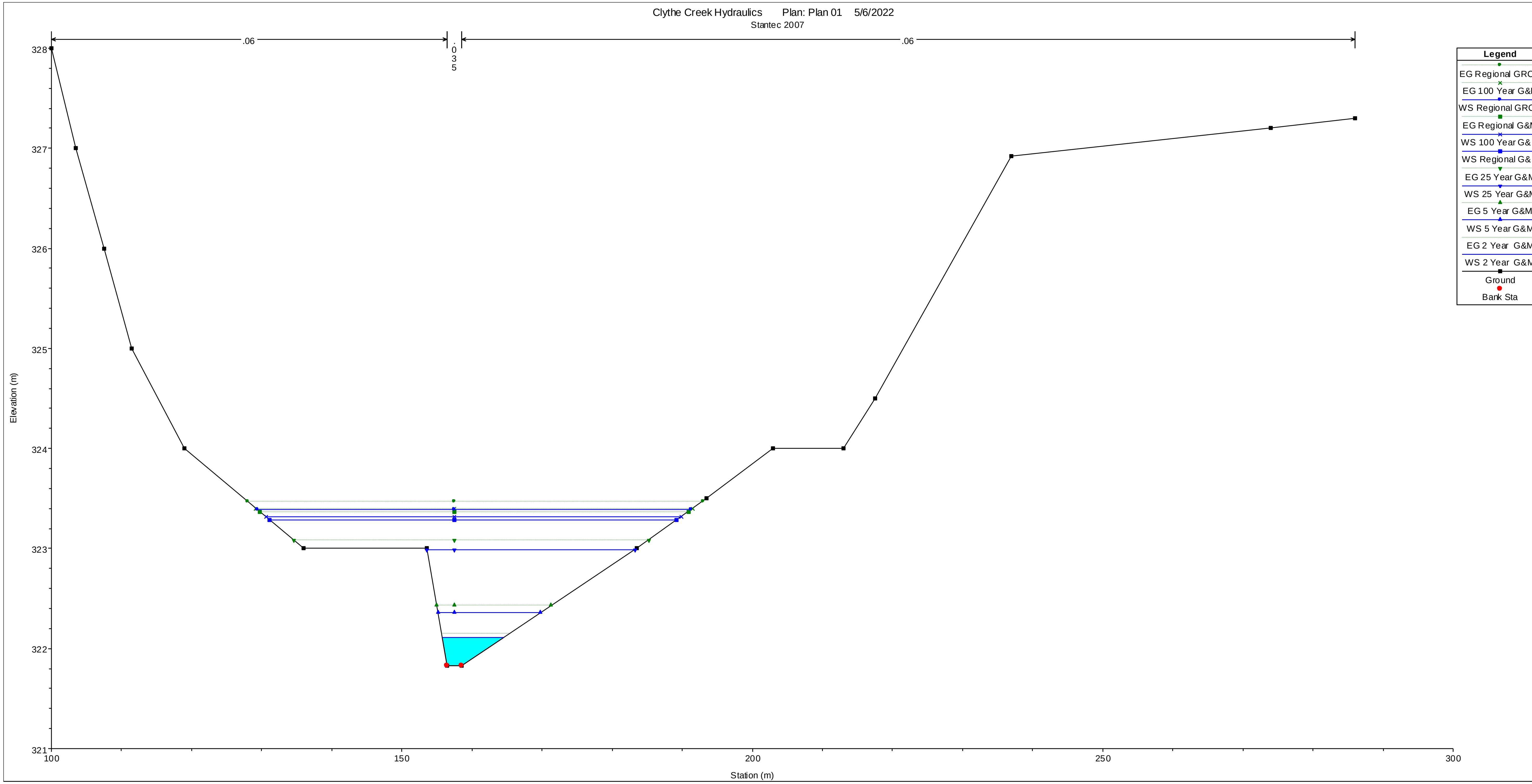
200
220
240
260
280


Clythe Creek Hydraulics Plan: Plan 01 5/6/2022
Stantec 2007
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Legend
EG Regional GRCA
EG 100 Year G&M
WS Regional GRCA
EG Regional G&M
WS 100 Year G&M
WS Regional G&M
EG 25 Year G&M
WS 25 Year G&M
EG 5 Year G&M
WS 5 Year G&M
EG 2 Year G&M
WS 2 Year G&M
Ground
Bank Sta
Elevation (m)
Station (m)
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321
100
150
200
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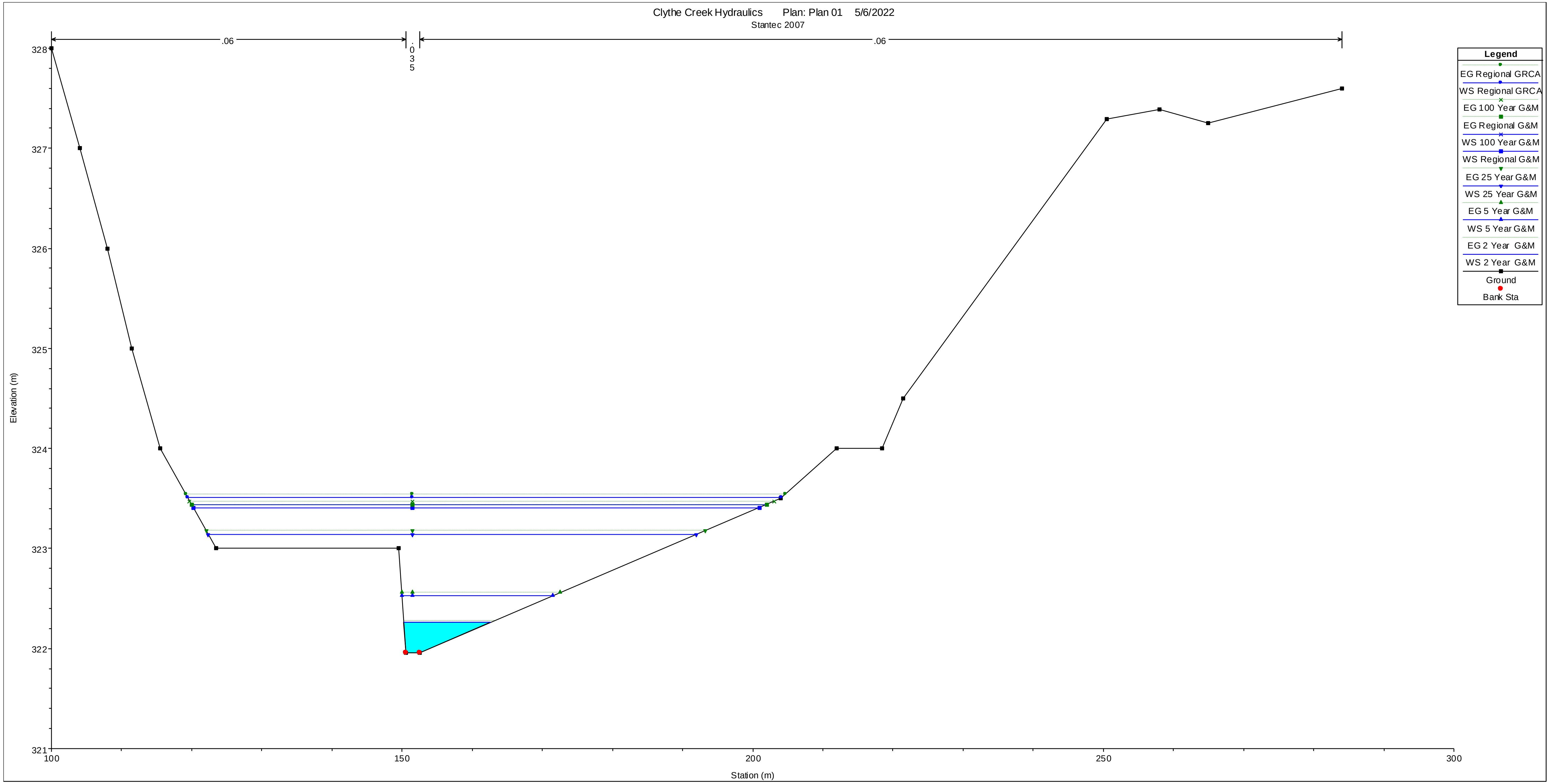
300


Clythe Creek Hydraulics Plan: Plan 01 5/6/2022
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EG Regional GRCA
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EG 25 Year G&M
WS 25 Year G&M
EG 5 Year G&M
WS 5 Year G&M
EG 2 Year G&M
WS 2 Year G&M
Ground
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Elevation (m)
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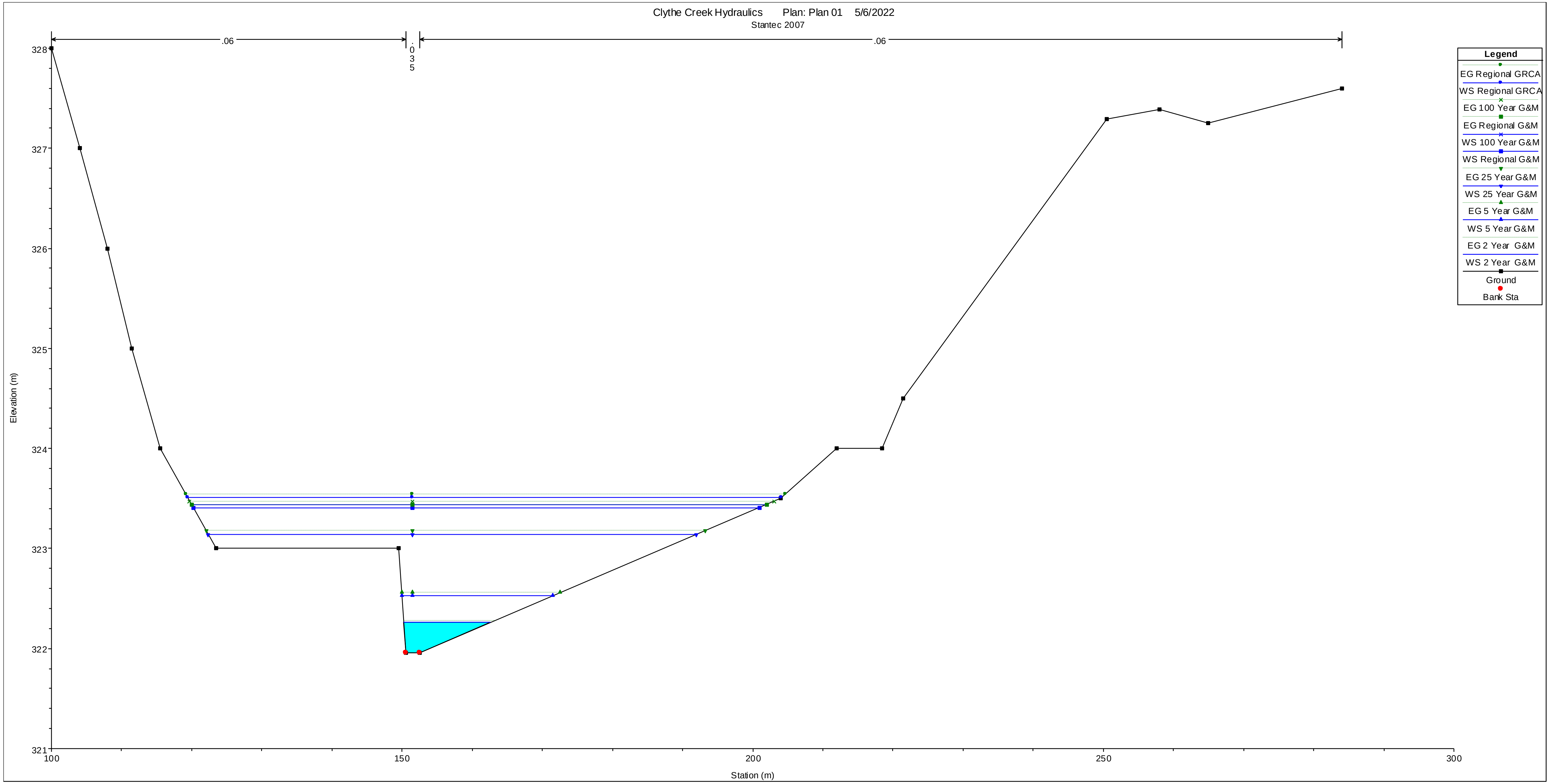
Station (m)


Clythe Creek Hydraulics Plan: Plan 01 5/6/2022
Stantec 2007
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Legend
EG Regional GRCA
WS Regional GRCA
EG 100 Year G&M
EG Regional G&M
WS 100 Year G&M
WS Regional G&M
EG 25 Year G&M
WS 25 Year G&M
EG 5 Year G&M
WS 5 Year G&M
EG 2 Year G&M
WS 2 Year G&M
Ground
Bank Sta
Elevation (m)
328
327
326
325
324
323
322
321
Station (m)
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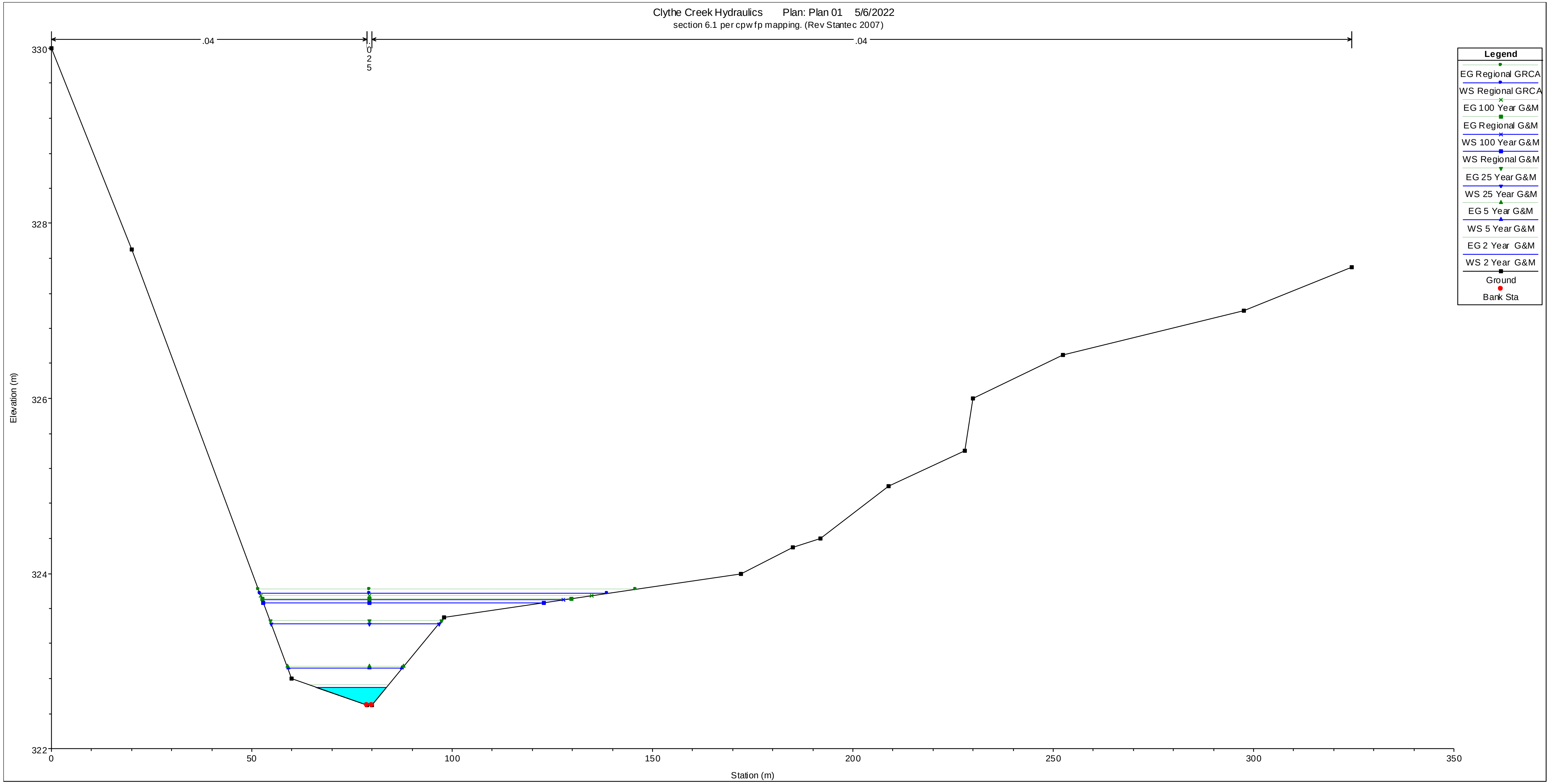
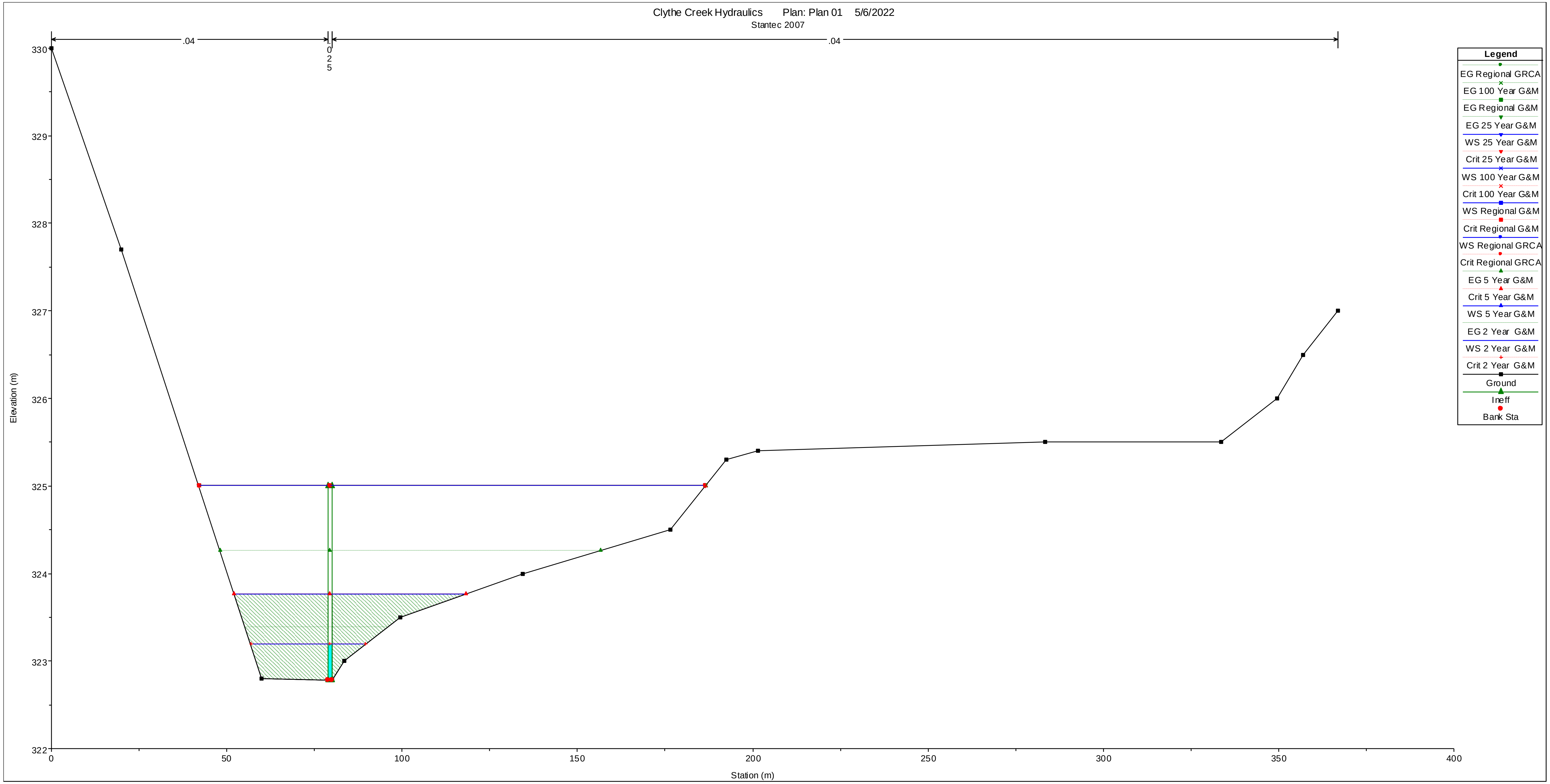
150
200
250
300


Clythe Creek Hydraulics Plan: Plan 01 5/6/2022
section 6.1 per cpw fp mapping. (Rev Stantec 2007)
.04
0
2
5
.04
Legend
EG Regional GRCA
WS Regional GRCA
EG 100 Year G&M
EG Regional G&M
WS 100 Year G&M
WS Regional G&M
EG 25 Year G&M
WS 25 Year G&M
EG 5 Year G&M
WS 5 Year G&M
EG 2 Year G&M
WS 2 Year G&M
Ground
Bank Sta
Elevation (m)
330
328
326
324
322
0
50
100
150
200
250
300
350
Station (m)


Clythe Creek Hydraulics Plan: Plan 01 5/6/2022
Stantec 2007
.04
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2
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.04
Legend
EG Regional GRCA
EG 100 Year G&M
EG Regional G&M
EG 25 Year G&M
WS 25 Year G&M
Crit 25 Year G&M
WS 100 Year G&M
Crit 100 Year G&M
WS Regional G&M
Crit Regional G&M
WS Regional GRCA
Crit Regional GRCA
EG 5 Year G&M
Crit 5 Year G&M
WS 5 Year G&M
EG 2 Year G&M
WS 2 Year G&M
Crit 2 Year G&M
Ground
Ineff
Bank Sta
Elevation (m)
330
329
328
327
326
325
324
323
322
0
50
100
150
200
250
300
350
400
Station (m)

The established elevations above have confirmed the Floodplain elevations and based on the existing topographic information obtained the revised floodline elevation has been plotted as per Figure FP-1 on page 19.

Based on the information obtained from recent and previous topographic surveys for the development, Figure FP-1 also shows the established 5m setback for the proposed development based the HEC-RAS Model elevations.

5.5 Water Balance

Pre-Development Water Balance

The site water balance requirements have been outlined in the Hydrogeology Investigation Report and Water Balance Assessment by Palmer Environmental Consulting Group. The pre-development water budget was calculated for the site area using the Thornwaite and Mather (1957).

A feature based water balance was also studied at the east and south sides of the site. The feature based water balance used the same methodology as the site water balance study. It was assumed in the calculation that 40% of the site surface runoff contributes to the wetland complex. A summary of both the site and feature based water balance is provided in the below table.

The pre-development water balance

Table 10 - Pre-Development Water Balance

Area I.D.	Area (ha)	Runoff Volume (m^3/yr)	Infiltration Volume (m^3/yr)
Total Site	6.45	7,730	18,036
Wetland	2.87	3,439	8,025

Post-Development Water Balance

To implement low impact development measures and meet the water balance criteria for the proposed development, a soakaway pit is proposed. The soakaway pit is located on the northeast side of the site in the park lands. Runoff from the northeast portion of the site is to be discharged into soakaway pit. Due to the large water balance deficit, it is not possible to achieve the water balance for the site solely through the infiltration of clean roof runoff. Therefore, flows for water balance are captured from a combination of roof runoff and hardscape area in the northeast portion of the site. Oil grit separators are proposed upstream of the soak away pit to provide water quality control.

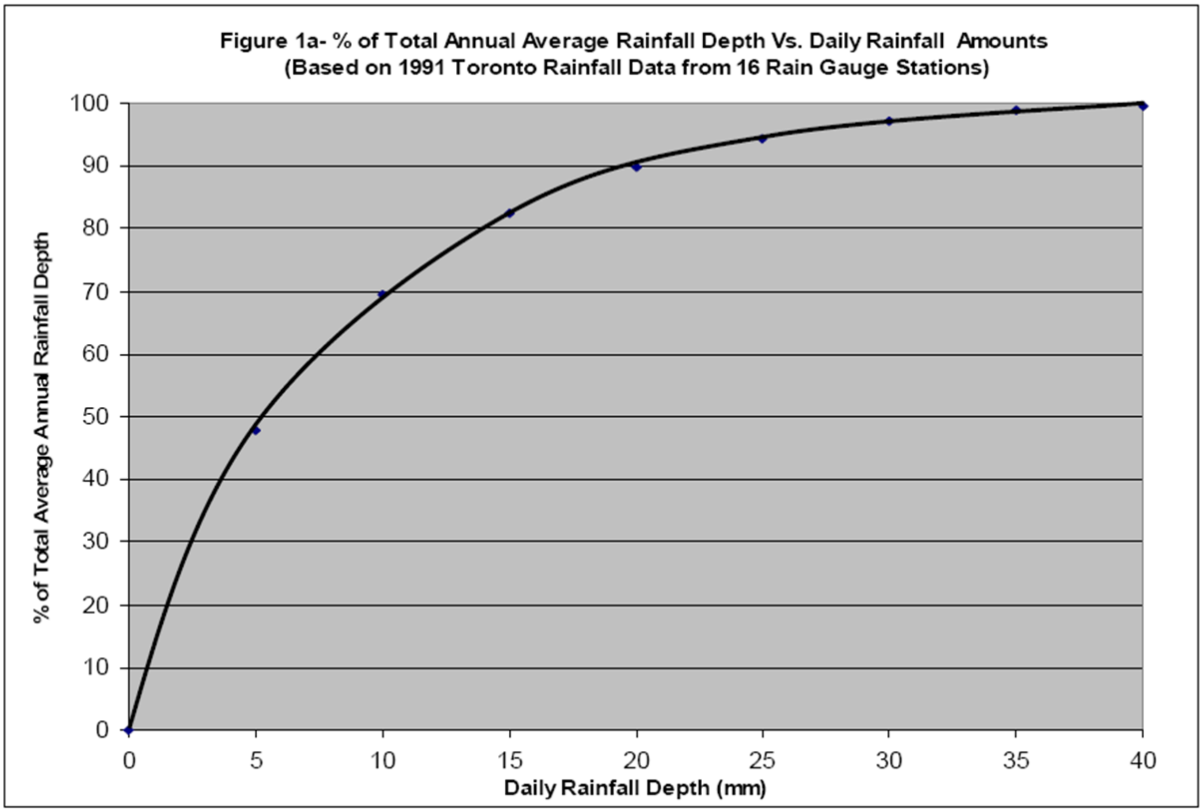
The soakaway pit is located in close proximity to groundwater monitoring well BH22-12s. The high groundwater level at this location is 323.07m, as provided in the Hydrogeology Report by Palmer. The soakaway pit has been provided with a bottom elevation of 324.51m. Therefore the required 1m separation from the soakaway bottom to the seasonally high groundwater level has been provided. The soakaway is designed to convey, and attenuate/infiltrate storm water runoff, ultimately reducing the peak flow and improving the quality of the runoff entering the receiving watercourses downstream. The runoff passes through to a layer of clear stone wrapped in geotextile for attenuation and infiltration into the underlying soils.

A summary of the site's post development water balance is provided in Table 11 below.

Table 11 - Water Balance for Post-Developed Site

Area I.D.	Area (ha)	Runoff Volume (m^3/yr)	Infiltration Volume (m^3/yr)	Runoff Volume Deficit (m^3/yr)	Infiltration Volume Deficit (m^3/yr)
Total Site	6.45	30,580	8,971	N/A	9,065
Wetland	0.48	551	1,286	2,888	6,739

Using *Figure 1a - % of Total Annual Average Rainfall Depth Vs. Daily Rainfall Amounts* provided below, a relationship is provided between these two variables which is used to determine the surface capture depth required to meet the infiltration deficit for the site. Note that although this relationship was developed by the City of Toronto, this relationship would remain very close for the hydrology in the City of Guelph. A relationship between these variables has not been developed by the City of Guelph.



The capture depth is used to calculate the volume of infiltration swale. It was determined that due to the lack of rooftop area on this site, it is not possible to achieve a water balance infiltrating clean roof runoff only.

Therefore, a portion of the site's storm system has been directed to the soakaway pit. Providing the infiltration swale volume to detain 7.5mm depth of rooftop capture will meet the annual recharge deficit for the site. Sample calculations are provided below.

$$1)\ Site\ Defecit = 9{,}062\ m^3\ (provided\ by\ Hydrogeology\ Report)$$

$$2)\ Soakaway\ Pit\ Impervious\ Area = Soakaway\ Pit\ Trib\ Area\ X\ Runoff\ Coefficient$$

$$11{,}325\ m^2 = 15{,}100\ m^2\ X\ 0.75$$

$$3)\ Annual\ Capture\ Required = \frac{Annual\ Recharge\ Volume}{Soakaway\ Impervious\ Area}$$

$$0.800m = \frac{9065\ m3}{11{,}325\ m2}$$

$$3)\ \%\ Total\ Average\ Annual\ Rainfall = \frac{Average\ Annual\ Capture}{Total\ Annual\ Precipitation}$$

$$87\% = \frac{800mm}{916mm}$$

4) Using Figure 1a, 90% of total annual rainfall depth occurs from daily rainfall depth of 18mm or less. Therefore, the total volume to be infiltrated.

$$Total\ Infiltration\ Volume = Trib\ Area\ X\ Depth\ of\ Rainfall$$

$$204\ m^3 = 11{,}325\ m^2\ X\ 0.018m$$

From the calculation above, 87% of the annual rainfall is to be captured and infiltrated. Using Figure 1a, a rainfall depth of 18mm over the soakaway pit tributary area must be infiltrated through the soakaway pit to achieve the site water balance.

An infiltration rate was provided in the Hydrogeology Report by Palmer. Multiple in-situ tests were conducted which resulted in an average design infiltration rate of 28.3mm/hr which includes a factor of safety of 2.5. The test completed in the closest proximity to the soakaway pit resulted in an infiltration rate of 51mm/hr. Therefore a design infiltration rate of 20.6mm/hr was used for the soak away pit calculation using a safety factor of 2.5. The in-situ infiltration rate must be verified at time of construction. The soakaway calculation is provided on the following page.

SOAK-AWAY PIT CALCULATION TEMPLATE

PROJECT: Tercot Communities

PROJECT No. : 22202

Location of soak-away pit: See Servicing Plan

							DESCRIPTION
						UNIT	A = 1,000V / (PnΔt)
	d = PΔt/1000		P=	20.6	mm/hr		Where:
			Δt =	36	hr		
d =	0.74	m					A = Filter bed surface area (m2)
			n=	0.40	-		V = Water volume (m3)
							Δt = time to drain (hr)
							n = void space ratio for aggregate used
V=	204	m3	Atrib	11325	m2		(note: void space ratio of 0.4 to be used)
			runoff	18	mm		
							d = PΔt/1000
							Where:
							d = maximum soak-away depth (m)
A =	1,000V / (PnΔt)						P= infiltration rate for native soils (mm/hr)
							Δt = time to drain (hr)
Af=	687	m2					
Af provided =	690	m2					Vpit req'd = V/n
							Vpit provided = L x W x d
Vpit req'd =	510	m3					
							L = length of pit (m)
Vpit provided =	**510.6**	m3	Af =	690	m2		W = width of pit (m)
			d =	0.74	m		d = depth of pit (m)

The post development site is able to meet the City criteria for post to pre-development water balance through a combination of infiltration to the site's landscape area as well as the implementation of a soakaway pit. A summary of the post to pre-water balance is provided in Table 12.

Table 12 - Summary of Water Balance

Scenario	Area (ha)	Annual Recharge Target Volume (m^3)
Pre-Development (**Deficit**)	6.45	9065
Post-Development	6.45	9065

As shown above, the total site infiltration water balance deficit has been met, including the wetland. A feature-based water balance will be provided at the detailed design stage which shows that the run-off deficit for the wetland will also be met. Based on an annual run-off deficit for the wetland of 2,888 m^3 and an annual precipitation of 916mm, an impervious roof area of 3152 m2 will be directed to the wetland to recharge the runoff deficit.

6.0 SOIL CONDITIONS

A geotechnical investigation was completed by Toronto Inspection Limited and Reports were provided entitled “Report on Preliminary Geotechnical Investigation, 115 Watson Pkwy North, Guelph Ontario (March, 2022)”, and “Addendum To Geotechnical Investigation Report (June, 2022)”.

Soil Type — Based on the Borehole information in the Geotechnical Report, the site consists of a layer of fill material, overlying native deposits of sand, silty sand and gravel, and sandy silt till.

Top Soil — There is minimal topsoil on site. The surface layer should be scrapped and cleaned of any deleterious material.

Fill — The site was uplifted/graded in the past will the placement of fill material.
Fill appears to have some compaction based on standard penetration tests.
Fill is approximately 1.2m in depth.

7.0 GROUND WATER

Depth is expected to be between 5-7m from ground level based on information available from adjacent developments.

8.0 EROSION CONTROL

Erosion and sediment controls for the site will be implemented according to The Ministry of Natural Resources Guidelines on Erosion and Sediment Control for Urban Construction Sites. A detailed erosion control plan will be provided at the detailed design stage.

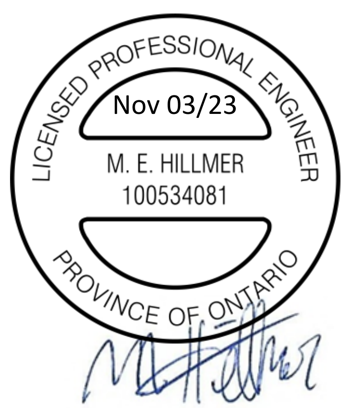
9.0 CONCLUSIONS

From our investigation, the site is serviceable utilizing existing sanitary, storm and watermain infrastructure adjacent to the site. The post development 2, 5, 10, 25, 50 & 100-year storm design have been maintained at the allocated flow rate for the site.

The following **Table 13** summarizes the components of the proposed development.

Table 13 – Summary Information	
Total Sanitary Flow (L/sec)	22.39
Total Water Demand : (L/sec)	2,498
Allowable release rate from site (L/sec) (100 year storm)	683
Actual release rate from site (L/sec) (100 year storm)	683
Total Storm Water Storage Required (m3)	783
Total Storm Water Storage Provided (m3)	1,247
Water Quality	Oil/Grit Separator or Alt.

Respectfully Submitted;
The Odan/Detech Group Inc.



Paul Hecomovic, P.Eng.

Mitchell Hillmer, P.Eng.

APPENDIX A

AERIAL PHOTO AND SURVEY OF EXISTING SITE

TOPOGRAPHIC SURVEY (REDUCED)

A.1 Aerial Photo of Existing Site

Appendix A – Figure 1: Aerial Photo of Existing Site is an excerpt from as shown on Google Maps as of January 2023 with the approximate property line shown (**red** line). For detailed information regarding the existing property line and topography site conditions, refer to the latest survey provided within, see also **Appendix A – Figure 2**.

A.2 Topography of Existing Site

Appendix A – Figure 2: Topography of Existing Site is topography from prepared by Schaeffer Dzaldov Bennett Ltd.

APPENDIX B

SANITARY FLOW CALCULATIONS

Mark Harris - Odan Detech Group

From:	Mary Angelo <Mary.Angelo@guelph.ca>
Sent:	Wednesday, July 06, 2022 4:27 PM
To:	mark@odandetech.com; drago@odandetech.com
Cc:	Michelle Thalen
Subject:	RE: Master Servicing - Sanitary Allocation - 115 Watson Parkway -
Attachments:	RE: Starwood and Watson Site - near SWM Pond - Sanitary Tributary Plans - (217 KB)

Hi Mark and Drago,

The model analysis with the 21.72L/s shows that the existing system has capacity today to manage this flow. Please note that the sewer on York is very full so we would not anticipate it being able to accept any more than the 21.72L/s. Also remember that we are not committing this capacity to you today; other developments could be approved before you that use up the capacity.

...Mary

Mary Angelo, P.Eng. (she/her) Manager, Development and Environmental Engineering,
Engineering and Transportation Services,
City of Guelph,
519-822-1260, ext. 2287,
mary.angelo@guelph.ca

APPENDIX C

FLOW TEST (if available)

HYDRANT FLOW TEST REPORT

Date:	25-Sep-20	Time:	8pm	Operator:	

Test Hydrant Information:

Static/Residual Hydrant	86	Location:	78 Starwood Dr

STATIC PRESSURE: 86 psi
RESIDUAL PRESSURE (1 PORT): 76 psi RESIDUAL PRESSURE (2 PORTS): 64

Flow Hydrants Information: Outlet port type: SMOOTH ROUNDED

	Hydrant No.	Diffuser Coeff	Outlet Dia. (in.)	Outlet Coefficient	Pitot Gauge Reading (psi)	Total Flow (USGPM)
1 PORT		0.815	2.5	0.9	45	916

Available Flow At Test Hydrant at 20 psi 2539 USGPM 2114 IGPM
N.F.P.A. Colour Code: BLUE

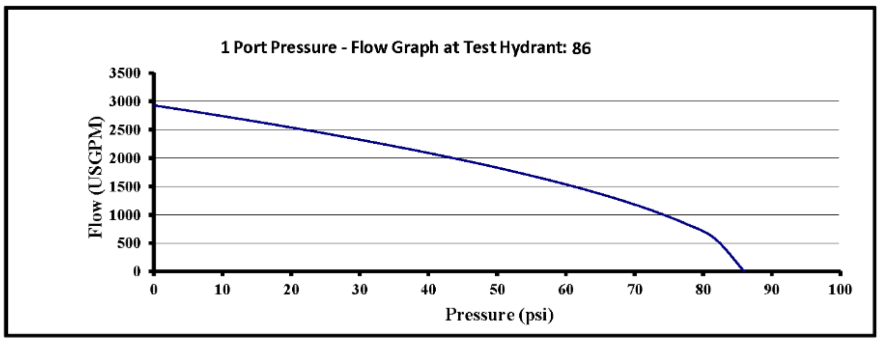
Minutes of flow: 0.5 1 Port Water Consumption: 1.73 m^3

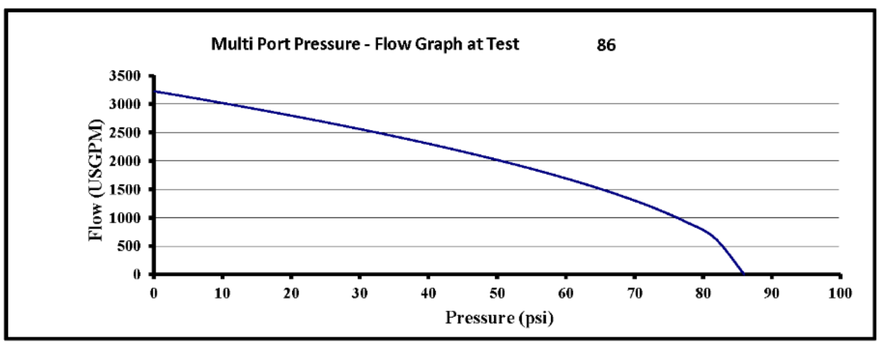
MULTI PORT	Number of ports flowed:	2				
Hydrant 1		0.815	2.5	0.9	32	1546
Hydrant 2		Flow calculated from Hydrant 1 row. Pick number of ports above.				

Available Flow At Test Hydrant at 20 psi 2797 USGPM 2329 IGPM
N.F.P.A. Colour Code: BLUE

Minutes of flow: 0.5 Multi Port Water Consumption: 2.93 m^3

Total Water Consumption: 4.66 m^3





Printed documents are uncontrolled Revision date: February 27, 2019
This document can be accessed electronically by searching the title on Guelph's Electronic Document Management System (EDMS)

APPENDIX D

PREDEVELOPMENT OTTHYMO PLAN (Not Applicable)

POST-DEVELOPMENT STORM DRAINAGE OTTHYMO PLAN

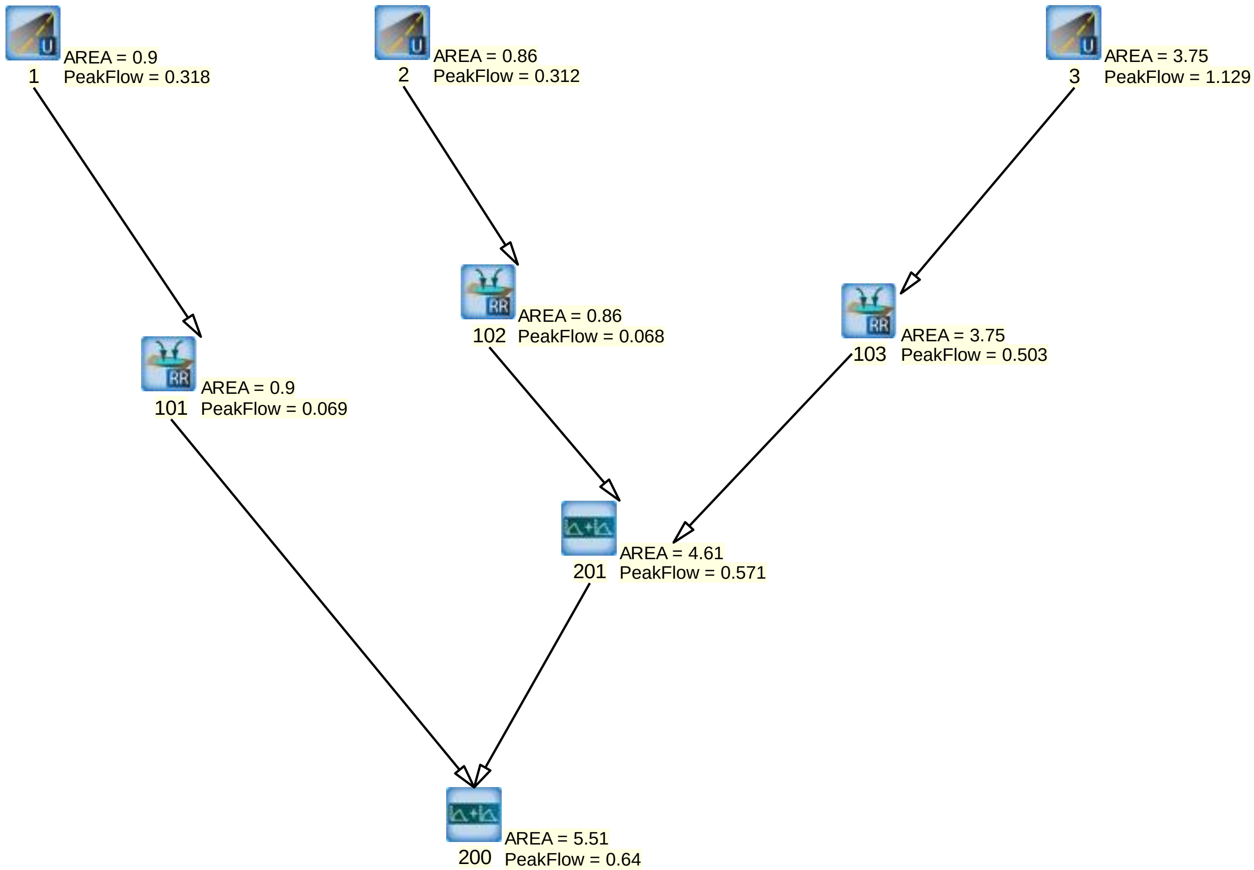
VISUAL OTTHYMO MODEL INPUT/OUTPUT

RAINFALL INTENSITY CHART

STAGE/STORAGE/DISCHARGE CALCULATION SHEETS

HEC-RAS DATA

STORM SEWER DESIGN SHEETS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PeakFlow = 0.068
102
AREA = 3.75
PeakFlow = 0.503
103
AREA = 0.9
PeakFlow = 0.069
101
AREA = 4.61
PeakFlow = 0.571
201
AREA = 5.51
PeakFlow = 0.64
200

==

```
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       V    V   I    SS     U   U   A A   L
        V  V    I     SS    U   U  AAAAA  L
        V  V    I      SS   U   U  A   A  L
         VV     I    SSSSS  UUUUU  A   A  LLLLL

        OOO   TTTTT  TTTTT  H   H  Y   Y  M   M   OOO
       O   O    T      T    H   H   Y Y   MM MM  O   O
       O   O    T      T    H   H    Y    M   M  O   O
        OOO     T      T    H   H    Y    M   M   OOO
```

Developed and Distributed by Clarifica Inc.


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                   *****  D E T A I L E D   O U T P U T *****


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  Output  filename: P:\2022\22202\Design and Reports\Computer Analysis\TERCOT Watson Parkway\Post 5 Year.out
  Summary filename: P:\2022\22202\Design and Reports\Computer Analysis\TERCOT Watson Parkway\Post 5 Year.sum


DATE: 3/8/2023                           TIME: 3:13:07 PM

USER:



COMMENTS: ____________________________________________________________


-------------------------------------------------------------------------------------------------------------
  ****************************
  ** SIMULATION NUMBER:   1 **
  ****************************

--------------------
| CHICAGO STORM    |    IDF curve parameters: A=1593.000
| Ptotal= 47.26 mm |                          B=  11.000
--------------------                          C=    .879
                        used in:   INTENSITY =  A / (t + B)^C

                        Duration of storm  =  3.00 hrs
                        Storm time step    =  5.00 min
                        Time to peak ratio =   .33

                 TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                  hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
                  .08    3.03 |   .83   24.60 |  1.58   11.05 |  2.33    4.23
                  .17    3.38 |   .92   57.39 |  1.67    9.46 |  2.42    3.95
                  .25    3.82 |  1.00  139.29 |  1.75    8.24 |  2.50    3.70
                  .33    4.39 |  1.08   72.74 |  1.83    7.29 |  2.58    3.48
                  .42    5.14 |  1.17   42.12 |  1.92    6.52 |  2.67    3.29
                  .50    6.20 |  1.25   28.38 |  2.00    5.89 |  2.75    3.11
                  .58    7.75 |  1.33   20.88 |  2.08    5.37 |  2.83    2.96
                  .67   10.24 |  1.42   16.28 |  2.17    4.93 |  2.92    2.82
                  .75   14.73 |  1.50   13.22 |  2.25    4.55 |  3.00    2.69


--------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (0001) |   Area    (ha)=    .90
|ID= 1 DT= 5.0 min |   Total Imp(%)=  90.00   Dir. Conn.(%)=  90.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .81          .09
     Dep. Storage     (mm)=       1.00         1.50
     Average Slope     (%)=       1.00         2.00
```

```
     Length            (m)=      77.50        40.00
     Mannings n           =       .013         .250

     Max.Eff.Inten.(mm/hr)=     139.29        76.21
                over (min)        5.00         5.00
     Storage Coeff.  (min)=       1.92 (ii)    4.49 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .31          .23
                                                            *TOTALS*
     PEAK FLOW       (cms)=        .30          .02           .318 (iii)
     TIME TO PEAK    (hrs)=       1.00         1.00           1.00
     RUNOFF VOLUME    (mm)=      46.26        15.02          43.13
     TOTAL RAINFALL   (mm)=      47.26        47.26          47.26
     RUNOFF COEFFICIENT   =        .98          .32            .91

***** WARNING: STORAGE COEFF. IS SMALLER THAN TIME STEP!

       (i) HORTONS EQUATION SELECTED FOR PERVIOUS LOSSES:
            Fo   (mm/hr)= 50.00          K  (1/hr)=  2.00
            Fc   (mm/hr)=  7.50     Cum.Inf.   (mm)=   .00
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

-------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (0002) |   Area    (ha)=    .86
|ID= 1 DT= 5.0 min |   Total Imp(%)=  95.00   Dir. Conn.(%)=  95.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .82          .04
     Dep. Storage     (mm)=       1.00         1.50
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      75.70        40.00
     Mannings n           =       .013         .250

     Max.Eff.Inten.(mm/hr)=     139.29        76.21
                over (min)        5.00         5.00
     Storage Coeff.  (min)=       1.89 (ii)    3.80 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .32          .25
                                                            *TOTALS*
     PEAK FLOW       (cms)=        .30          .01           .312 (iii)
     TIME TO PEAK    (hrs)=       1.00         1.00           1.00
     RUNOFF VOLUME    (mm)=      46.26        15.02          44.69
     TOTAL RAINFALL   (mm)=      47.26        47.26          47.26
     RUNOFF COEFFICIENT   =        .98          .32            .95

***** WARNING: STORAGE COEFF. IS SMALLER THAN TIME STEP!

       (i) HORTONS EQUATION SELECTED FOR PERVIOUS LOSSES:
            Fo   (mm/hr)= 50.00          K  (1/hr)=  2.00
            Fc   (mm/hr)=  7.50     Cum.Inf.   (mm)=   .00
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

-------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (0003) |   Area    (ha)=   3.75
|ID= 1 DT= 5.0 min |   Total Imp(%)=  85.00   Dir. Conn.(%)=  85.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=       3.19          .56
     Dep. Storage     (mm)=       1.00         1.50
     Average Slope     (%)=       1.00         2.00
     Length            (m)=     158.10        40.00
     Mannings n           =       .013         .250

     Max.Eff.Inten.(mm/hr)=     139.29        76.21
```

```
              over (min)        5.00        10.00
     Storage Coeff.  (min)=       2.95 (ii)    6.03 (ii)
     Unit Hyd. Tpeak (min)=       5.00        10.00
     Unit Hyd. peak  (cms)=        .28          .15
                                                      *TOTALS*
     PEAK FLOW       (cms)=       1.09          .09         1.129 (iii)
     TIME TO PEAK    (hrs)=       1.00         1.08          1.00
     RUNOFF VOLUME    (mm)=      46.26        15.02         41.57
     TOTAL RAINFALL   (mm)=      47.26        47.26         47.26
     RUNOFF COEFFICIENT   =        .98          .32           .88

***** WARNING: STORAGE COEFF. IS SMALLER THAN TIME STEP!

       (i) HORTONS EQUATION SELECTED FOR PERVIOUS LOSSES:
            Fo   (mm/hr)= 50.00          K  (1/hr)=  2.00
            Fc   (mm/hr)=  7.50     Cum.Inf.   (mm)=   .00
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

-------------------------------------------------------------------------------
--------------------
| RESERVOIR (0101) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                           .0000      .0000   |    .0790       .0243
                           .0320      .0041   |    .0850       .0284
                           .0450      .0081   |    .0910       .0324
                           .0560      .0122   |    .0960       .0365
                           .0640      .0162   |    .1020       .0405
                           .0720      .0203   |    .0000       .0000

                                AREA      QPEAK       TPEAK       R.V.
                                (ha)      (cms)       (hrs)       (mm)
     INFLOW : ID= 2 (0001)       .900       .318        1.00       43.13
     OUTFLOW: ID= 1 (0101)       .900       .069        1.25       43.11

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 21.71
                   TIME SHIFT OF PEAK FLOW         (min)= 15.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .0188

-------------------------------------------------------------------------------
--------------------
| RESERVOIR (0102) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                           .0000      .0000   |    .0790       .0243
                           .0320      .0041   |    .0850       .0284
                           .0450      .0081   |    .0910       .0324
                           .0560      .0122   |    .0960       .0365
                           .0640      .0162   |    .1020       .0405
                           .0720      .0203   |    .0000       .0000

                                AREA      QPEAK       TPEAK       R.V.
                                (ha)      (cms)       (hrs)       (mm)
     INFLOW : ID= 2 (0002)       .860       .312        1.00       44.69
     OUTFLOW: ID= 1 (0102)       .860       .068        1.25       44.67

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 21.87
                   TIME SHIFT OF PEAK FLOW         (min)= 15.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .0184

-------------------------------------------------------------------------------
--------------------
| RESERVOIR (0103) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                           .0000      .0000   |    .5000       .0270
                           .2888      .0090   |    .5777       .3600
```

```
                     .4085      .0180   |    .6328       .0432

                           AREA      QPEAK       TPEAK       R.V.
                           (ha)      (cms)       (hrs)       (mm)
     INFLOW : ID= 2 (0003)      3.750      1.129        1.00       41.57
     OUTFLOW: ID= 1 (0103)      3.750       .503        1.17       41.57

               PEAK   FLOW   REDUCTION [Qout/Qin](%)= 44.56
               TIME SHIFT OF PEAK FLOW         (min)= 10.00
               MAXIMUM  STORAGE   USED       (ha.m.)=   .0411

     **** ERROR : CHECK THE STORAGE-DISCHARGE TABLE.
--------------------------------------------------------------------------------

--------------------
| ADD HYD   (0201) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0102):       .86    .068     1.25    44.67
        + ID2= 2 (0103):      3.75    .503     1.17    41.57
          ===============================================
          ID = 3 (0201):      4.61    .571     1.25    42.15

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------

--------------------
| ADD HYD   (0200) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0101):       .90    .069     1.25    43.11
        + ID2= 2 (0201):      4.61    .571     1.25    42.15
          ===============================================
          ID = 3 (0200):      5.51    .640     1.25    42.31

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
  ****************************
  ** SIMULATION NUMBER:   2 **
  ****************************

--------------------
| CHICAGO STORM    |    IDF curve parameters: A=4688.000
| Ptotal= 87.07 mm |                          B=  17.000
--------------------                          C=    .962
                        used in:   INTENSITY =  A / (t + B)^C

                        Duration of storm  =  3.00 hrs
                        Storm time step    =  5.00 min
                        Time to peak ratio =   .33

                 TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                  hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
                  .08    4.09 |   .83   50.50 |  1.58   20.97 |  2.33    6.37
                  .17    4.73 |   .92  113.67 |  1.67   17.46 |  2.42    5.82
                  .25    5.57 |  1.00  239.35 |  1.75   14.80 |  2.50    5.34
                  .33    6.68 |  1.08  141.25 |  1.83   12.73 |  2.58    4.93
                  .42    8.21 |  1.17   86.23 |  1.92   11.09 |  2.67    4.56
                  .50   10.40 |  1.25   58.55 |  2.00    9.76 |  2.75    4.24
                  .58   13.73 |  1.33   42.60 |  2.08    8.68 |  2.83    3.95
                  .67   19.18 |  1.42   32.53 |  2.17    7.78 |  2.92    3.70
                  .75   29.10 |  1.50   25.76 |  2.25    7.02 |  3.00    3.47


--------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (0001) |   Area    (ha)=    .90
|ID= 1 DT= 5.0 min |   Total Imp(%)=  90.00   Dir. Conn.(%)=  90.00
--------------------
```

```
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .81          .09
     Dep. Storage     (mm)=       1.00         1.50
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      77.50        40.00
     Mannings n          =       .013         .250

     Max.Eff.Inten.(mm/hr)=     239.35       165.41
                over (min)        5.00         5.00
     Storage Coeff.  (min)=       1.55 (ii)    3.61 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .33          .25
                                                         *TOTALS*
     PEAK FLOW       (cms)=        .53          .04           .571 (iii)
     TIME TO PEAK    (hrs)=       1.00         1.00          1.00
     RUNOFF VOLUME    (mm)=      86.07        49.16         82.38
     TOTAL RAINFALL   (mm)=      87.07        87.07         87.07
     RUNOFF COEFFICIENT   =        .99          .56           .95

***** WARNING: STORAGE COEFF. IS SMALLER THAN TIME STEP!

       (i) HORTONS EQUATION SELECTED FOR PERVIOUS LOSSES:
            Fo   (mm/hr)= 50.00          K  (1/hr)=  2.00
            Fc   (mm/hr)=  7.50     Cum.Inf.   (mm)=   .00
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

-------------------------------------------------------------------------------
--------------------
| CALIB             |
| STANDHYD  (0002) |   Area    (ha)=    .86
|ID= 1 DT= 5.0 min |   Total Imp(%)=  95.00   Dir. Conn.(%)=  95.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .82          .04
     Dep. Storage     (mm)=       1.00         1.50
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      75.70        40.00
     Mannings n          =       .013         .250

     Max.Eff.Inten.(mm/hr)=     239.35       165.41
                over (min)        5.00         5.00
     Storage Coeff.  (min)=       1.52 (ii)    3.06 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .33          .27
                                                         *TOTALS*
     PEAK FLOW       (cms)=        .53          .02           .554 (iii)
     TIME TO PEAK    (hrs)=       1.00         1.00          1.00
     RUNOFF VOLUME    (mm)=      86.07        49.16         84.22
     TOTAL RAINFALL   (mm)=      87.07        87.07         87.07
     RUNOFF COEFFICIENT   =        .99          .56           .97

***** WARNING: STORAGE COEFF. IS SMALLER THAN TIME STEP!

       (i) HORTONS EQUATION SELECTED FOR PERVIOUS LOSSES:
            Fo   (mm/hr)= 50.00          K  (1/hr)=  2.00
            Fc   (mm/hr)=  7.50     Cum.Inf.   (mm)=   .00
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

-------------------------------------------------------------------------------
--------------------
| CALIB             |
| STANDHYD  (0003) |   Area    (ha)=   3.75
|ID= 1 DT= 5.0 min |   Total Imp(%)=  85.00   Dir. Conn.(%)=  85.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=       3.19          .56
     Dep. Storage     (mm)=       1.00         1.50
     Average Slope     (%)=       1.00         2.00
```

```
     Length            (m)=     158.10        40.00
     Mannings n           =       .013         .250

     Max.Eff.Inten.(mm/hr)=     239.35       165.41
                over (min)        5.00         5.00
     Storage Coeff.  (min)=       2.37 (ii)    4.86 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .30          .22
                                                          *TOTALS*
     PEAK FLOW       (cms)=       1.98          .24          2.219 (iii)
     TIME TO PEAK    (hrs)=       1.00         1.00           1.00
     RUNOFF VOLUME    (mm)=      86.07        49.16          80.53
     TOTAL RAINFALL   (mm)=      87.07        87.07          87.07
     RUNOFF COEFFICIENT   =        .99          .56            .92

***** WARNING: STORAGE COEFF. IS SMALLER THAN TIME STEP!

       (i) HORTONS EQUATION SELECTED FOR PERVIOUS LOSSES:
            Fo    (mm/hr)= 50.00           K  (1/hr)=  2.00
            Fc    (mm/hr)=  7.50     Cum.Inf.   (mm)=   .00
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

-------------------------------------------------------------------------------
--------------------
| RESERVOIR (0101) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)      (ha.m.)   |   (cms)      (ha.m.)
                           .0000      .0000   |    .0790       .0243
                           .0320      .0041   |    .0850       .0284
                           .0450      .0081   |    .0910       .0324
                           .0560      .0122   |    .0960       .0365
                           .0640      .0162   |    .1020       .0405
                           .0720      .0203   |    .0000       .0000

                                 AREA      QPEAK       TPEAK       R.V.
                                 (ha)      (cms)       (hrs)       (mm)
     INFLOW : ID= 2 (0001)       .900       .571        1.00       82.38
     OUTFLOW: ID= 1 (0101)       .900       .102        1.33       82.35

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 17.93
                   TIME SHIFT OF PEAK FLOW         (min)= 20.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .0408

-------------------------------------------------------------------------------
--------------------
| RESERVOIR (0102) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)      (ha.m.)   |   (cms)      (ha.m.)
                           .0000      .0000   |    .0790       .0243
                           .0320      .0041   |    .0850       .0284
                           .0450      .0081   |    .0910       .0324
                           .0560      .0122   |    .0960       .0365
                           .0640      .0162   |    .1020       .0405
                           .0720      .0203   |    .0000       .0000

                                 AREA      QPEAK       TPEAK       R.V.
                                 (ha)      (cms)       (hrs)       (mm)
     INFLOW : ID= 2 (0002)       .860       .554        1.00       84.22
     OUTFLOW: ID= 1 (0102)       .860       .100        1.33       84.19

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 18.10
                   TIME SHIFT OF PEAK FLOW         (min)= 20.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .0394

-------------------------------------------------------------------------------
--------------------
| RESERVOIR (0103) |
| IN= 2---> OUT= 1 |
```

```
| DT=  5.0 min     |      OUTFLOW    STORAGE    |  OUTFLOW    STORAGE
--------------------       (cms)     (ha.m.)    |   (cms)     (ha.m.)
                            .0000      .0000    |    .5000      .0270
                            .2888      .0090    |    .5777      .3600
                            .4085      .0180    |    .6328      .0432

                                 AREA      QPEAK       TPEAK       R.V.
                                 (ha)      (cms)       (hrs)       (mm)
     INFLOW : ID= 2 (0003)      3.750      2.219        1.00       80.53
     OUTFLOW: ID= 1 (0103)      3.750       .523        1.33       80.53

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 23.58
                   TIME SHIFT OF PEAK FLOW         (min)= 20.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .1281

     **** ERROR : CHECK THE STORAGE-DISCHARGE TABLE.
--------------------------------------------------------------------------------

--------------------
| ADD HYD   (0201) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0102):      .86    .100     1.33    84.19
        + ID2= 2 (0103):     3.75    .523     1.33    80.53
          ==================================================
          ID = 3 (0201):     4.61    .624     1.33    81.21

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------

--------------------
| ADD HYD   (0200) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0101):      .90    .102     1.33    82.35
        + ID2= 2 (0201):     4.61    .624     1.33    81.21
          ==================================================
          ID = 3 (0200):     5.51    .726     1.33    81.40

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------

 FINISH
=============================================================================================================
```

Box Culvert

ORIFICE DISCHARGE CALCULATOR

This program calculates the discharge from a circular orifice when given elevations and orifice diameters by the user.

Discharge based on orifice equ.: Q = CA x sqrt(2gh)

Prop. Stm. MH 1 (Orifice and Weir Structure)

Orifice Diameter =	**0.450**	m ←	**Enter the orifice diameter in metres**
Area	**0.15904**	m2	
Discharge Coeff. =	**0.820**	←	**Enter discharge coeff. to use**

Elev.	Head	Discharge	Volume	
0.00	0	**0.0000**	**0.0000**	<- Centroid of Orifice
0.25	0.25	**0.2888**	**0.0090**	
0.50	0.50	**0.4085**	**0.0180**	
0.75	0.75	**0.5003**	**0.0270**	
1.00	1.00	**0.5777**	**0.0360**	
1.20	1.20	**0.6328**	**0.0432**	

Enter elevations

DEVICE = 450mm dia. Orifice Tube

BLOCK 1 TANK

ORIFICE DISCHARGE CALCULATOR - BLOCK 1 TANK

This program calculates the discharge from a circular orifice when given elevations and orifice diameters by the user.

Discharge based on orifice equ.: Q = CA x sqrt(2gh)

Block 1

Orifice Diameter =	**0.150**	m ←	**Enter the orifice diameter in metres**
Area	**0.01767**	m2	
Discharge Coeff. =	**0.820**	←	**Enter discharge coeff. to use**

Description	Actual Elevation	Stage	Discharge	Volume	
Centroid of Orifice	0.00	0	**0.0000**	0	0.0000
	0.25	0.25	**0.0321**	40.5	0.0041
	0.50	0.50	**0.0454**	81.0	0.0081
	0.75	0.75	**0.0556**	121.5	0.0122
	1.00	1.00	**0.0642**	162.0	0.0162
	1.25	1.25	**0.0718**	202.5	0.0203
	1.50	1.50	**0.0786**	243.0	0.0243
	1.75	1.75	**0.0849**	283.5	0.0284
	2.00	2.00	**0.0908**	324.0	0.0324
	2.25	2.25	**0.0963**	364.5	0.0365
	2.50	2.50	**0.1015**	405.0	0.0405

Parking Spot		
Width	3	m
Length	6	m
Height	2.5	
Tank Volume	45	
Parking Spots	9	
Total Volume	405	

BLOCK 2 TANK

ORIFICE DISCHARGE CALCULATOR

This program calculates the discharge from a circular orifice when given elevations and orifice diameters by the user.

Discharge based on orifice equ.: Q = CA x sqrt(2gh)

Block 2

Orifice Diameter =	**0.150**	m ←	**Enter the orifice diameter in metres**
Area	**0.01767**	m2	
Discharge Coeff. =	**0.820**	←	**Enter discharge coeff. to use**

Description	Actual Elevation	Stage	Discharge	Volume	Total Volume
Centroid of Orifice	0.00	0	**0.0000**	0	
	0.25	0.25	**0.0321**	40.5	
	0.50	0.50	**0.0454**	81.0	
	0.75	0.75	**0.0556**	121.5	
	1.00	1.00	**0.0642**	162.0	
	1.25	1.25	**0.0718**	202.5	
	1.50	1.50	**0.0786**	243.0	
	1.75	1.75	**0.0849**	283.5	
	2.00	2.00	**0.0908**	324.0	
	2.25	2.25	**0.0963**	364.5	
	2.50	2.50	**0.1015**	405.0	

Parking Spot		
Width	3	m
Length	6	m
Height	2.5	
Tank Volume	45	
Parking Spots	9	
Total Volume	405	

5.5.1.1 RAINFALL INTENSITY (I)

The intensity of rainfall shall be determined based on City of Guelph IDF Curve data (Table 1/Figure 4) using the following equations:

Table 1 – Rainfall Equations

Return Period	Equation of Curve
2 Year	$I = \frac{743}{(td + 6)^{0.7989}}$
5 year	$I = \frac{1593}{(td + 11)^{0.8789}}$
10 Year	$I = \frac{2221}{(td + 12)^{0.9080}}$
25 Year	$I = \frac{3158}{(td + 15)^{0.9355}}$
50 Year	$I = \frac{3886}{(td + 16)^{0.9495}}$
100 Year	$I = \frac{4688}{(td + 17)^{0.9624}}$

The initial time of concentration (td) shall be 5 minutes in all cases, except single family residential unit and park areas, where td shall be 10 minutes.

HEC-RAS DATA

HEC-RAS Plan: Plan 01 River: RIVER-1 Reach: Reach-2

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Reach-2	21	100 Year G&M	30.10	323.20	325.90		325.90	0.000029	0.26	258.55	142.71	0.05
Reach-2	21	Regional GRCA	33.10	323.20	325.93		325.93	0.000034	0.28	261.80	143.31	0.05
Reach-2	20	100 Year G&M	30.10	322.80	325.90		325.90	0.000020	0.24	257.97	109.13	0.04
Reach-2	20	Regional GRCA	33.10	322.80	325.92		325.92	0.000024	0.26	260.36	109.44	0.05
Reach-2	19	100 Year G&M	32.10	322.50	325.90		325.90	0.000004	0.12	569.91	225.63	0.02
Reach-2	19	Regional GRCA	35.80	322.50	325.92		325.92	0.000005	0.13	574.83	225.98	0.02
Reach-2	18	100 Year G&M	32.10	322.30	325.90		325.90	0.000011	0.19	472.18	284.35	0.03
Reach-2	18	Regional GRCA	35.80	322.30	325.92		325.92	0.000013	0.21	478.33	285.76	0.04
Reach-2	17	100 Year G&M	31.60	323.25	325.90	325.59	325.90	0.000002	0.10	640.76	387.85	0.02
Reach-2	17	Regional GRCA	35.80	323.25	325.92	325.59	325.92	0.000002	0.11	648.22	389.23	0.02
Reach-2	16.5 Watson Road		Culvert									
Reach-2	16.1	100 Year G&M	31.60	322.78	325.00	325.00	325.00	0.000036	0.41	177.43	144.52	0.09
Reach-2	16.1	Regional GRCA	35.80	322.78	325.00	325.00	325.00	0.000046	0.46	177.43	144.52	0.10
Reach-2	16	100 Year G&M	31.60	322.50	323.70		323.75	0.001639	1.83	40.15	75.17	0.53
Reach-2	16	Regional GRCA	35.80	322.50	323.78		323.82	0.001566	1.86	46.13	86.70	0.53
Reach-2	15	100 Year G&M	31.60	322.30	323.65		323.69	0.002727	1.82	46.30	76.40	0.50
Reach-2	15	Regional GRCA	35.80	322.30	323.72		323.76	0.002653	1.86	52.36	84.70	0.50
Reach-2	14.5	100 Year G&M	31.60	322.24	323.52		323.62	0.005600	2.52	31.54	53.14	0.71
Reach-2	14.5	Regional GRCA	35.80	322.24	323.59		323.70	0.005945	2.69	35.29	60.73	0.74
Reach-2	14.4	100 Year G&M	31.60	322.11	323.49		323.53	0.002526	1.78	44.69	65.51	0.48
Reach-2	14.4	Regional GRCA	35.80	322.11	323.56		323.60	0.002595	1.86	49.25	70.39	0.49
Reach-2	14.3	100 Year G&M	31.60	321.96	323.44		323.47	0.001856	1.60	52.77	82.02	0.42
Reach-2	14.3	Regional GRCA	35.80	321.96	323.51		323.54	0.001812	1.63	58.53	84.72	0.42
Reach-2	14.2	100 Year G&M	31.60	321.83	323.32		323.40	0.003830	2.30	35.66	59.24	0.60
Reach-2	14.2	Regional GRCA	35.80	321.83	323.39		323.47	0.003703	2.34	40.03	61.91	0.60
Reach-2	14.1	100 Year G&M	31.60	321.56	323.19		323.27	0.003042	2.18	36.80	50.18	0.55
Reach-2	14.1	Regional GRCA	35.80	321.56	323.25		323.34	0.003244	2.31	40.15	54.44	0.57
Reach-2	14	100 Year G&M	33.20	321.30	323.14		323.18	0.001476	1.65	66.07	117.04	0.39
Reach-2	14	Regional GRCA	37.50	321.30	323.22		323.25	0.001312	1.60	74.95	118.73	0.37
Reach-2	13	100 Year G&M	33.20	320.50	323.09	321.44	323.10	0.000134	0.49	95.97	110.99	0.11
Reach-2	13	Regional GRCA	37.50	320.50	323.16	321.48	323.17	0.000153	0.54	100.34	114.68	0.11
Reach-2	12	100 Year G&M	33.20	320.30	323.09		323.09	0.000075	0.52	195.56	125.46	0.10
Reach-2	12	Regional GRCA	37.50	320.30	323.15		323.16	0.000085	0.56	203.98	127.11	0.11
Reach-2	11	100 Year G&M	33.20	320.20	323.08	321.61	323.09	0.000110	0.63	157.53	97.09	0.12
Reach-2	11	Regional GRCA	37.50	320.20	323.15	321.71	323.16	0.000127	0.68	164.00	98.83	0.13
Reach-2	10.5 Watson Parkway		Bridge									
Reach-2	10	100 Year G&M	33.20	319.30	322.84	320.50	322.85	0.000039	0.43	223.12	102.96	0.07
Reach-2	10	Regional GRCA	37.50	319.30	322.92	320.50	322.93	0.000045	0.47	231.39	104.59	0.08
Reach-2	9	100 Year G&M	33.20	318.80	322.84		322.84	0.000040	0.40	206.50	74.98	0.06
Reach-2	9	Regional GRCA	37.50	318.80	322.92		322.92	0.000047	0.44	212.49	75.81	0.07
Reach-2	8	100 Year G&M	33.20	318.80	322.84		322.84	0.000040	0.40	206.50	74.98	0.06
Reach-2	8	Regional GRCA	37.50	318.80	322.92		322.92	0.000047	0.44	212.49	75.81	0.07
Reach-2	7	100 Year G&M	34.10	317.00	322.84	319.01	322.84	0.000003	0.15	432.64	256.72	0.02
Reach-2	7	Regional GRCA	38.60	317.00	322.92	319.11	322.92	0.000003	0.16	449.13	257.16	0.02
Reach-2	6.5 Railway Bridge		Bridge									
Reach-2	6	100 Year G&M	34.10	317.00	319.74	318.91	319.76	0.000380	1.02	66.64	70.69	0.20
Reach-2	6	Regional GRCA	38.60	317.00	320.04	318.96	320.06	0.000230	0.85	90.27	84.08	0.16
Reach-2	5	100 Year G&M	34.10	316.60	319.74		319.75	0.000138	0.63	166.89	141.82	0.11
Reach-2	5	Regional GRCA	38.60	316.60	320.05		320.05	0.000096	0.56	212.30	153.57	0.10
Reach-2	4	100 Year G&M	34.10	316.60	319.74		319.75	0.000138	0.63	166.86	141.81	0.11
Reach-2	4	Regional GRCA	38.60	316.60	320.05		320.05	0.000096	0.56	212.28	153.57	0.10
Reach-2	3	100 Year G&M	34.10	316.60	319.74	318.91	319.74	0.000073	0.73	166.45	141.68	0.13
Reach-2	3	Regional GRCA	38.60	316.60	320.04	319.00	320.05	0.000051	0.65	211.96	153.57	0.11
Reach-2	2.5 York Road		Bridge									
Reach-2	2	100 Year G&M	34.10	316.20	318.56	318.56	319.74	0.004610	4.82	7.08	43.67	1.00

HEC-RAS Plan: Plan 01 River: RIVER-1 Reach: Reach-2 (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Reach-2	2	Regional GRCA	38.60	316.20	318.76	318.76	320.05	0.004491	5.02	7.69	52.24	1.00
Reach-2	1	100 Year G&M	34.10	316.10	316.66	316.66	316.88	0.029691	3.90	19.46	45.22	1.67
Reach-2	1	Regional GRCA	38.60	316.10	316.70	316.70	316.93	0.029016	4.04	21.34	46.41	1.67

ODAN·DETECH CONSULTING ENGINEERS

STORM SEWER DESIGN

RAINFALL: 5 YEAR DESIGN STORM $i = \dfrac{1593}{(Tc+11)^{0.8789}}$

PROJECT:
PROJECT No.:
LOCATION:
MUNICIPALITY:

DESIGN BY:
CHECKED BY:
DATE:

Vmin = 1.2m/s

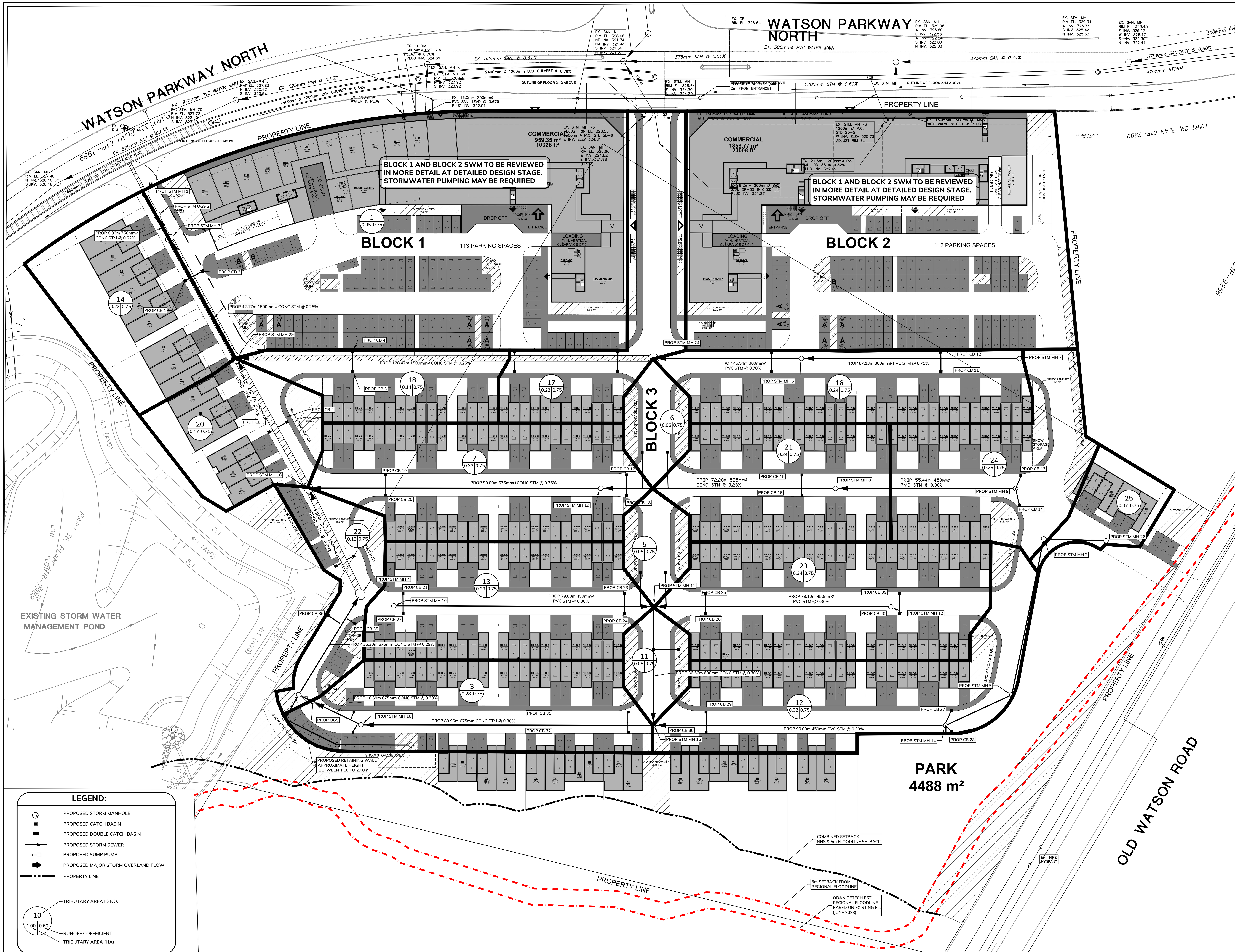
PIPE ROUGHNESS: 0.013 n For Manning's Equation
% of Full Flow: Peak Flow / Full Flow Capacity

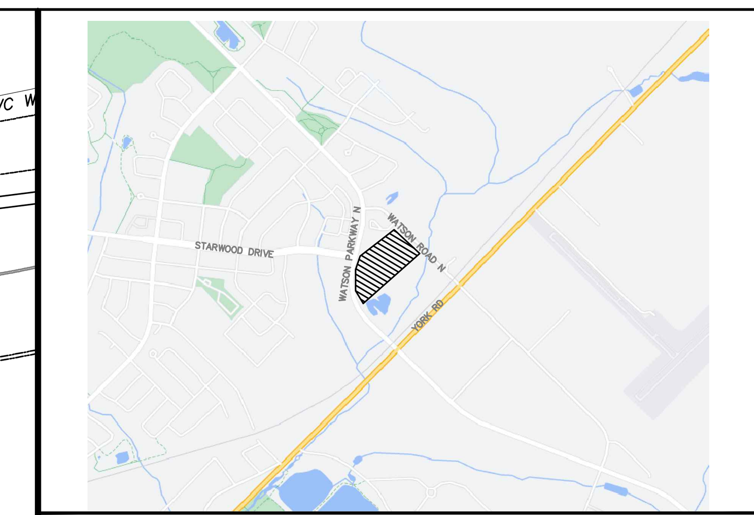
LOCATION				STORMWATER ANALYSIS Mountaingate Subdivision										STORM SEWER DATA					
Tributary ID No.	STREET NAME	From Manhole	To Manhole	A Area (ha)	C Runoff Coeff.	A*C	Accumulated A*C	Time of Concentration (min)	Acc. Time of Concentration (min)	5 Yr Rainfall Intensity (mm/hr)	5 Yr Peak Flow (L/s)	Flow Time (min)	Pipe Length (m)	Pipe Height/ Diameter (mm)	Pipe Slope (%)	Pipe Full Flow Capacity (l/s)	Pipe Full Flow Velocity (m/s)	Percent of Full Flow Capacity (%)
23		12	11	0.340	0.75	0.255	0.255	5.00	6.24	139.29	99	1.24	73.1	450	0.3	156	0.98	63%
13		10	11	0.290	0.75	0.218	0.218	5.00	6.36	139.29	84	1.36	79.9	450	0.3	156	0.98	54%
11+5		11	15	0.10	0.75	0.075	0.548	6.36	6.87	129.68	197	0.51	36.6	600	0.3	336	1.19	59%
25		26	2	0.07	0.75	0.053	0.053	5.00	5.43	139.29	20	0.43	19.2	300	0.3	53	0.75	38%
12		2	5	0.00	0.75	0.000	0.053	5.43	6.26	136.10	20	0.83	49.0	450	0.3	156	0.98	13%
12		5	14	0.00	0.75	0.000	0.053	6.26	7.09	130.31	19	0.83	49.0	450	0.3	156	0.98	12%
12		14	15	0.32	0.75	0.240	0.293	7.09	7.47	125.03	102	0.38	22.5	450	0.3	156	0.98	65%
3		15	16	0.28	0.75	0.210	1.050	7.09	8.26	125.03	365	1.17	90.0	675	0.30	460	1.29	79%
3		16	OGS	0.00	0.75	0.000	1.050	8.26	8.47	118.36	345	0.22	16.69	675	0.3	460	1.29	75%
22		OGS	226	0.00	0.75	0.000	1.050	8.47	8.53	117.20	342	0.06	4.6	675	0.3	460	1.29	74%
22		226	4	0.12	0.75	0.090	1.140	8.53	9.01	116.88	370	0.48	36.3	675	0.29	453	1.26	82%
22		4	18	0.00	0.75	0.000	1.140	9.01	9.30	114.42	363	0.29	36.5	1500	0.28	3740	2.12	10%
24		9	8	0.25	0.75	0.188	0.188	5.00	5.94	139.29	73	0.94	55.4	450	0.3	156	0.98	46%
6+21		8	19	0.30	0.75	0.225	0.413	5.94	7.21	132.46	152	1.26	72.3	525	0.23	206	0.95	74%
		19	18	0.33	0.75	0.248	0.660	7.21	8.28	124.34	228	1.08	90	675	0.35	497	1.39	46%
20		18	29	0.17	0.75	0.128	1.928	9.01	9.39	114.42	613	0.38	45.77	1500	0.25	3534	2.00	17%
16		7	6	0.24	0.75	0.180	0.368	5.00	5.74	139.29	142	0.74	67.1	450	0.71	240	1.51	59%
		6	24	0.00	0.75	0.000	0.368	5.74	6.25	133.86	137	0.51	45.5	450	0.7	239	1.50	57%
17+18		24	29	0.37	0.75	0.278	0.645	6.25	7.32	130.40	234	1.07	128.5	1500	0.25	3534	2.00	7%
		29	3	1.18	0.75	0.885	3.458	9.01	9.36	114.42	1100	0.35	42.2	1500	0.25	3534	2.00	31%
		3	OGS2	0.00	0.75	0.000	3.458	9.36	9.42	112.69	1083	0.06	8.0	900	0.62	1425	2.24	76%
		OGS2	1	0.00	0.75	0.000	3.458	9.42	9.49	112.40	1080	0.06	10.7	900	1	1810	2.85	60%

Project: 11208 LANCASTER HEIGHTS
File: 22202 storm design sheet.xls

THE ODAN/DETECH GROUP INC.

9/13/2023
Project Engineer: Paul Hecimovic, P. Eng.





KEY PLAN
Scale : N.T.S.

SUBJECT LANDS

NOTE :

THE POSITION OF POLE LINES, CONDUITS, WATERMAINS, SEWERS AND UNDERGROUND AND ABOVE GROUND UTILITIES IS NOT NECESSARILY SHOWN ON THE CONTRACT DRAWINGS, AND WHERE SHOWN, THE ACCURACY OF THE POSITION OF SUCH UTILITIES AND STRUCTURES IS NOT GUARANTEED. BEFORE STARTING THE WORK THE CONTRACTOR SHALL INFORM HIMSELF OF THE EXACT LOCATION OF ALL UTILITIES AND STRUCTURES, AND SHALL ASSUME ALL LIABILITY FOR DAMAGE TO THEM.

THE CONTRACTOR MUST CHECK AND VERIFY ALL DIMENSIONS ON THE JOB AND REPORT ANY DISCREPANCY TO THE ARCHITECTS/ENGINEERS BEFORE PROCEEDING WITH THE WORKS.

ALL DRAWINGS AND SPECIFICATIONS ARE INSTRUMENTS OF SERVICE AND THE PROPERTY OF THE ENGINEER WHICH MUST BE RETURNED AT THE COMPLETION OF WORK.

THIS DRAWING IS NOT TO BE SCALED. CONTRACTOR TO USE DIGITAL FILES FOR LAYOUT PROVIDED BY ENGINEER.

THIS PLAN MUST NOT BE USED TO SITE THE PROPOSED BUILDINGS.

THE APPROVAL OF THIS PLAN DOES NOT EXEMPT THE OWNER'S CONTRACTOR FROM OBTAINING, BUT NOT LIMITED TO THE FOLLOWING PERMITS: ROAD CUT, SEWER PERMITS, RELOCATION OF SERVICES, ENCROACHMENT AGREEMENTS, APPROACH APPROVAL PERMITS, ETC...

EXISTING TOPOGRAPHICAL INFORMATION SUPPLIED BY SCHAEFFER DZALDOV BENNETT LTD., OLS.

BOUNDARY DATA DERIVED FROM INFORMATION FROM SCHAEFFER DZALDOV BENNETT LTD., OLS.

BENCH MARK:

ELEVATIONS SHOWN HEREON ARE GEODETIC AND ARE REFERRED TO CITY OF GUELPH BENCHMARK No. 413 HAVING A PUBLISHED ELEVATION OF 330.034 METRES

DURING THE SURVEY WE ATTEMPTED TO UNCOVER ALL SURFACE FEATURES, HOWEVER, WE ARE NOT LIABLE FOR ANY SUCH FEATURES THAT WERE COVERED BY SNOW OR ICE AT THE TIME OF THE SURVEY.

BEARING NOTE:

METRIC NOTE:

DISTANCES AND ELEVATIONS ON THIS PLAN ARE TYPICALLY SHOWN IN METRES AND CAN BE CONVERTED TO FEET BY DIVIDING BY 0.3048.

NO.	REVISIONS	DATE	BY
3	ISSUED FOR ZBA	OCT 19/23	MHH
2	ISSUED FOR REVIEW & COORDINATION	SEPT 13/23	MHH
1	ISSUED FOR REVIEW & COORDINATION	JUNE 15/23	MHH

SCALE : 1:500 (0 5 10 20 30 40 50)

DRAWING :
STORM DRAINAGE AREA PLAN

CLIENT :
GUELPH WATSON HOLDINGS INC.

PROJECT :
GUELPH WATSON HOLDINGS INC.
115 WATSON PARKWAY NORTH
GUELPH, ON

ODAN·DETECH CONSULTING ENGINEERS
The Odan/Detech Group Inc. P: (905) 632-3811 F: (905) 632-3363
5230 SOUTH SERVICE ROAD, BURLINGTON, ONTARIO, L7L 5K2

SCALE :	PROJ. NO.:	DATE STARTED:	DESIGN BY:
1:500	22202	MAR/23	M.H.H.

DRAWN BY: A.M.
CHECKED BY: M.H.H.
APPROVED BY: P.H.

LICENSED PROFESSIONAL ENGINEER
Oct 19/23
P.J. HECIMOVIC
100126980
PROVINCE OF ONTARIO

ENGINEER | FILE NUMBER

SWM

APPENDIX E

CITY OF GUELPH CRITERIA

City of Guelph

Community Design & Development Services

date:	27 May 2022	pages including cover 1	
to:	**Mark Harris**	cc:	Rajan Philips, P.Eng.
company:	**The Odan/Detech Group Inc.**		
email:	**mark@odandetech.com**		
from:	Kime D. Toole, C.E.T. kime.toole@guelph.ca	title:	Engineering Technologist II
department:	Engineering	division:	Development Services
phone:	(519) 837-5604 x 2250	fax:	(519) 822-6194

re: **STORMWATER MANAGEMENT CRITERIA for: 115 Watson Parkway N (± 6.45 ha)**

NOTE: *The following information is supplied to aid in the engineering or design of a project and is not all-inclusive. The applicant is advised to contact all relevant Departments and Agencies to determine the requirements which pertain to a specific site.*

- The City of Guelph's allowable outlet rate is **0.798 m^3/s -** City of Guelph Q_{5yr} design storm.
- Sites that do not have a positive outlet must be designed to provide storage on site for twice the five year design storm runoff volume.
- On site control and storage (roof top/parking lot/ponds/superpipes) may be required to attenuate flows
- For commercial, institutional and high density residential developments, excess runoff for the two year design storm is to be stored underground or on roof tops.
- Excess runoff from the five year design storm may pond in parking areas of least anticipated use to a maximum depth of 0.3m.
- Major storms are to be routed overland to the City's R.O.W. without exceeding a maximum parking lot pond depth of 0.3m. Sites which cannot meet these criteria are required to provide storage on the site for twice the five year design storm runoff volume.
- Clean runoff (roof water) must be directed to pervious areas for infiltration to encourage ground water recharge
- Infiltration devices are acceptable for the drainage of grassed and roof areas as long as they provide an overflow connection to the storm sewer and are fully infiltrated within a 24 hour time period. Soils reports must be provided along with the design of infiltration facilities, indicating "k" values (co-efficient of permeability) for soils and confirmation of at least 1.0m separation between the bottom of the infiltration gallery/drywell and ground water table/bedrock elevation.
- Quality control facilities are required to remove suspended solids (oil and grit) from areas draining driveways and parking lots (i.e. oil/grit interceptors, catch basins and vegetative buffer strips or a combination thereof). Please note that Goss traps are not acceptable for areas larger than $250m^2$.
- The minimum acceptable water quality level for discharge to the municipal collection system is 70% TSS removal.
- The SWM report must include an erosion and sedimentation control plan to be employed during construction of the project.
- Existing overland drainage patterns from adjoining properties must be maintained and shown on the submitted drawing.
- A Professional Engineer must certify the design and construction of the SWM facility.

We require that the SWM modelling be submitted in Miduss98 format using the Horton Equation as this enables our office to complete our review in a timely fashion. The SWM Report is to show system performance for the 5 year and 100 year design storms and must include scale drawings showing drainage catchment areas, delineated pond limits for the 5yr and 100yr design storms (where applicable) and a schematic diagram reflecting the model (complex models). City of Guelph design storm hyetographs and Miduss stormwater modelling parameters for the design storms and Miduss Guelph design storm electronic files are available upon request.

Please call if you require further information or if you have any questions regarding these criteria.

Facsimile

Original Will Follow: Yes ___ No _***X***_

This fax message is intended only for the person or entity to which it is addressed and contains information that is privileged and confidential. If you receive this in error, please notify us immediately by telephone.

APPENDIX F

ODAN/DETECH GROUP ENGINEERING DRAWINGS

CONCEPT SITE SERVICING

CONCEPT SITE GRADING

CONCEPT SECTIONS

HEC-RAS FLOODPLAIN & SECTIONS